

Three-dimensional model for nonlinear analysis of slender flanged reinforced concrete walls

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ARTICLE INFO

Keywords:

Flanged reinforced concrete walls
Flexure-dominated response
OpenSees
Threedimensional analysis
Multidirectional loading
Reinforced concrete buildings

ABSTRACT

A new, three-dimensional, four-node, macroscopic model element for nonlinear analysis of reinforced concrete walls is developed and implemented in the structural analysis software OpenSees. The two-dimensional, beam-column formulation of the Multiple-Vertical-Line-Element Model (MVLEM) is extended by implementing the geometrical transformation of the element degrees of freedom that convert the model element from a two-node to a four-node element, and by incorporating linear-elastic element out-of-plane behavior. The three-dimensional analytical model developed, namely the MVLEM-3D, is validated based on load-deformation responses and vertical strains obtained from tests on one T-shaped wall specimen tested under uniaxial cyclic loading and four U-shaped wall specimens tested under complex multidirectional loading protocols. In addition, results of sensitivity studies on model geometric discretization and selection of an effective shear stiffness are presented. The model exhibits a high level of numerical stability and computational efficiency for all specimens investigated. Comparisons of experimental and analytical results demonstrate that the proposed MVLEM-3D captures, with good accuracy, the cyclic load-displacement behavior of non-planar walls in loading directions that are parallel to the principal axes of the cross-section. However, the analysis results overestimate the wall lateral strength in diagonal (relative to the principal axes) loading directions by 20–50%. This discrepancy is associated with the inability of the plane-sections hypothesis implemented in the model to capture the experimentally-observed nonlinear strain distributions developing along the wall cross-section due to shear lag effect. Despite this issue, the proposed model is shown to be a powerful tool for practical applications, as nonlinear response history analysis is becoming more common in engineering practice.

1. Introduction

Reinforced concrete (RC) structural walls are very effective in resisting lateral loads imposed on buildings by wind or earthquakes, due to their significant stiffness and strength. Thus, they are one of the most widely-used lateral load resisting elements used in buildings worldwide. RC walls are most commonly slender, with height-to-length ratio larger than 2.0. Slender RC walls are typically designed to have sufficient shear strength to ensure essentially linear elastic shear behavior and to experience nonlinear flexural deformations due to yielding of vertical (flexural) reinforcement under strong ground shaking (e.g., Maximum Considered Earthquake). The lateral deformation capacity of RC walls is achieved by seismic detailing of the flexural compression zone, with transverse reinforcement at the wall boundaries that enables a ductile

flexural response. RC walls are characterized with a variety of cross-sectional shapes, ranging from simple rectangular (planar) walls, which resist primarily in-plane lateral loads, to complex flanged walls (e.g., T-shaped, U-shaped, I-shaped) that can resist multidirectional lateral loads. Non-planar walls with complex cross-sections (e.g., core walls) are very common around the perimeter of elevator and stair openings of a building.

Building design and evaluation methodologies that target a desired seismic structural performance (performance-based seismic design) have been widely applied in recent years to new and existing RC buildings. The performance-based design approach heavily relies on results of nonlinear response history analyses of complete three-dimensional building models to obtain engineering demand parameters that are compared against a set of acceptance criteria (e.g.,

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<https://doi.org/10.1016/j.engstruct.2021.112105>

Received 22 September 2020; Received in revised form 11 January 2021; Accepted 19 February 2021

Available online 31 March 2021

0141-0296/© 2021 The Author(s).

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LATBSDC, TBI-II, ASCE-41) to judge if the structural response is acceptable. Therefore, there are two key components to the development and implementation of performance-based seismic design: 1) the availability of experimental data for various structural components (e.g., columns, beams, walls) that are used to develop appropriate acceptance criteria and analysis tools, and 2) the availability of robust and computationally-efficient nonlinear analytical models that can capture the important response characteristics of various structural elements under seismic actions.

In the case of RC walls, a large volume of experimental data from tests on planar RC wall components subjected to in-plane loading has been generated and documented over the past 20 years (e.g., [2]). However, the amount of test data obtained from experiments on flanged (non-rectangular) RC wall specimens tested under biaxial (multidirectional) loading, which is the most common condition for walls in taller buildings, is significantly more sparse, with only few well-documented experimental studies available in the literature (e.g., [18,17,51,7,6,25,11]). Naturally, the absence of experimental results for non-rectangular walls resulted in fewer analytical studies that dealt with development and validation of models that can simulate their three-dimensional behavior. In addition, complex interactions between structural components in the building system are typically not considered in engineering design. However, coupling between RC walls via slabs and beams can lead to unexpected force transfer mechanisms that can lead to increased demands on wall components in the building structural system. This may lead to significant unexpected damage to RC walls, as reported after the 2010 Chile earthquake [9] and 2011 Christchurch earthquake [13]. Currently there is only limited experimental data that provide information on interactions between the walls and the surrounding structural elements (e.g., [19,30]), and analytical RC wall models that are suitable for investigation of this problem are also sparse.

Commonly used nonlinear models for seismic analysis of flexure-controlled RC walls are macroscopic models based on fiber section discretization (i.e., so-called fiber models), which are available in popular research-oriented and commercial software. Prominent examples of these models include the Multiple-Vertical-Line-Element-Model (MVLEM [33,20]), the displacement-based beam-column element [12] and the force-based beam-column element [42] implemented in OpenSees [28], the nonlinear fiber beam-column element available in SeismoStruct [40], and the shear wall element available in Perform 3D [11]. Existing fiber-based models have been proven to be effective and accurate numerical tools for nonlinear analysis of planar, slender (flexure dominated), and isolated RC walls subjected to unidirectional loading

[32,35,20,6]. However, there are not many analytical studies that have investigated the application of these models to three-dimensional and/or system-level problems, which can be contributed to the lack of experimental data as well as the limited practicality of using fiber-based models in analysis of three-dimensional walls or wall systems.

Many fiber-based models are implemented as two-node elements (e.g., displacement/force-based beam-column elements available in OpenSees or SeismoStruct), which requires the use of additional embedded beam elements to connect the surrounding structural components (e.g., beams, slabs, wall spandrels, outrigger walls) to the RC wall elements; Fig. 1b illustrates the application of embedded beams for a typical configuration of a RC core wall system (wall piers connected by coupling beams, Fig. 1a). Depending on the model geometry and layout, the use of embedded beams makes the modeling process of a complete three-dimensional building system cumbersome, and it introduces additional arbitrary modeling parameters (e.g., stiffness of embedded beams) that can affect the numerical stability, efficiency, and accuracy of the analysis. For example, Beyer et al. [6] demonstrated considerable sensitivity of predicted load-deformation behavior of U-shaped walls subjected to cyclic biaxial loading to stiffness properties and configuration of the embedded beams. Even commercial analysis packages, such as Perform 3D, require the use of embedded beams when connecting beam elements to the shear wall elements (see Perform 3D user manual), making the application of this model somewhat impractical. Currently, in the literature, there is a very limited number of analytical models for RC walls that overcome this issue (e.g., [46,47]).

The increasing tendency in applications of nonlinear structural analysis and performance-based design and evaluation methodologies is evident over the recent years in regions with considerable seismicity. Therefore, it is essential that practical three-dimensional nonlinear models for RC walls that can simulate the behavior of non-rectangular (flanged) RC walls under multidirectional earthquake loading, also considering the interaction between the walls and the connecting structural components, are available for design and research applications.

2. Scope and objectives

A research study was conducted to formulate, validate, and incorporate into OpenSees, a computationally stable and efficient three-dimensional nonlinear model for RC walls that addresses the aforementioned shortcomings of currently available analytical models. Specific goals of the research presented in this paper are to:

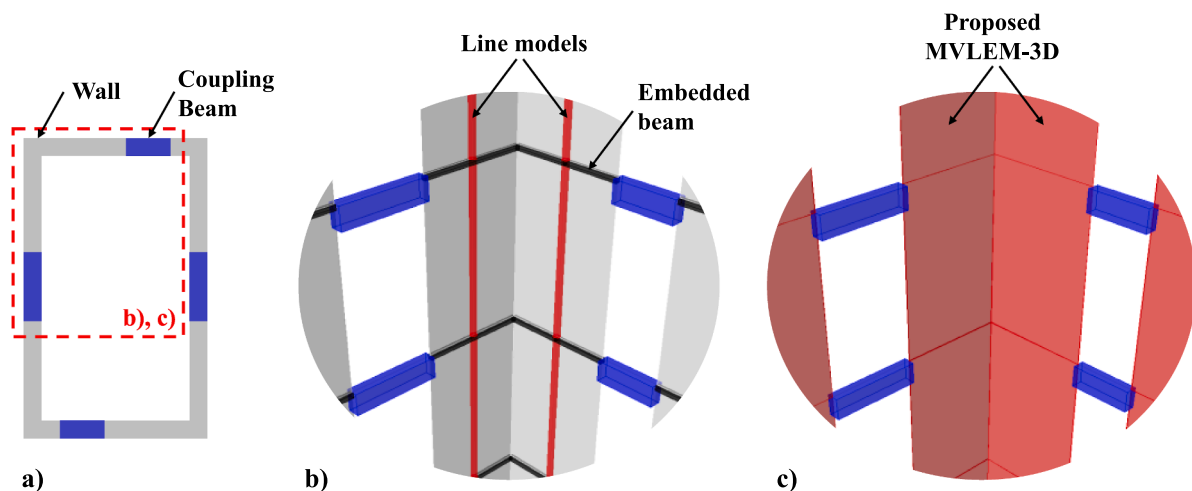


Fig. 1. Conceptual illustration of modeling approaches for connecting beam and wall elements in a structural model: a) plan view of a RC core wall, b) walls modeled with line elements and embedded beams, c) walls modeled with the proposed MVLEM-3D where embedded beams are not needed.

- 1) Extend the current two-dimensional, beam-column formulation of the MVLEM element to a three-dimensional, four-node model element (MVLEM-3D) that can predict the behavior of flexure-dominated RC walls with non-rectangular cross-sections under multidirectional loading and allows direct connectivity with surrounding structural components without the use of embedded beams (conceptually illustrated in Fig. 1c).
- 2) Validate the proposed model formulation using comprehensive test data recorded from quasi-static experiments conducted on slender RC walls with non-rectangular cross-sections subjected to cyclic uni- and multi-directional loading protocols, and
- 3) Implement the developed model into the computational platform OpenSees and provide corresponding user documentation (user manuals and examples).

The proposed analytical model provides the opportunity for improved three-dimensional and/or system-level modeling of RC wall components or building systems, as well as a flexible platform for further model improvements. The presented study focuses on analysis of non-rectangular walls subjected to multidirectional lateral loading using the proposed MVLEM-3D, where an example application of the proposed MVLEM-3D model for response history analysis of a tall RC core wall building can be found in paper by Terzic and Kolozvari [43].

3. Analytical model formulation

Analytical model formulation proposed in this study is the extension of the two-dimensional, two-node Multiple-Vertical-Element-Model (MVLEM; [48,49,33]). In this section, the description of the existing MVLEM is presented and followed by the descriptions of the two extensions of the model baseline formulation to obtain the three-dimensional, four-node MVLEM-3D element: 1) geometric transformation that converts the model element from a two-node to a four-node element, and 2) implementation of the Kirchhoff plate formulation to simulate the out-of-plane response of the element.

3.1. Baseline Two-Dimensional, Two-Node MVLEM formulation

The six in-plane degrees of freedom (DOFs) Ψ_i are defined at the two centerline nodes of the MVLEM to describe the in-plane axial, bending, and shear deformations of the element, including horizontal translational, vertical translational, and a rotational DOFs at each node (Fig. 2a). RC wall cross-section is represented using m longitudinal fibers defined by corresponding areas of concrete and steel reinforcement.

Axial strains at each of the m fibers are derived based on

displacements and rotations along the six element DOFs and the plane sections (Bernoulli-Euler) assumption enforced at the top and bottom of the element via rigid beams (Fig. 2a). It is assumed that relative rotations between these rigid beams occur around the point located at the model centerline at the height of $c \cdot h$ from the bottom element node; c is a parameter that ranges between 0.0 and 1.0 and h is the element height (Fig. 2a). The hysteretic stress-strain behavior of concrete and reinforcing steel materials comprising each element fiber are defined according to the fiber strain history and the uniaxial constitutive models adopted for the two materials, governing the axial and flexural behavior of the model. It is assumed that there is perfect bond between concrete and steel reinforcement, i.e., the same strain is imposed on both materials within each fiber. Note that a modified version of the MVLEM for simulating bond-slip behavior of RC columns with lap splice deficiencies was developed and experimentally-validated by Chowdhury and Orakcal [36,37].

Element shear behavior is represented using the horizontal spring located at $c \cdot h$ from the element bottom node. The deformations of the shear spring are obtained based on the displacements at the six element DOFs, and the behavior of the spring is governed by a hysteretic shear force vs. deformation model adopted. It is important to mention that the behavior of the shear spring is independent from the behavior of the longitudinal element fibers, assuming that axial/flexural behavior is decoupled from shear behavior in the element formulation. Further details on the MVLEM formulation are provided by Orakcal et al. [33].

3.2. Transformation from Two-Node to Four-Node MVLEM element formulation

The in-plane behavior of the proposed MVLEM-3D model element is described using a four-node, quadrilateral MVLEM formulation with twelve DOFs defined at the four element nodes (Δ_k , Fig. 2b), including horizontal and vertical translational DOFs and one rotational (drilling) DOF at each node. Displacements at the twelve DOFs Δ_k of the four-node MVLEM element are obtained from the displacements at the two centerline nodes based on the six DOFs (Ψ_i , Fig. 2a) defined in the parent MVLEM using geometric transformation that assumes rigid offset between the centerline nodes and the corner nodes of the element, as described below.

Six DOFs Ψ_i are converted to the twelve DOFs Δ_k as:

$$\{\Delta\}_{12 \times 1} = [C]_{12 \times 6} \times \{\Psi\}_{6 \times 1} \quad (1)$$

where matrix $[C]$ is the geometric transformation matrix defined as:

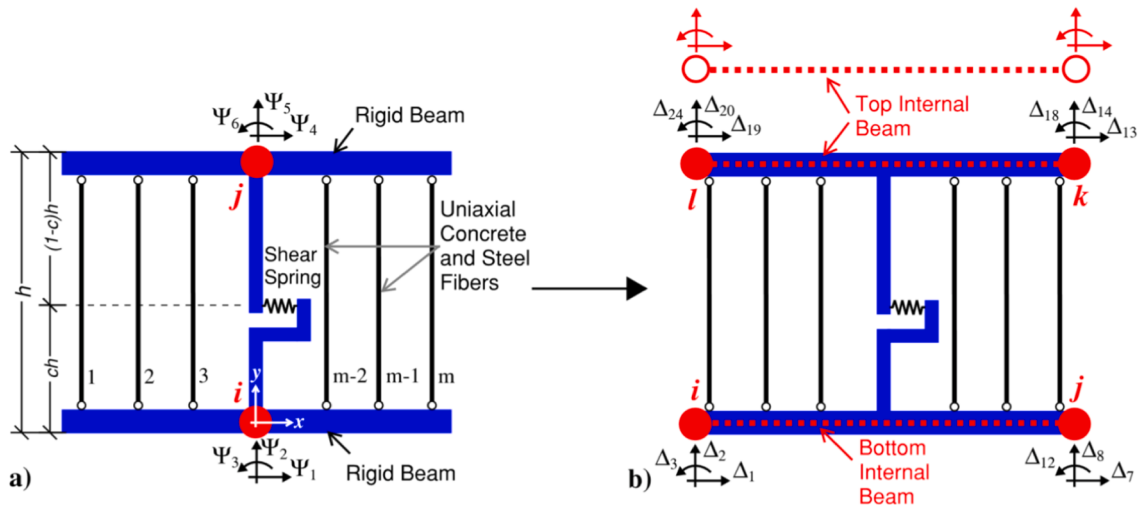


Fig. 2. Transformation of MVLEM from: a) 2-node, 6-DOFs formulation to b) 4-node, 12-DOFs formulation.

$$[C] = \begin{bmatrix} [c_1]_{3 \times 3} & [0]_{3 \times 3} \\ [c_2]_{3 \times 3} & [0]_{3 \times 3} \\ [0]_{3 \times 3} & [c_3]_{3 \times 3} \\ [0]_{3 \times 3} & [c_4]_{3 \times 3} \end{bmatrix}_{12 \times 6} \quad (2)$$

The submatrices $[c_i]$ ($i = [1,4]$) represent rigid offset transformation for each of the four element nodes, and are expressed as:

$$[c_i] = \begin{bmatrix} 1 & 0 & 0 \\ 0 & 1 & x \\ 0 & 0 & 1 \end{bmatrix}_{3 \times 3} \quad (3)$$

where $x = \pm L_w/2$ (horizontal distance of each of the element corner nodes from its centerline, Fig. 2), where L_w is the wall horizontal length.

The in-plane stiffness matrix corresponding to the twelve DOFs Δ_k (K_Δ) is obtained by transforming the element stiffness matrix corresponding to the six DOFs Ψ_i of the original two-node MVLEM formulation (K_Ψ) as:

$$[\hat{K}_\Delta]_{12 \times 12} = [C]^{-1T}_{12 \times 6} \times [K_\Psi]_{6 \times 6} \times [C]^{-1}_{6 \times 12} \quad (4)$$

Subsequently, the rank of the stiffness matrix $\hat{K}_\Delta$ is increased from six to twelve by assigning two linear elastic internal beams (IB) to the four-node formulation of the MVLEM, as illustrated in Fig. 2b. Stiffness properties of the linear elastic internal beams are defined based on the wall geometry and material properties and correspond to half of the wall height ($h/2$). The final stiffness matrix corresponding to the 12 in-plane DOFs Δ_k of the four-node MVLEM formulation (K_Δ) is then calculated as:

$$[K_\Delta]_{12 \times 12} = [\hat{K}_\Delta]_{12 \times 12} + [K_{IB}]_{12 \times 12} \quad (5)$$

where $[K_{IB}]$ is expressed as:

$$[K_{IB}] = \begin{bmatrix} [K_{Bot,Beam}]_{6 \times 6} & [0]_{6 \times 6} \\ [0]_{6 \times 6} & [K_{Top,Beam}]_{6 \times 6} \end{bmatrix}_{12 \times 12} \quad (6)$$

Matrices $[K_{Bot,Beam}]$ and $[K_{Top,Beam}]$ refer to the stiffness matrix of an elastic two-dimensional Bernoulli-Euler beam (e.g., [10]) corresponding to the bottom and top internal beams, respectively.

The in-plane resisting force vector of the four-node MVLEM formulation corresponding to the twelve DOFs Δ_k (R_Δ) is derived using the force vector of the baseline two-node planar MVLEM corresponding to the six DOFs Ψ_i (R_Ψ). Firstly, geometric transformation of the forces is performed as:

$$\{\hat{R}_\Delta\}_{12 \times 1} = [C]^{-1T}_{12 \times 6} \times \{R_\Psi\}_{6 \times 1} \quad (7)$$

Secondly, the internal beam forces are removed from the four-node MVLEM force vector:

$$\{R_\Delta\}_{12 \times 1} = \{\hat{R}_\Delta\}_{12 \times 1} - \{R_{IB}\}_{12 \times 1} \quad (8)$$

where $\{R_{IB}\}$ is expressed as:

$$\{R_{IB}\} = \left\{ \begin{bmatrix} \{R_{Bot,Beam}\}_{6 \times 1} \\ \{R_{Top,Beam}\}_{6 \times 1} \end{bmatrix} \right\}_{12 \times 1} \quad (9)$$

Vectors $\{R_{Bot,Beam}\}$ and $\{R_{Top,Beam}\}$ refer to a resisting force vector of the bottom and top elastic internal beams, respectively (e.g., [10]).

3.3. Element out-of-plane behavior

The behavior of the proposed MVLEM-3D RC wall element in the out-of-plane direction is described with a four-node Kirchhoff plate finite element formulation (e.g., [10]). This standard finite element plate model is based on the following assumptions: 1) material is homogeneous with linear elastic behavior, 2) element thickness is small relative to its other two dimensions and is constant across the element, 3) element out-of-plane displacements are small comparing to its in-plane

dimensions, and 4) shear strain energy is ignored. Element nodal DOFs that correspond to its out-of-plane deformations include one translation and two rotations, resulting in a total of twelve out-of-plane DOFs (Fig. 3b).

3.4. Combination of element in-plane and out-of-plane behavior

Fig. 3 illustrates the assembly of the 12 in-plane DOFs corresponding to the four-node MVLEM formulation (Fig. 3a; $\Delta_1, \Delta_2, \Delta_6, \Delta_7, \Delta_8, \Delta_{12}, \Delta_{13}, \Delta_{14}, \Delta_{18}, \Delta_{19}, \Delta_{20}$ and Δ_{24}) together with the twelve out-of-plane DOFs corresponding to the Kirchhoff plate model (Fig. 3b; $\Delta_3, \Delta_4, \Delta_5, \Delta_9, \Delta_{10}, \Delta_{11}, \Delta_{15}, \Delta_{16}, \Delta_{17}, \Delta_{21}, \Delta_{22}$ and Δ_{23} in Fig. 3b) into a complete three-dimensional MVLEM-3D element with 24 DOFs (Fig. 3c). Terms in the stiffness matrix and force vector that correspond to the in-plane element DOFs are assembled with the corresponding terms of the element out-of-plane behavior to form the complete 24×24 stiffness matrix and 24×1 resisting force vector of the MVLEM-3D element. There is no coupling between the wall element out-of-plane and in-plane behaviors in current model formulation.

4. OpenSees implementation

The presented MVLEM-3D model is implemented in nonlinear analysis software OpenSees [28] as a new element referred to as *MVLEM_3D*. The model element works in the three-dimensional domain with six DOFs per node. The input parameters used to define the model in OpenSees are presented in Table 1. Parameters necessary to define the *MVLEM_3D* element include: 1) the unique element tag (*eleTag*), 2) tags of four element nodes defined in the counterclockwise direction (*iNode jNode kNode lNode*), 3) number of element longitudinal fibers (*m*), 4) the array of *m* fiber thicknesses (*Thicknesses*), 5) the array of *m* fiber widths (*Widths*), 6) the array of *m* fiber reinforcing ratios (*Reinforcing ratios*), 6) the array of *m* uniaxial concrete material tags (*Concrete tags*), 7) the array of *m* uniaxial steel material tags (*Steel tags*), and 8) tag of the uniaxial material shear used for shear behavior (*Sear tag*). Additional optional parameters that can be defined for the *MVLEM_3D* element are: 1) the relative location, measured from the wall base, of the element center of rotation for in-plane bending (*c*, default value is 0.4 per [49]), 2) the element thickness modifier for out-of-plane bending (*tMod*, default value is 0.63, which is equivalent to 25% of uncracked out-of-plane bending stiffness), which can be used to account for reduction of the wall out-of-plane stiffness due to concrete cracking, 3) Poisson ratio used to calculate the element out-of-plane stiffness (*Nu*, default value is 0.25), and 4) and material density (*Dens*, default value is 0.0). The *MVLEM_3D* model is available in public release of OpenSees software, while corresponding documentation (user manual and examples) is available at <https://kkolozvari.github.io/MVLEM-3D/>.

5. Experimental database used for model validation

Validation of the proposed analytical model is conducted using experimental data obtained from tests on one T-shaped wall specimen (TW2, [44]) tested under unidirectional cyclic loading applied parallel to the wall web, as well as four U-shaped wall specimens subjected to multidirectional testing protocols (TUA and TUB [7]; TUC and TUD [11]). The selected specimens are representative of a broad range of wall properties and behaviors under lateral load, which allowed a comprehensive validation of the analytical model proposed. Important properties of the five wall specimens investigated are presented in Table 2.

The T-shaped wall TW2 was 3660 mm tall with web and flange thicknesses and lengths of 102 mm and 1220 mm, respectively, which corresponded to a quarter-scale model of a four-story wall prototype. Boundary elements at wall edges were well-detailed with transverse reinforcement along the bottom 1220 mm of the specimen. Specimen reinforcement was designed to ensure flexure-dominated wall response. The four U-shaped wall specimens considered (TUA, TUB, TUC, TUD)

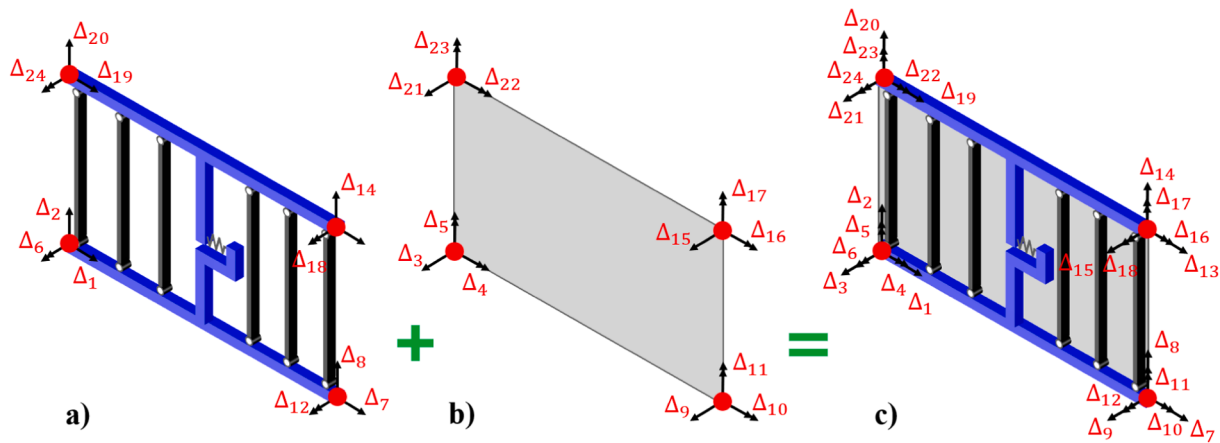


Fig. 3. Formulation of MVLEM-3D: a) element in-plane behavior described with DOFs of the four-node MVLEM formulation, b) element out-of-plane behavior described with 12 DOFs of the four-node Kirchhoff plate model, c) 24 DOFs describing the combined in-plane and out-of-plane behavior.

Table 1

OpenSees user input for MVLEM_3D element.

User input format	element MVLEM_3D eleTag iNode jNode kNode lNode m -thick {Thickesses} -width {Widths} -rho {Reinforcing_ratios} -matConcrete {Concrete_tags} -matSteel {Steel_tags} -matShear {Shear_tag} <-CoR c> <-thickMod tMod> <-Poisson Nu> <-Density Dens>
	eleTag Unique element tag
	iNode jNode kNode lNode Tags of element nodes defined in counterclockwise direction
	m Number of element macro-fibers
	{Thickesses} Array of m macro-fiber thicknesses
	{Widths} Array of m macro-fiber widths
	{Reinforcing_ratios} Array of m reinforcing ratios corresponding to macro-fibers
Description of input parameters	{Concrete_tags} Array of m <i>uniaxialMaterial</i> tags for concrete
	{Steel_tags} Array of m <i>uniaxialMaterial</i> tags for steel
	{Shear_tag} Tag of <i>uniaxialMaterial</i> for shear material
	c Location of center of rotation from the base(optional; default = 0.4 (recommended))
	tMod Thickness modifier for out-of-plane bending behavior (optional; default = 0.63, which is equivalent to 25% of uncracked stiffness)
	Nu Poisson ratio for out-of-plane bending behavior (optional; default = 0.25)
	Dens Element density (optional; default = 0.0)

had similar geometry with flange and web widths of 1050 mm and 1300 mm (outside dimensions), respectively. Specimen TUA had a wall thickness of 150 mm and specimens TUB, TUC, and TUD had wall thicknesses of 100 mm. The longitudinal reinforcement was not spliced and was kept the same for all U-shaped specimens throughout the wall height. For specimens TUA and TUB, the horizontal web reinforcement was identical and the amount of longitudinal reinforcement was approximately the same. For specimens TUC and TUD, wall vertical reinforcement was identical, while the wall horizontal web reinforcement was approximately 25% larger for specimen TUD. For specimens TUA and TUB, both wall flanges had concentrated longitudinal reinforcement at the boundary elements, while for specimens TUC and TUD, one flange was detailed similarly to specimens TUA and TUB, whereas in the other flange and in the web, longitudinal reinforcement was uniformly distributed.

All considered wall specimens were tested as vertical cantilever walls initially subjected to constant axial load, where axial load ratios ranged from low ($0.02A_gf_c$) to moderate ($0.15A_gf_c$) (Table 2), and subsequently subjected to cyclic lateral load applied at the top of the walls. Specimen TW2 was tested under cyclic lateral loading parallel to the wall web applied via a hydraulic actuator that exerted forces at the height of 3810 mm from the wall base, where the loading history consisted of displacement-controlled cycles to 0.1%, 0.25%, 0.5%, 0.75%, 1.0%, 1.5%, 2.0%, and 2.5% drift ratio. For the U-shaped specimens, displacements along the two translational directions and the twisting of the top of the wall were controlled using three actuators: one actuator that applied lateral loads along the wall web (E-W actuator) at 3.35 m from

the foundation, and two actuators that applied loads along the flanges (N-S actuators, one actuator per flange) at 2.95 m from the wall base. All four U-shaped specimens were tested under complex multidirectional horizontal displacement patterns applied at the top of the walls, and twisting of the specimens (torsional deformation) was prevented. The loading pattern of specimens TUA and TUB (Fig. 5a) consisted of cycles in east-west (A-B), north-south (C-D), and diagonal (E-F) directions, as well as a complex O-A-G-D-O-C-H-B-O cycle (sweep) protocol, where maximum applied displacements for each loading cycle corresponded to displacement ductility levels (μ) of 1.0, 2.0, 3.0, 4.0, and 6.0. For specimens TUC and TUD (Fig. 5b), the applied loading history consisted of cycles in east-west and north-south directions for peak drift levels smaller than 1.0% and cycles in diagonal (G-H and E-F) directions for peak drift levels equal to and larger than 1.0%, where the applied peak top displacements during the entire loading history corresponded to drifts levels of 0.1%, 0.2%, 0.3%, 0.4%, 0.6%, 0.8%, 1.0%, 1.5%, 2.0% for both specimens, and 2.5%, and 3.0% for TUC only.

All specimens were extensively instrumented to measure lateral and vertical displacements at various locations (e.g., displacements along wall height, base slip, base uplift, etc.), forces in actuators, and strains at critical locations. Instrumentation on specimen TW2 also included linear variable differential transducers (LVDTs) installed vertically at four locations along the web and five locations along the flange outside face to measure axial strains over a gauge length of 229 mm. In addition, strain gauges were placed at various locations to measure strains in concrete and reinforcement, and wire potentiometers were mounted in an “X” configuration to measure shear deformations over the bottom two

Table 2

Key properties of considered RC wall specimens.

Specimen ID ¹	$\frac{M}{V_w}$ ²	t_w (mm)	f_c ³ (MPa)	f_y ⁴ (MPa)	Boundary Reinforcement ⁵	Web Reinforcement (Vertical / Horizontal)	$\rho_{v,boundary}$ (%)	$\rho_{vert,web}$ (%)	 (%)		Failure Mode
TW2	3.12	102	42.8	#2: 448, #3: 434	Flange Edge: 8#3Web Edge: 8#3 + 2#2Web/Flange: 8#3	Flange: #2@191, Web: #2@140	Flange Edge: 2.93, Web Edge: 1.41, Web/Flange: 2.93	Flange: 0.32, Web: 0.44	Flange: 0.32, Web: 0.44	0.075	Crushing and buckling of longitudinal reinforcement
TUA	2.81 N-S 2.58 E-W	150	77.9	D6: 518, D12: 488	Egde: 7D12Corner: 4D12 + 4D6	D6@150 / D6@125	Egde: 2.10, Corner: 0.84	0.25	0.30	0.02	Fracture of longitudinal reinforcement
TUB	2.81 N-S 2.58 E-W	100	54.7	D6: 518, D12: 471	Egde: 6D12 + 2D6Corner: 4D12 + 4D6	D6@150 / D6@125	Egde: 2.45, Corner: 1.88	0.38	0.45	0.04	Crushing of compression strut in the web
TUC	2.81 N-S 2.58 E-W	100	42	D6: 492D8: 563	SE Egde: 8D8SCorner: 10D8NW	E Flange:D8@100 / D6@125, W Flange: D6@150 / D6@125, Web: D8@100 / D6@125	SE Edge: 1.34, SW Edge: 2.45, NE Corner: 1.25, NW Corner: 1.15	W Flange: 0.31, E Flange: 0.91, Web: 1.0	0.45	0.06	Flange out-of- plane buckling and compression failure
TUD	2.81 N-S 2.58 E-W	100	41.5	D12: 529	W Egde: 6D12 + 2D6NE, Corner: 10D8NW, Corner: 8D8 + 2D6	E Flange: D8@100 / D6@100, W Flange: D6@150 / D6@100, Web: D8@100 / D6@100			0.57	0.15	Crushing of compression strut in the web

¹ TW2 [44]; TUA, TUB [7,6]; TUC, TUD [11].² Shear span to depth ratio.³ Reported compressive strengths obtained from tests on concrete cylinders.⁴ Reported yield strengths obtained from tests on reinforcing bar coupons.⁵ #2 bar ($d_{bar} = 6.35$ mm), #3 bar ($d_{bar} = 9.53$ mm); D6 refers to 6 mm diameter bar, others similar.

stories. For the U-shaped wall specimens, horizontal displacements at the top of the wall and the vertical deformations at the wall boundaries were recorded using LVDTs. Four chains of eight LVDTs were mounted at wall boundaries along the wall height spanning 0–50 mm, 50–150 mm, 150–250 mm, 250–450 mm, 450–850 mm, 850–1250 mm, 1250–1650 mm, and 1650–2650 mm from the wall base. In addition, shear deformations were measured using string pots applied on the outer faces of the web and the flanges between 50 and 850 mm, 850–1650 mm, 1650–2650 mm from the wall base. For specimens TUC and TUD, additional LVDTs were installed on the inner side of each flange boundary to capture gradient of vertical strains through the flange thickness. Additional information on the specimens, the experimental setup, and the loading protocols are presented by Thomsen and Wallace [44], Beyer et al. [7], and Constantin [11].

6. Description of the modeling approach

6.1. Constitutive models and material model calibration

Adopted uniaxial constitutive models for concrete, reinforcing steel, and wall shear response are briefly described in this section, along with the procedure used to calibrate the model parameters. Note that MVLEM-3D allows use of any appropriate uniaxial constitutive model available in OpenSees.

6.1.1. Concrete constitutive behavior

The uniaxial hysteretic behavior of concrete within the longitudinal fibers of the MVLEM-3D is represented with the material model proposed by Yassin [52], i.e., *Concrete02* uniaxial material model available in OpenSees. The material model *Concrete02* is selected in this study because of its relatively simple and numerically efficient formulation that also captures important behavioral characteristics of concrete under cyclic loading, including tension stiffening and hysteretic stiffness degradation. The model compression envelope curve is based on the

Kent and Park [24] and Scott et al. [39] relationships. The ascending branch of the compression envelope follows a quadratic relation, the post-peak descending branch is a linear relation, and the residual stress capacity of concrete is defined as constant. The concrete tension envelope is defined as a tri-linear relationship. It commences with the slope that corresponds to the concrete initial tangent modulus until concrete cracking is reached at the prescribed tensile capacity of concrete. For strains larger than the concrete cracking strain, the tension backbone has a descending slope, which allows capturing the tension stiffening effects in reinforced concrete. Finally, after reaching zero tensile stress, the model assumes zero stiffness henceforth. Unloading and reloading branches of the concrete material model in both tension and compression are described with relatively simple linear or bi-linear strain-stress relationships. This feature of the constitutive model provides its numerical stability and efficiency, but it also fails to simulate gradual concrete gap closure, which is considered to be its primary shortcoming. More details on the model formulation can be found in dissertations by Yassin [52] and Orakcal [31].

The unconfined concrete compression envelope of *Concrete02* was calibrated based on the stress and strain values corresponding to peak compressive concrete strength obtained from concrete cylinder test results reported for the test programs. The confined concrete strain-stress envelope in compression was obtained using the confinement model by Mander et al. [27]. For both unconfined and confined concrete, the slope of the descending branch of the constitutive model envelope was calibrated to comply with that of the Saatcioglu and Razvi [38] model. The concrete tensile strength was calculated as $f_t = 0.31\sqrt{f'_c}$ (MPa), as recommended by Belarbi and Hsu [5]. Tension stiffening effects on the strain-stress behavior of concrete were implemented by adopted the slope of the post-cracking model envelope in tension as 5% of the initial concrete tangent modulus per Yassin's [52] recommendations.

6.1.2. Reinforcing steel constitutive behavior

The behavior of reinforcing steel within the longitudinal fibers of the

MVLEM-3D is modeled using the cyclic material model developed by Menegotto and Pinto [29] and Filippou et al. [14]. In this constitutive model, both pre-yield (elastic) and post-yield branches of the monotonic strain-stress relationship are defined as straight-lines. Under cyclic loading, the pre- and post-yield asymptotes are connected by curved transitions, where the curvature of the transition degrades based on the imposed strain history. This constitutive relationship for reinforcing steel is relatively simple, yet efficient and accurate, and it captures important behavioral characteristics of reinforcing steel bars under cyclic loading, such as Bauschinger's effect. The constitutive model is available in OpenSees in the form of two different uniaxial material models *Steel02* and *SteelMPF*. In this study, the more recently implemented model *SteelMPF* [23] is used because anomalies associated with stress overshooting after partial unloading/reloading present in *Steel02* (see [14]) are corrected in its formulation.

SteelMPF material model was calibrated according to the as-tested reinforcing steel properties for all wall specimens considered. Monotonic model parameters (yield stress and strain hardening ratio) were calibrated based on the yield and ultimate stress values obtained from the coupon tests on reinforcing bars, where the strain-hardening ratio was defined assuming that the post-yield strain-stress relationship is linear between the yield and ultimate stress points. A modulus of elasticity of 200,000 MPa (29,000 ksi) was used for all bar sizes and all specimens. Cyclic stiffness degradation characteristics of hysteretic steel stress-strain behavior were considered via the parameters $R_0 = 20$, $a_1 = 18.5$ ($0.925 \times R_0$), and $a_2 = 0.15$ in the material model, based on recommendations by Menegotto and Pinto [29].

6.1.3. Shear behavior

The in-plane shear response of the proposed MVLEM-3D model is represented with the shear spring (Fig. 2b) that is uncoupled from the element flexural response. Therefore, explicit definition of the shear force-deformation behavior is required, where viable options include use of either a linear elastic model with a specified effective shear stiffness, or a multilinear backbone curve (e.g., ASCE [3]). Kolozvari and Wallace [22] demonstrated that use of an elasto-plastic shear force-deformation relationship can produce unrealistic model behavior during dynamic analysis, where the entire lateral deformation of the model element becomes associated with shear deformation after reaching the prescribed element shear capacity. Therefore, this option is disregarded in this study, and a linear elastic shear force-deformation relationship is selected to represent the shear behavior of the MVLEM-3D elements, which is also the approach most commonly used in engineering practice.

The effective shear stiffness associated with the linear elastic shear force-deformation relationship is an important modeling parameter that can considerably influence the analytically-predicted wall response [22]. Current design provisions and recent research results suggest a relatively wide range of effective shear stiffness values. For example, performance-based evaluation and design guidelines (e.g., [26,41]) recommend the effective shear modulus of $0.5G$ (G is the shear modulus of concrete), whereas PEER/ATC 72 [34] suggests the secant-to-yield shear modulus of $0.05G$ for walls with maximum shear stress capacity smaller or equal to $5\sqrt{f'_c}$, and $0.1G$ for walls with shear stress capacity between 5 and $10\sqrt{f'_c}$. In addition, recent test results on planar medium-rise walls [45] revealed an effective shear stiffness at ultimate lateral load of approximately $0.025G$, while analytical studies on slender walls by Gogus [16] demonstrated that effective shear modulus of $0.025G$ is the most appropriate after shear cracking.

All of the abovementioned values for the effective shear stiffness are derived based on experiments on planar RC walls tested under unidirectional in-plane loading and might not be appropriate for analysis of flanged walls, particularly when they are loaded multidirectionally. Beyer et al. [7] reported that shear deformations recorded on the web and the two flanges of U-shaped RC walls tested under multidirectional loading are greatly influenced by the loading direction, where considerably larger shear deformations were measured on the regions

subjected to resultant (axial/flexural) tension. In addition, Beyer et al. [7,8] observed that the magnitudes of shear deformations were generally larger than what is encountered in typical rectangular walls. Experimental studies by Ji et al. [50], in which the coupled axial tension-shear behavior of RC walls were investigated, have also revealed that axial tension force on a wall cross section can significantly decrease the shear stiffness of the wall. Evaluation of test results reported by Beyer et al. [7] and Constantin [11], considering contributions of shear deformations to wall displacements in various loading directions and various displacement demands, shows that the most appropriate linear elastic shear stiffness to be used with the MVLEM-3D elements for analysis of the U-shaped wall specimens under multidirectional loading is $0.025G$.

6.2. Geometric discretization

Model geometry for the considered wall specimens was discretized in the horizontal direction such that one MVLEM-3D element represented each of the flanges or the web of the wall. Each model element was discretized into a number of uniaxial fibers based on the reinforcement layout in the wall cross-section. To enable consistent comparison between the model results and local test measurements, the heights of the MVLEM-3D elements in the vertical direction were selected to match the locations of the vertical LVDTs used to measure wall vertical strains. Longitudinal reinforcement was assumed to be distributed uniformly (smeared) within each of the boundary and web fibers of the model element, where the corresponding reinforcement ratios were calculated based on the areas of reinforcing steel and concrete in each fiber. Model geometric discretizations used for analysis of specimen TW2 and TUB are illustrated in Fig. 4. All model elements were pinned at the base. All DOFs corresponding to top wall nodes were free for specimen TW2, while for the four U-shaped wall models, the torsional DOF was fixed at the top to replicate the boundary conditions applied during testing, where twisting of the specimens was restricted during multidirectional loading.

6.3. Load application

Analysis of each specimen commenced with applying constant axial load at the top nodes of the wall model (Fig. 4). The magnitudes of axial forces applied to the models (N) are determined such that the resultant axial force corresponds to the average axial load value applied on the walls during the experiments (Table 2), and its location aligns with the location of the axial load application during the tests. Subsequently, a cyclic lateral loading history was applied on the wall models. For specimen TW2, a displacement-controlled analysis was used to apply loading protocol at the top of the wall model that corresponds to the one recorded during the experiment. A force pattern used to conduct the analysis consisted of a single point force applied in the direction of loading at the height of 3810 mm, which corresponded to the location of the actuator connected to the wall specimen (Fig. 4a). A constant displacement increment of 0.25 mm was used throughout the analysis. For the U-shaped wall specimens, the wall top loading history was applied to the models by exerting lateral forces at the locations of the actuators (Fig. 4b) via sequential force-controlled analyses to subject the analytical models to the same multidirectional loading history measured during the experiments. A representative comparison of experimental and analytical loading histories exerted at the wall height of 2950 mm for specimens TUB and TUD are compared in Fig. 5. Each force-controlled loading sequence ($O \rightarrow A$ is one loading sequence, $A \rightarrow O$ is another loading sequence, etc.) was executed in 100 analysis steps. Twisting of the specimens was restrained during the analyses to match the boundary conditions enforced on the walls during the experiments.

A Newton-Raphson solution algorithm (OpenSees command *algorithm Newton*) was employed to conduct the numerical nonlinear analyses for all considered specimens. Convergence was reached if the norm

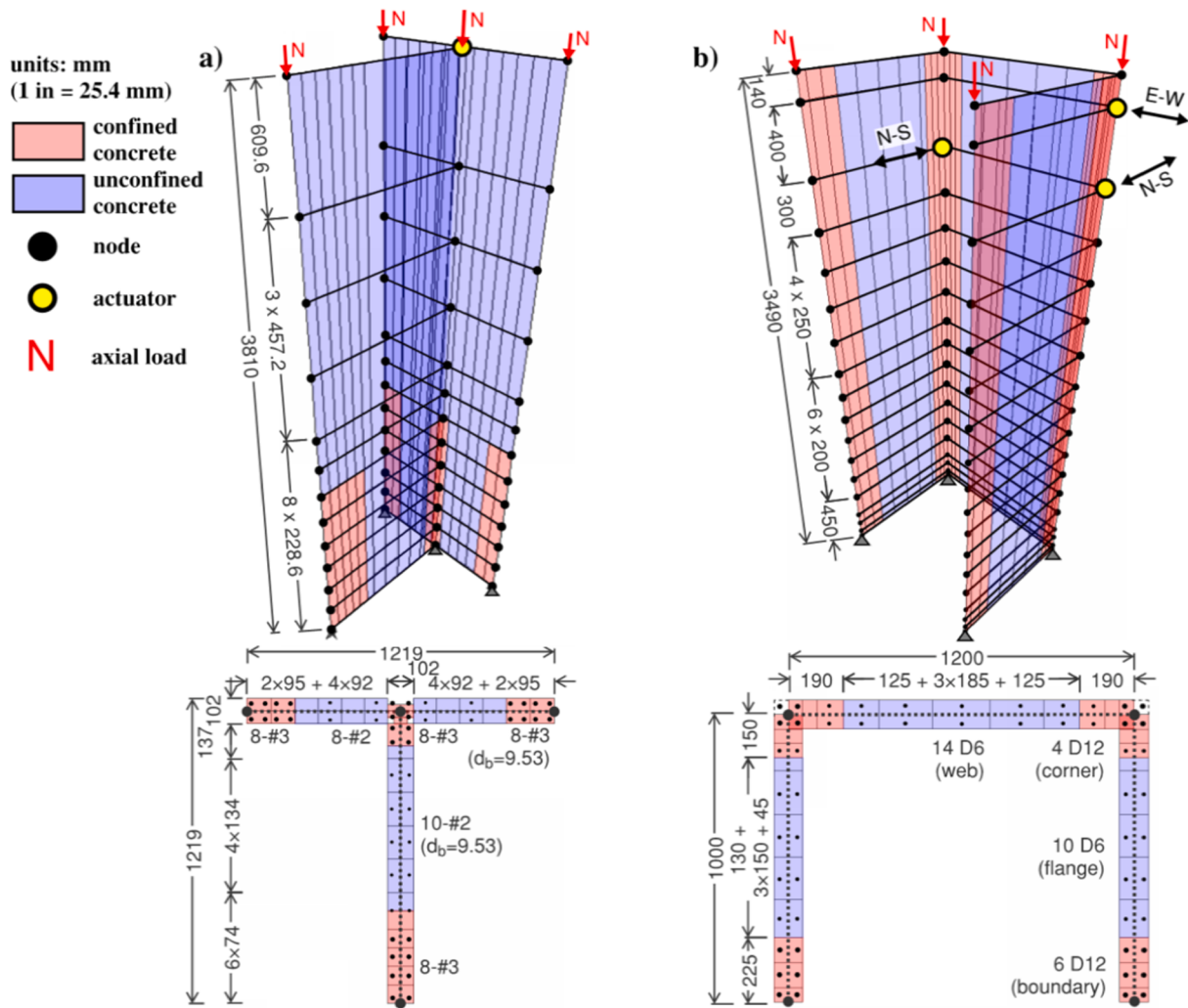


Fig. 4. Model discretization for specimens: a) TW2, b) TUB (TUA, TUC, TUD similar).

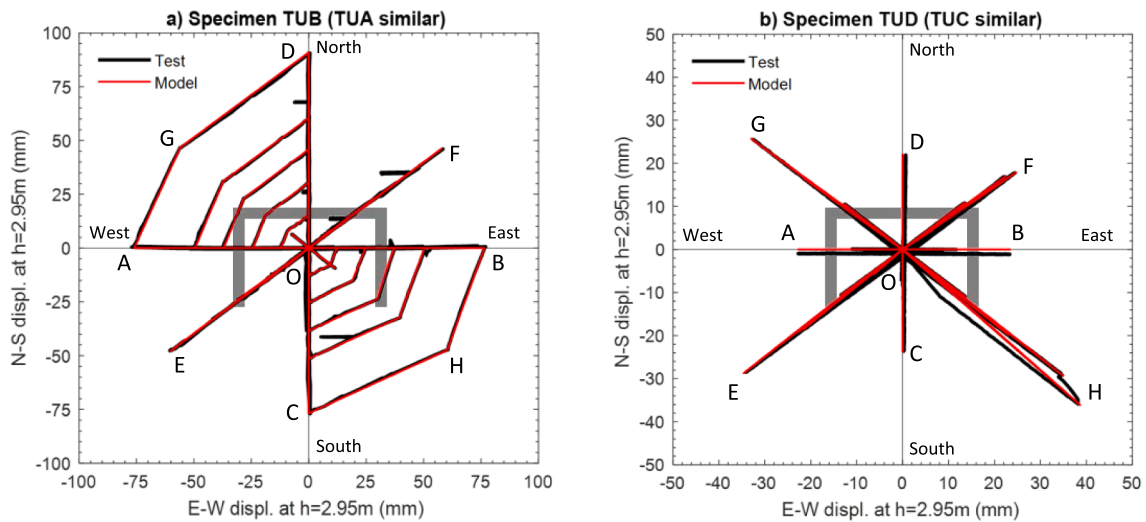


Fig. 5. Comparison of experimental and analytical top displacement histories applied at $h = 2.95$ m of U-shaped wall specimens.

of the displacement increment, defined using the OpenSees command *test NormDispIncr*, was less than the adopted error tolerance of 10^{-5} . Convergence was first pursued using the updated (current) tangent stiffness matrix corresponding to the beginning of each analysis step,

and if convergence is not achieved at a given load step, the iterations were performed using the initial stiffness matrix of the model, by using the OpenSees command *algorithm Newton-initial*.

7. Analytical sensitivity studies

This section presents sensitivity of analysis results generated using the proposed MVLEM-3D with respect to user-defined modeling parameters, including geometric discretization of the model (mesh size) and the effective shear stiffness. The sensitivity studies consider the influence of variation in the selected modeling parameters on the predicted load-deformation behavior and the vertical strain predictions obtained from cyclic analysis of wall specimen TW2.

7.1. Mesh sensitivity

Sensitivity of analysis results to variations in mesh size of the wall model are presented in this section. Three different model mesh configurations are considered for the sensitivity study: 1) mesh with one element per web/flange over the height of one story (coarse mesh, Fig. 6a), 2) mesh with one element per web/flange and multiple elements over one story height (intermediate mesh, Fig. 6b; see Section 6.2 and Fig. 4a for details), and 3) mesh that has the same vertical discretization as the intermediate mesh, but includes several model elements along the length of the web/flange (dense mesh, Fig. 6c). As discussed in Section 6.2, the intermediate mesh is adopted as the baseline mesh, whereas the coarse mesh replicates the geometric discretization used commonly by practicing engineers (i.e., one element over the story height) and the dense mesh is considered for investigating the model behavior when multiple elements are used along the horizontal length of a single wall pier.

As shown in Fig. 6, no significant differences can be observed among the three different mesh configurations in terms of the overall strength, stiffness and cyclic characteristics of the wall load-deformation behavior. It can be observed by careful inspection that the predicted wall stiffness and capacity is slightly larger for the coarse mesh (Fig. 6a), whereas intermediate mesh (Fig. 6b) and dense mesh (Fig. 6c) configurations yield nearly identical load-deformation responses.

Fig. 7 presents analytically-obtained vertical (flexural) strains corresponding to peak drift levels of 0.5% and 1.5% in both directions recorded from the bottommost web elements of the analytical models of specimen TW2 for the three mesh configurations considered. Results demonstrate that strains in the wall web obtained from the models with intermediate and dense mesh, which had the same bottommost elements heights, are almost identical, suggesting that using multiple MVLEM-3D elements along the wall web maintains the plane-sections assumption and does not influence the predicted strain profile. Analytical results also suggest that the strains calculated by the model with coarse mesh, which had element heights within the bottom two stories that were four times that of the intermediate and dense meshes, are about 60% and 80% of the strains predicted by the intermediate and dense meshes, in the positive (Fig. 7a) and negative (Fig. 7b) loading directions,

respectively, because element vertical deformations are averaged over a much larger element height.

7.2. Sensitivity to shear stiffness

Effective shear stiffness values equal to 0.50G, 0.25G, and 0.025G were considered for the sensitivity study, based on the discussion presented in Section 6.1.3. The predicted load-deformation responses of specimen TW2 for the three selected values of elastic shear stiffness are shown in Fig. 8. As expected, the initial and unloading/reloading stiffness values are sensitive to the choice of the effective shear stiffness, where major differences can be observed between results obtained using effective shear stiffness values of 0.25G and 0.025G. The results also indicate minor sensitivity of the predicted wall lateral load capacity to shear stiffness, which is more pronounced for loading cycles in the negative loading direction (web in compression), where the model with an effective shear stiffness of 0.50G predicts about 5% larger peak lateral load compared to the model with a shear stiffness of 0.025G.

Fig. 9 shows analytically-obtained vertical (flexural) strains at peak drift levels of 0.5% and 1.5% in the positive and negative loading directions, obtained from the bottommost model elements representing the web of specimen TW2, for the three shear stiffness values considered. Results presented reveal that use of a lower shear stiffness results in smaller vertical strains in the model results, because it leads to larger lateral deformations associated with shear behavior, which reduces flexural deformation contributions to lateral displacement, therefore decreasing the vertical strain predictions. Specifically, reducing the shear stiffness from 0.5G to 0.25G leads to only minor reduction of vertical strains by less than 5%. However, reducing the shear stiffness from 0.25G to 0.025G results in vertical strains that are approximately 10% and 40% smaller for the positive (Fig. 9a, flange in compression) and negative (Fig. 9b, web in compression) loading directions, respectively.

Sensitivity studies with respect to the effective shear stiffness were conducted for all five wall specimens considered in this study. Analytical results were compared against test data considering the wall initial and unloading stiffness, deformation capacity, and vertical strains. It was observed that using a shear stiffness of 0.025G in the analytical model consistently resulted in closest agreement with experimental measurements.

8. Comparison of experimental and analytical results

8.1. Convergence rates and analysis times

Analyses of the five test specimens considered were conducted on a computer running on a 64-bit Windows 10 operating system that was equipped with an Intel Core i7-9820HQ CPU@2.70 GHz processor and

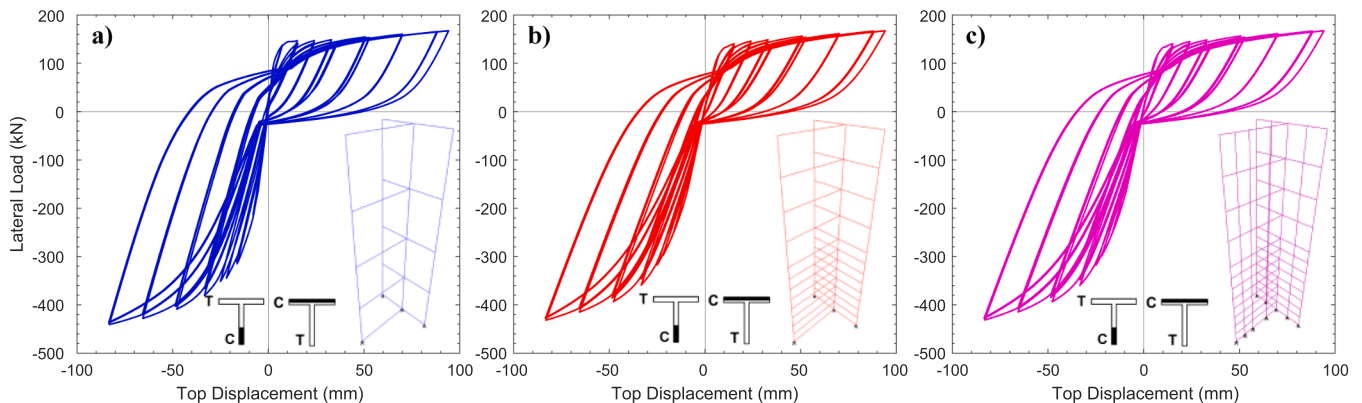


Fig. 6. Mesh sensitivity of predicted load-deformation response for specimen TW2: a) coarse mesh, b) intermediate mesh, c) dense mesh.

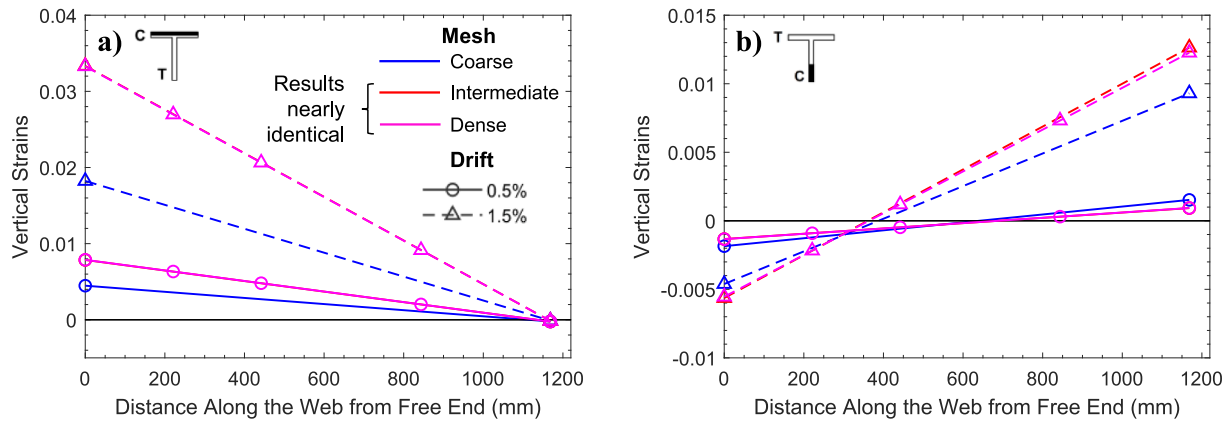


Fig. 7. Mesh sensitivity of predicted vertical strain profiles obtained from the bottommost model elements for specimen TW2: a) peaks of positive load cycles, b) peaks of negative loading cycles.

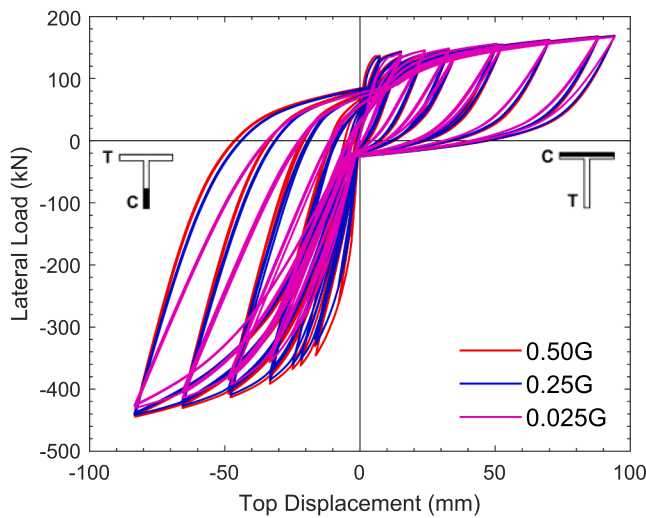


Fig. 8. Sensitivity of predicted load-deformation response to adopted shear stiffness for specimen TW2.

32.0 GB of RAM memory. Table 3 summarizes the information collected during analysis of the specimens including analysis type (displacement-controlled vs. force-controlled), total number of analysis steps, convergence success rate with the Newton algorithm that uses a current tangent stiffness, and runtime. As it can be observed from the table, convergence success rate with current tangent stiffness matrix is equal or

very close to 100% for all models, where the lowest success rate is 99.81% for specimen TUB. This information demonstrates the excellent numerical stability of the MVLEM-3D element proposed. In addition, analysis times ranged between 46 s and 1 min 45 s, which reveals the exceptional numerical efficiency of the proposed model. In general, the longer analysis times are associated with either the larger total number of analysis steps, or the larger number of analysis steps where convergence was pursued using the Newton method with initial tangent stiffness, because the convergence rate of that algorithm is inherently much slower compared to the Newton method that uses current (updated) tangent stiffness.

8.2. Load displacement responses

8.2.1. Specimen TW2

The first specimen analyzed is the T-shaped wall specimen TW2 subjected to unidirectional cyclic loading parallel to the web. The measured lateral load versus wall top displacement response of specimen TW2 is compared in Fig. 10 with the one predicted using the MVLEM-3D model. Comparison of results reveals that the proposed analytical model underestimates the lateral load capacity of the wall specimen in the positive loading direction (wall flange in compression) by approximately 10%. The reason for this inconsistency lies in the inability of the model to capture the variation (gradient) of vertical strains (and the resulting axial stress distribution) developing across the thickness of the MVLEM-3D elements used in the flange of the wall (Fig. 4a), which prevents correct predictions of the neutral axis depth when the wall flange is in compression (in the positive loading

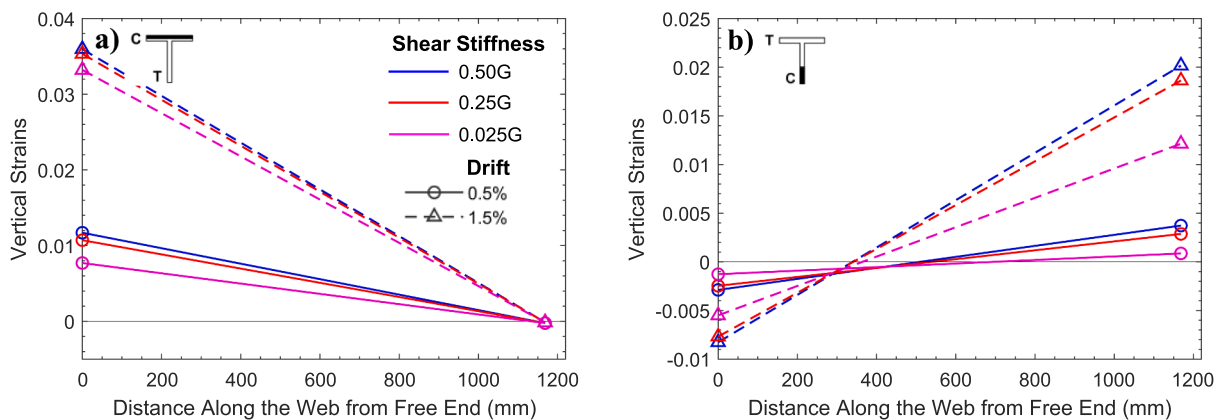


Fig. 9. Sensitivity of predicted vertical strain profiles to adopted shear stiffness obtained from the bottommost model element for specimen TW2: a) peaks of positive loading cycles, b) peaks of negative loading cycles.

Table 3
Analysis convergence rates and runtimes.

Specimen	Analysis Type	Displ./Force Increment per Analysis Sequence	No. of Analysis Sequences	Total No. of Analysis Steps	No. of Steps when Initial Tangent is Used	Convergence Rate with Current Tangent	Runtime (min:sec)
TW2	Displ.-controlled	0.25 mm	46	14,916	0	100.00%	01:19
TUA	Force-controlled	0.01	123	12,300	20	99.84%	01:32
TUB	Force-controlled	0.01	124	12,400	2	99.98%	01:45
TUC	Force-controlled	0.01	147	14,700	2	99.99%	1:08
TUD	Force-controlled	0.01	116	11,600	0	100.00%	0:46

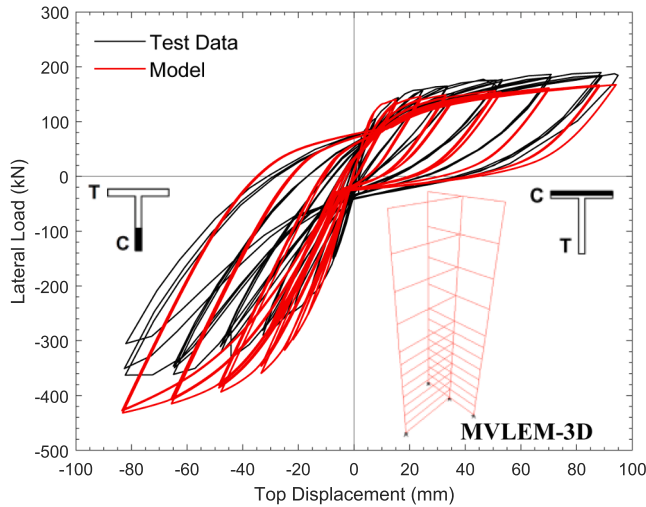


Fig. 10. Comparison of measured and simulated load-displacement responses for specimen TW2.

direction). Results presented in Fig. 10 further show that the model overestimates the wall capacity in the negative loading direction (when the wall flange is in tension) by approximately 15%. For the negative loading cycles, the experimentally-measured tensile strains follow a nonlinear distribution along the width of the flange (Fig. 15a), while the MVLEM-3D produces a uniform distribution of strains along the flange due to the plane-sections assumption. Therefore, tensile strains predicted by the model are overestimated in the wall flange (Fig. 15a), which leads to the overestimation of the wall lateral load capacity for loading cycles when the flange is in tension. The analysis results predict, with reasonable accuracy, the initial and cyclic reloading/unloading wall stiffness as well as the pinching properties of the measured load-

displacement response. Model does not predict the strength loss (in the negative loading direction) observed during testing of specimen TW2 due to buckling of longitudinal reinforcing bars and out-of-plane instability of the wall web boundary region, since these failure mechanisms are not implemented in the current formulation of the model.

8.2.2. Specimens TUA and TUB

Figs. 11 and 12 present comparisons between experimental and analytical lateral load versus wall top displacement relationships for loading cycles in N-S, E-W, and diagonal loading directions for specimens TUA and TUB, respectively. Comparison of results indicate that the MVLEM-3D can predict the measured lateral load capacity of the two specimens in the E-W loading direction, with good accuracy (Figs. 11a and 12a). For loading cycles in the N-S loading direction (Figs. 11b and 12b), the lateral load capacities of specimens TUA and TUB are underestimated by 5% when the wall web is in compression (Pos. D) and overestimated by 5–10% when the web is in tension (Pos. C), similarly to that observed for specimen TW2 (Fig. 10). Figs. 11c and 12c further reveal that the analytical model overestimates the capacities of specimens TUA and TUB by approximately 50% in the diagonal loading direction at position E and by only 5%–10% while pushing the model towards position F. This overestimation of wall strength in diagonal loading directions is associated with the plane-sections hypothesis implemented in MVLEM-3D formulation, which overestimates the length of the flange in tension that contributes to wall strength (i.e., the shear lag effect). This issue is investigated in more detail in Section 8.3.2 by analyzing distributions of vertical axial strains at along the wall base. Furthermore, wall initial stiffness is predicted accurately in the analysis results for both specimens TUA and TUB in all loading directions, while the unloading stiffness is generally modestly overestimated in the analysis results. The pinching and the overall shape of the cyclic load-displacement responses in all loading directions are predicted reasonably well. Finally, the significant strength degradation observed during the experiments due to fracture of longitudinal reinforcement (TUA) and

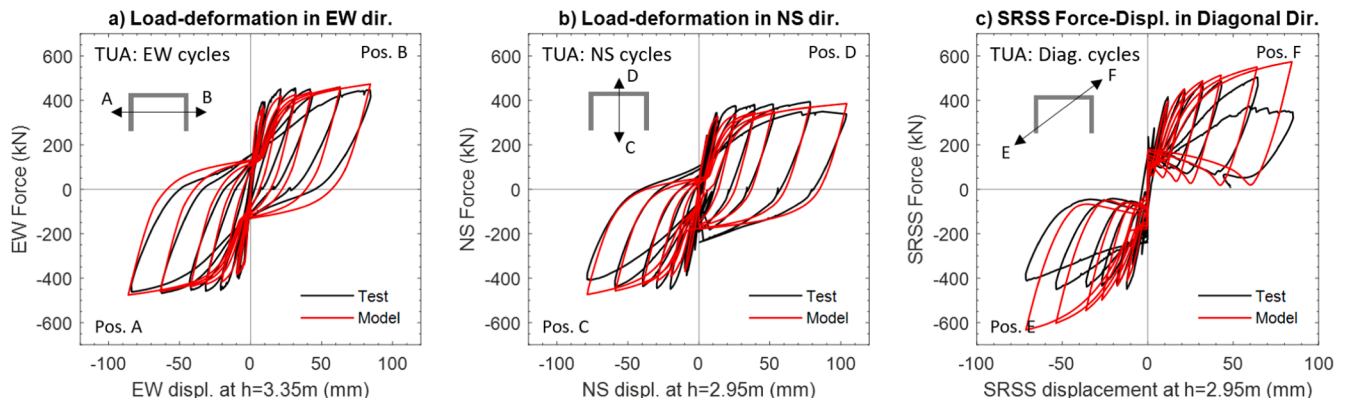


Fig. 11. Comparison of measured and simulated load-displacement responses for specimen TUA.

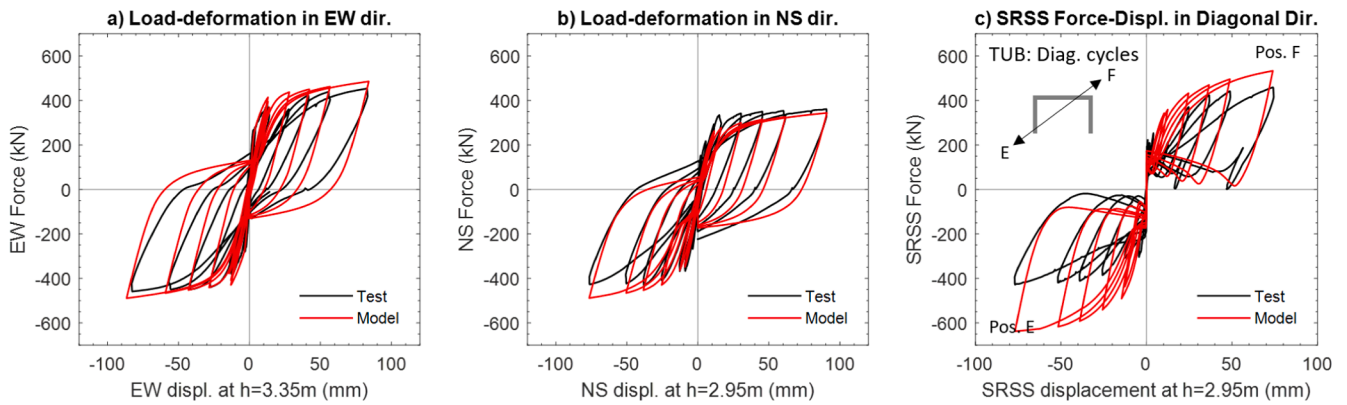


Fig. 12. Comparison of measured and simulated load-displacement responses for specimen TUB.

crushing along the diagonal compression strut in the web (TUB) is not captured in the analysis results, since these strength degradation mechanisms are not implemented in the model formulation.

8.2.3. Specimens TUC and TUD

Experimentally-measured and analytically-predicted load-displacement behavior of specimens TUC and TUD are compared in Figs. 13 and 14, respectively, for loading cycles in the east-west, north-south, and the two diagonal directions between positions E-F and G-H. Note that

specimens TUC and TUD were loaded predominantly along diagonal directions. In this case, the overall accuracy of the MVLEM-3D is similar to that observed for specimens TUA and TUB. The model captures, with good accuracy, wall stiffness and strength in the loading directions parallel to the cross-sectional principal axes (Figs. 13a-b and 14a-b), where analytical results overestimate the wall capacity in diagonal loading directions by 20%-50% (Figs. 13c-d and 14c-d). The largest overestimation of wall strength under diagonal loading is observed at loading position E, because of the same reasons discussed in the previous

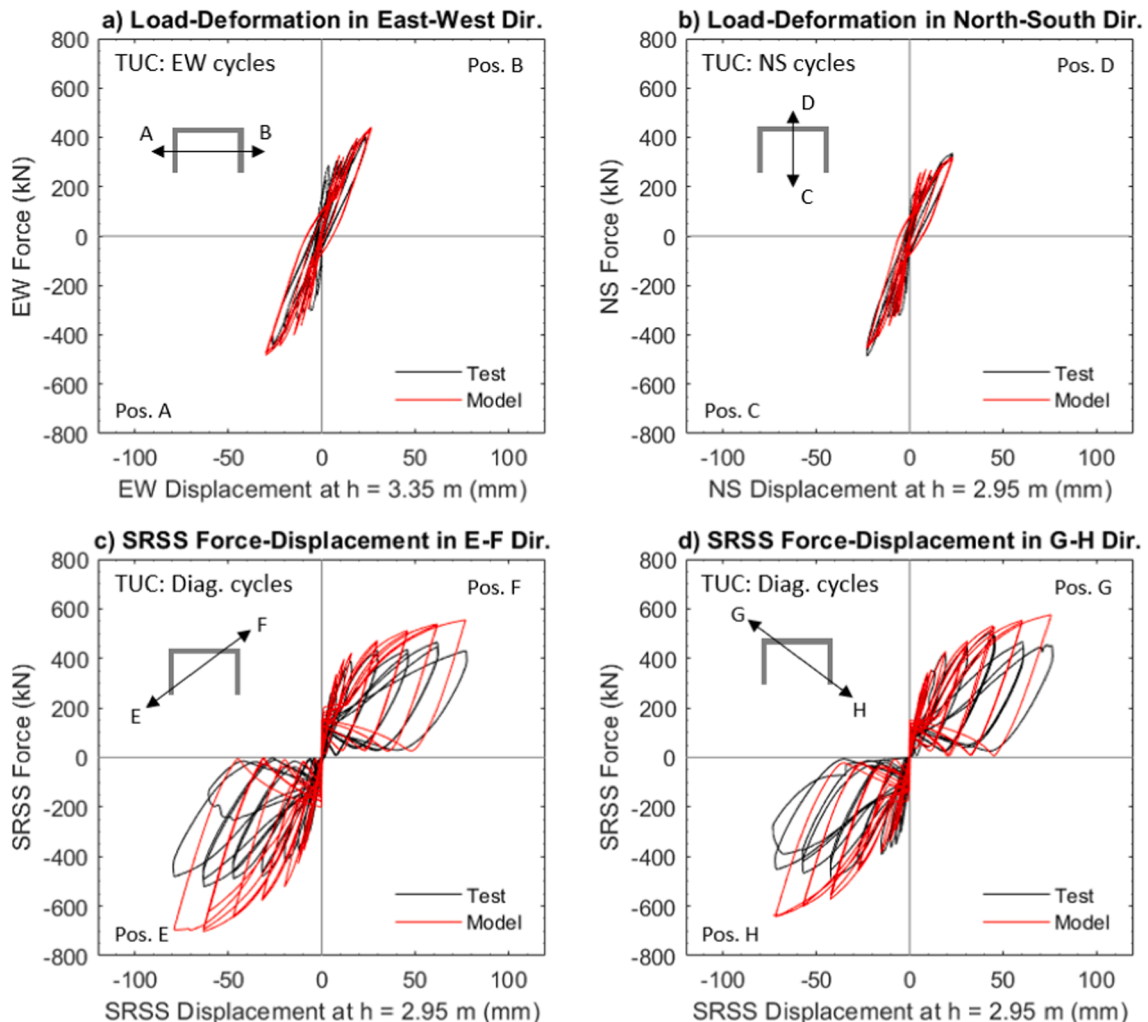


Fig. 13. Comparison of measured and simulated load-displacement responses for specimen TUC.

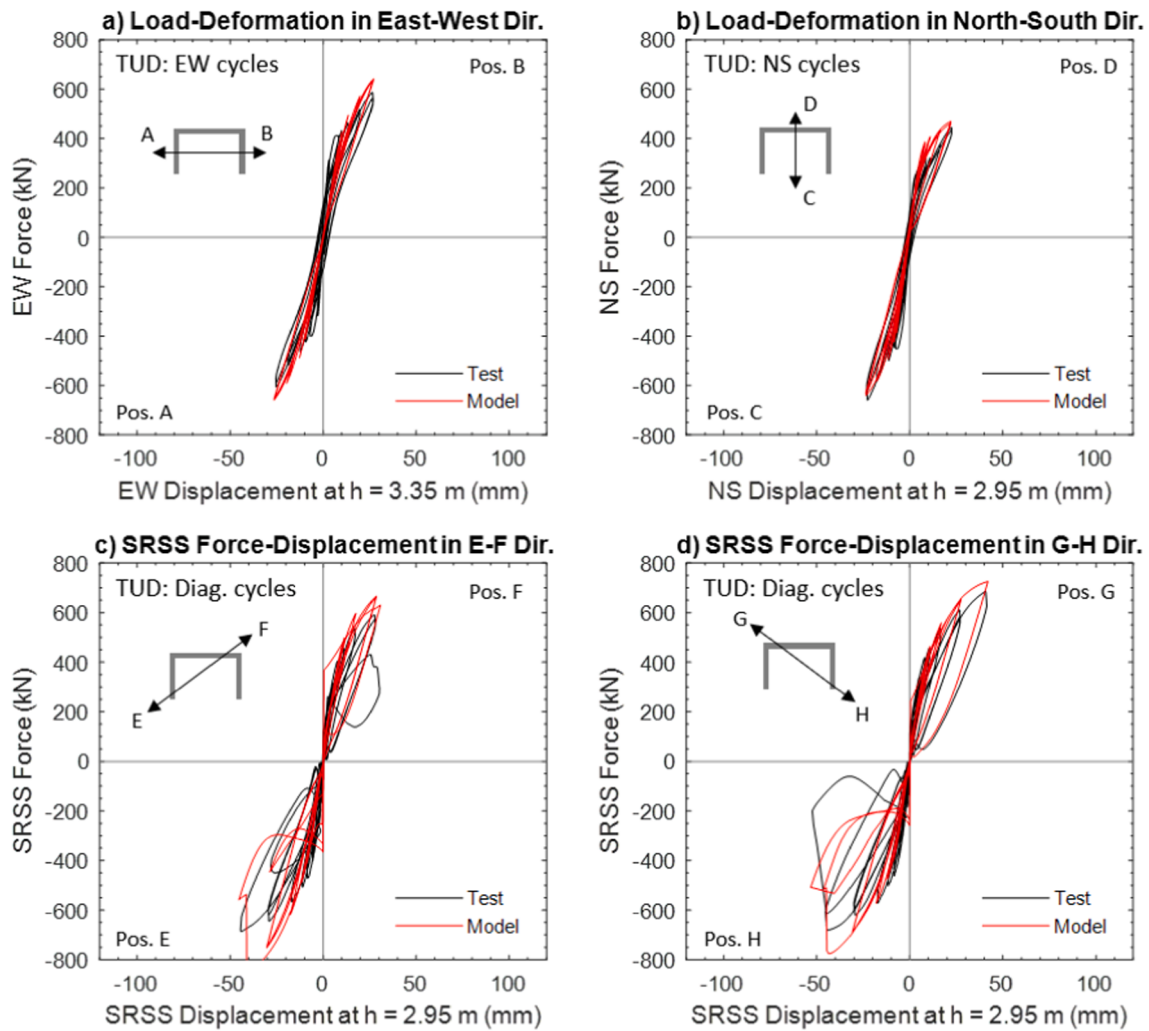


Fig. 14. Comparison of measured and simulated load-displacement responses for specimen TUD.

section for specimens TUA and TUB. The overall stiffness characteristics and cyclic attributes of the load-displacement behavior are well-predicted by the MVLEM-3D model. Strength degradation due to out-of-plane instability (specimen TUC) and concrete crushing along the diagonal compression strut in the web (specimen TUD) is not captured in the analysis results, since the present model formulation does not incorporate these experimentally-observed failure mechanisms.

8.3. Vertical strains

The ability of the proposed analytical model to predict wall axial strains in vertical direction is assessed in this section using experimentally-measured strain profiles at various locations for specimens TW2 and TUC.

8.3.1. Specimen TW2

Fig. 15 compares the measured and predicted strain distributions along the flange (Fig. 15a) and the web (Fig. 15b-c) of specimen TW2 at peaks of positive and negative loading cycles to drift ratios of 0.5%, 1.0% and 1.5%. Strains were measured during the experiment using vertical LVDTs at the base of the wall at four locations along the web and five locations along the outside face of the flange over a gauge length of 229 mm, while the analysis results were obtained from the bottommost model elements of the same height. Since MVLEM-3D elements representing the web terminate at the mid-thickness of the flange (Fig. 4a), analytically-obtained strains are extrapolated to the outer face of the

flange in order to allow a more consistent comparison with experimental data.

The measured tensile strains follow a nonlinear distribution along the width of the flange (Fig. 15a), while the analytically-predicted strain distribution is uniform due to plane-sections assumption of the MVLEM-3D element. Also because of this assumption, tensile strains at the web/flange intersection are overestimated (Fig. 15c). Results presented in Fig. 15a also show that the compressive strains measured in the flange are approximately two times larger than the compressive strains predicted by the MVLEM-3D model, also due to plane sections not remaining plane. Fig. 15b further demonstrates that the analytical tensile strains at the free end of the web are 40% (at drift of 1.5%) to 100% (at drift of 0.5%) larger than the measured strains. Furthermore, Fig. 15c reveals that change in the cross-section geometry at the intersection of the web and the flange leads to sudden decrease of the compressive strains, while the compressive strains tend to increase at the web free end, which again cannot be captured by the plane-sections assumption implemented in the proposed analytical model. Magnitudes of the compressive strains obtained from the analysis are only about 50% of the experimental compressive strains at the web free end.

8.3.2. Specimen TUC

Fig. 16 compares vertical strain profiles obtained from the experimental data and the MVLEM-3D analysis for specimen TUC, along the height of the south-west and south-east flange boundaries of the wall, for diagonal loading cycles at drift levels of 0.4% (approximately yield drift)

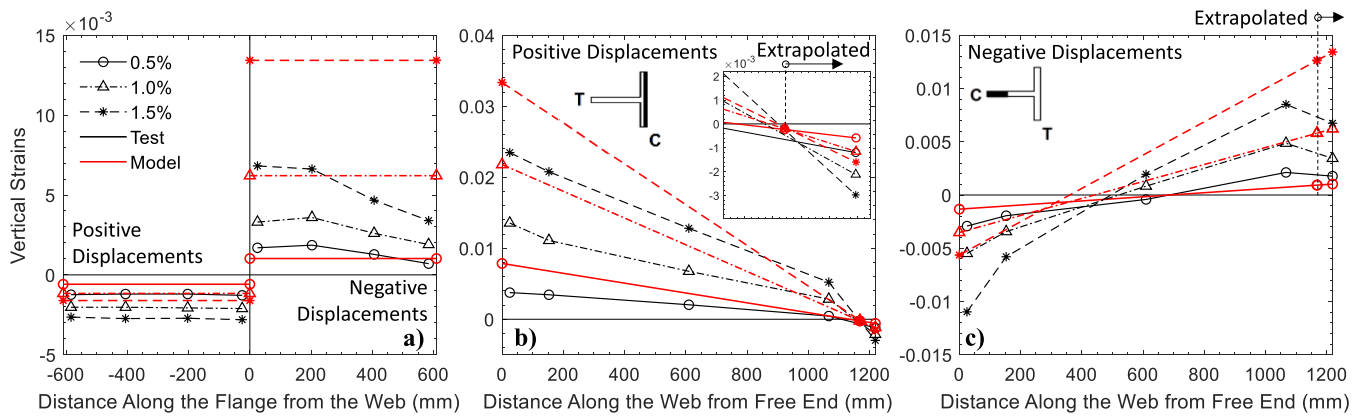


Fig. 15. Comparison of measured and simulated distributions of axial vertical strains at the base of specimen TW2 along: a) flange, b) web (under positive displacements), and c) web (under negative displacements).

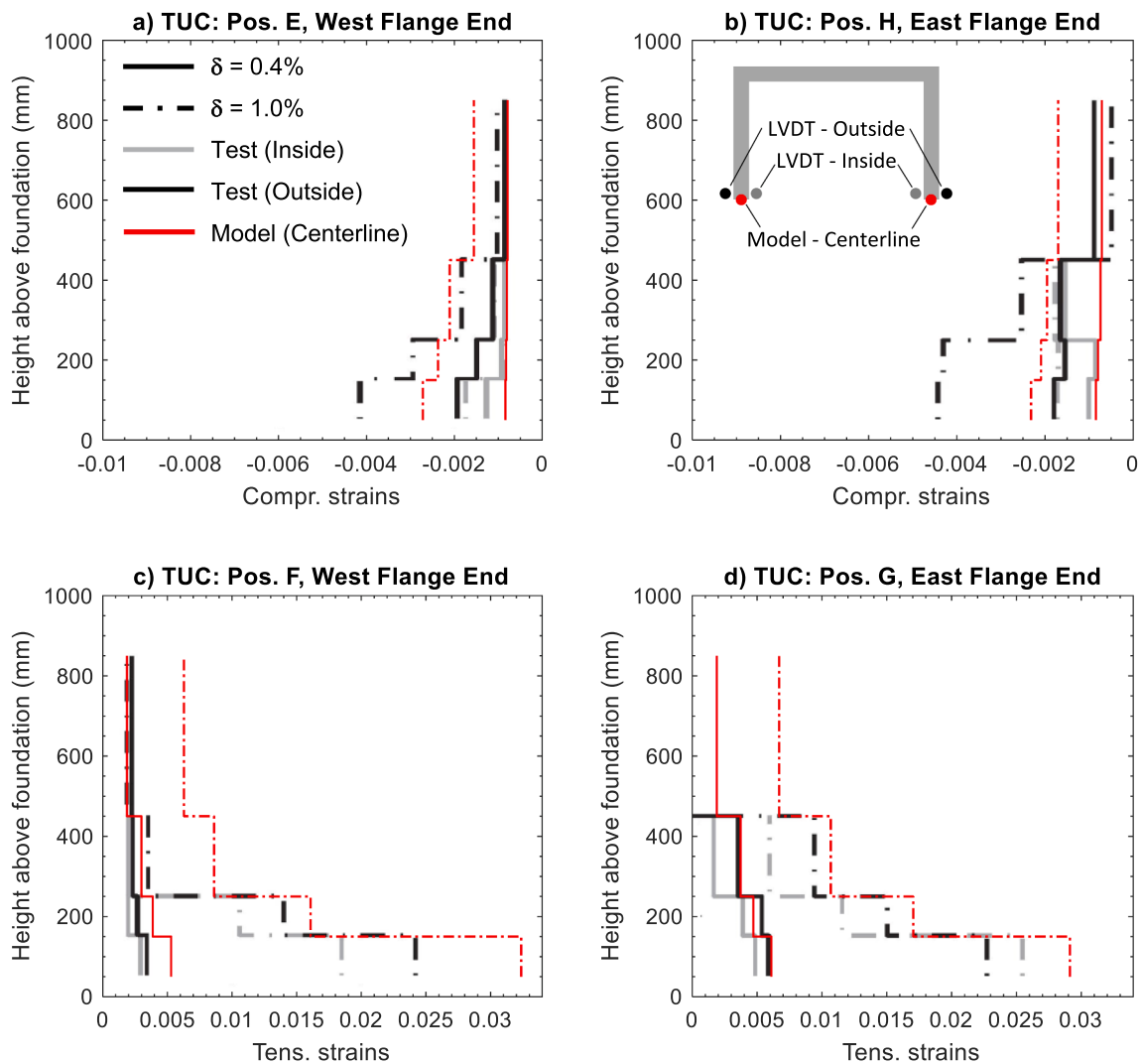


Fig. 16. Comparison of measured and simulated vertical strain profiles for specimen TUC.

and 1.0%. The reported experimental vertical strains were measured using LVDT chains mounted on the inner and outer sides of the flange boundaries, while the analytically-obtained vertical strains are obtained at the mid-surface of the wall element. Experimental data presented in Fig. 16 reveals a considerable difference between the experimentally-

measured vertical strains on the inner and the outer faces of the wall flanges during the diagonal loading cycles, indicating a significant flexural strain gradient across the flange thickness. For example, at loading positions E and H (flange in compression), the compressive strains measured over a height of 50–150 mm from the wall base on the

outer face of the wall flange end are up to approximately 2.5 times those measured on the inner face (Fig. 16a-b). Furthermore, at loading positions F and G, when the flange ends are in tension, the tensile strains measured on the outside face are also generally larger, by up to approximately 1.4 times those on the inner face (Fig. 16c-d).

Comparisons of experimental and analytical vertical strain profiles presented in Fig. 16 show that the new MVLEM-3D model reasonably predicts the distribution of tensile and compression strains along the flange height for wall TUC at considered displacement demands. The analysis results well-replicate the height over which the strains are localized at the wall base (i.e., plastic hinge length), as well as the overall trend of strain profiles over the height of the flange. Fig. 16a-b reveal that the compressive strains are slightly underestimated at the drift level of 0.4%, while at the drift level of 1.0%, the predicted compressive strains are generally in between the strains measured on the inner and outer faces of the flanges. However, Fig. 16c-d show that model generally overestimates the vertical tensile strains on the wall, by a margin of 10%-60% of the measured values.

Fig. 17 plots measured and predicted strain distributions along the base of specimen TUC at loading positions C (Fig. 17a), E (Fig. 17b), and G (Fig. 17c); results are presented for a wall drift ratio of 0.6% at position C and for drift levels of 0.4% and 1.0% at positions E and G. The experimental strains are obtained from optical measurements on the outer perimeter of the wall over a height of 0–75 mm above the foundation, while the analytical strains are obtained at the wall mid-thickness, using the bottommost model elements that had the same height of 75 mm.

Fig. 17a reveals that the analytical model accurately captures the experimental strain distributions measured along the wall base, at loading position C (loading parallel to wall flanges) and at a drift ratio of 0.6%, where both the tensile and compressive strains as well as the neutral axis depth are predicted within $\pm 10\%$ of the experimental measurements. For cycles in the diagonal loading directions (Fig. 17b-c), the shear lag effect (plane sections not remaining plane) becomes pronounced in the experimental results, leading to nonlinear strain distributions measured along the base of the specimen TUC, which cannot be captured by the MVLEM-3D that is based on the plane-sections assumption. Despite this shortcoming of the model, the analytical results are reasonably accurate across the majority of the wall base at both diagonal loading positions E (Fig. 17b) and G (Fig. 17c) at the drift ratio of 0.4% (yield drift), since the overall behavior is still in the linear elastic range at this drift level. However, at a drift of 1.0%, where considerable reinforcement yielding is observed on the test specimen, the plane-sections assumption of the MVLEM-3D element fails to capture the concentration (or reduction) of tensile and compressive strains at the corners of the wall cross-section. At loading position E (Fig. 17b), MVLEM-3D results overestimate the tensile strains by approximately 40% at the north-east corner of the cross-section and inaccurately

predict the strain distribution along the west flange, which was entirely in compression during the experiment. Similarly, at loading position G (Fig. 17c), the model overestimates the tensile strains along the east flange by 50–60%, overestimates the tensile strains at the north-east corner by 400%, and significantly underestimates the compressive strains concentrated at the north-west boundary. This inability of the model to predict nonlinear distributions of vertical strains along the base of the wall during diagonal loading cycles leads to overestimation of the wall lateral load capacity by the MVLEM-3D model, as presented in Section 8.2.2 and Section 8.2.3. Note that use of more sophisticated models for RC walls (e.g., finite-element models) overcomes this issue [21].

9. Summary and conclusions

A new three-dimensional, four-node macroscopic model for simulation of nonlinear behavior of RC walls is proposed, experimentally-validated, and implemented in OpenSees. The proposed analytical model is an extension of the two-dimensional, two-node Multiple-Vertical-Line-Element Model (MVLEM), which has been proven to be efficient and accurate computational tool for nonlinear analysis of planar flexure-dominated RC walls. Extensions of the original MVLEM include: 1) implementation of a geometric transformation that converts the two-node model element formulation into the four-node formulation, and 2) implementation of linear elastic out-of-plane element behavior. The proposed model, namely MVLEM-3D, has been validated against detailed experimental data recorded from tests on five non-planar RC wall specimens including one T-shaped wall tested under unidirectional cyclic loading and four U-shaped walls tested under complex multidirectional cyclic loading regimes. In addition, sensitivity studies were conducted with respect to model geometric discretization and to the effective shear stiffness selected. The model is publicly available in OpenSees, which provides the engineering community worldwide with a tool for improved three-dimensional, system-level, nonlinear analysis of RC wall buildings, as well as with a flexible platform for further model improvements including implementation of experimentally-observed failure mechanisms.

Analytical studies conducted using the proposed MVLEM-3D model have demonstrated the following:

- 1) The model is computationally very efficient (in terms of run time) and stable (in terms of convergence stability), where analysis durations for the investigated wall specimens ranged between 46 s and 1 min 45 s, and convergence was achieved using the Newton method with current tangent stiffness during more than 99% of all load steps, for all specimens considered.
- 2) Model discretization in horizontal direction does not noticeably influence the predicted global and local wall responses, while element

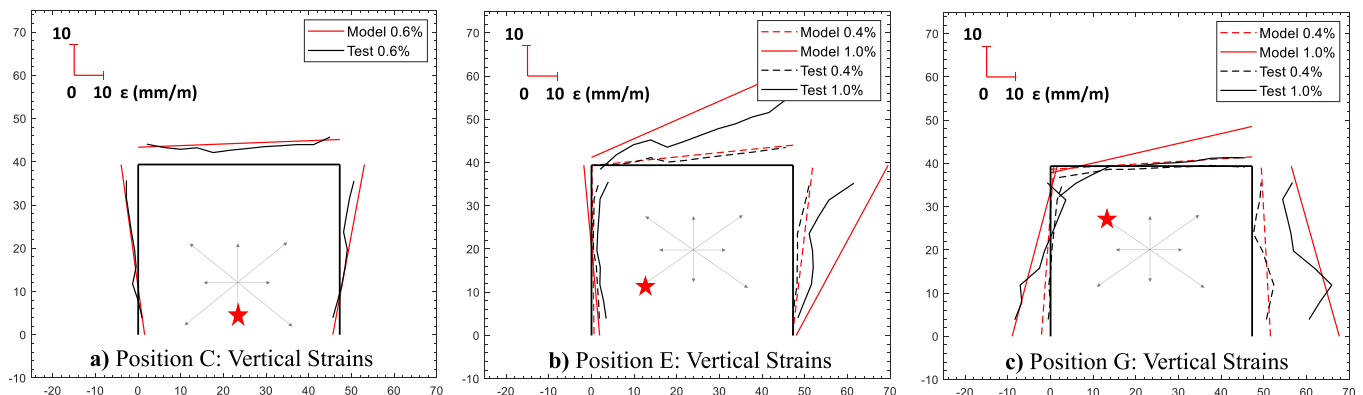


Fig. 17. Distributions of vertical strains at the base of specimen TUC at: a) Position C, b) Position E, and c) Position G. Positive (compressive) strains are shown at the outer face of the wall; negative (tensile) strains are plotted at the inner face of the wall.

size variations in the vertical direction have minor effect on the calculated load-displacement behavior and considerable impact on the flexural strain estimations.

- 3) Considerable sensitivity of analysis results was observed with respect to the choice of the effective shear stiffness, where the major impact is observed for the predicted local (vertical strain) wall responses. An effective shear stiffness equal to 2.5% of the uncracked wall shear stiffness was found to be reasonable for analysis, using the MVLEM-3D model under unidirectional and multidirectional loading, of all of the non-planar walls investigated in this study.
- 4) The MVLEM-3D model predicts, with reasonable accuracy, the overall load-displacement behavior of the analyzed non-rectangular walls subjected to unidirectional and multidirectional loading, including their stiffness, strength, and pinching characteristics. The predicted lateral load capacities of the walls are within 10% of the experimentally-measured values for loading cycles applied in the directions of principal axes of the wall cross-sections. In diagonal loading directions, the wall capacities are overestimated by between 20% and 50%, due to inability of the plane-sections-remain-plane assumption incorporated in the MVLEM-3D element to accurately predict the extent of the flexural tension and compression regions on the non-rectangular wall cross-section as well as the location of the neutral plane. The model generally does not capture the significant degradation in lateral load carrying capacity that was experienced by the wall specimens, because various failure mechanisms observed during the tests (buckling of longitudinal reinforcement and lateral instability of the compression wall boundary) are not incorporated in the present model formulation.
- 5) The proposed analytical model can predict, with good accuracy, the measured vertical strains at low lateral displacement demands (prior to significant nonlinear behavior) and when the loading is applied parallel to the principal cross-sectional axes of a non-planar wall. However, analysis results corresponding to post-yield loading cycles and in diagonal loading directions tend to considerably underestimate compressive strains and overestimate tensile strains developing on the wall. Under these loading conditions, experimentally-measured strain distributions along the wall cross-sections become highly nonlinear due to the shear lag effect, which cannot be captured by the model that is based on the plane-sections-remain-plane assumption.

CRedit authorship contribution statement

Kristijan Kolozvari: Conceptualization, Methodology, Software, Validation, Formal analysis, Data curation, Resources, Writing - original draft, Writing - review & editing, Visualization, Supervision, Funding acquisition, Project administration. **Kamiar Kalbasi:** Methodology, Software, Writing - original draft. **Kutay Orakcal:** Methodology, Writing - original draft, Writing - review & editing. **John Wallace:** Methodology, Writing - original draft, Writing - review & editing.

Declaration of Competing Interest

The authors declare that they have no known competing financial interests or personal relationships that could have appeared to influence the work reported in this paper.

Acknowledgments

This work was supported by the National Science Foundation, Award No. CMMI-1563577. Any opinions, findings, and conclusions expressed herein are those of the authors and do not necessarily reflect those of the sponsors.

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