



Hybrid approach for simulating shear–flexure interaction in RC walls with nonlinear truss and fiber models

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Abstract

An alternative approach to the nonlinear truss model (NLT) is proposed to simulate the seismic behavior of reinforced concrete (RC) walls with various aspect ratios (ratio between the height and the length of the wall, h_w/l_w). The alternative consists of a hybrid model comprising an NLT panel connected in series with a forced-based fiber-beam–column model through a rigid elastic beam element (HyNLT-F model). The HyNLT-F approach saves computational time as compared to the default NLT model while keeping the capabilities of modeling inelastic shear response and shear–flexure interaction under static cyclic or dynamic loading. These capabilities are first validated for the default NLT model using the experimental test data of two RC-wall panels subjected to reversed cyclic loading, one whose response is flexure-dominated and another with a shear–predominant behavior. The numerically computed lateral force–displacement relationships, lateral displacement contributions, shear–flexure interaction, and vertical strain distribution at different levels of drift demand show a good agreement with the experimentally recorded responses. The hybrid alternative is then validated using three RC-wall panel experiments with high ($h_w/l_w = 3.1$), moderate ($h_w/l_w = 2.3$) and low aspect ratio ($h_w/l_w = 1.5$). The capabilities of the hybridization are evaluated at the global and local level of response, as well as in terms of computational time needed to run the model in comparison with the default NLT model. As an application, a HyNLT-F model is implemented in a multistory non-ductile RC frame-wall structure, to evaluate the impact of the nonlinear shear response at the critical section in the global structural behavior under static and dynamic loading. The results show that the HyNLT-F can model features of the wall response such as inelastic shear and shear–flexure interaction at the critical section, which of interest for certain structural typologies.

Keywords Hybrid · Nonlinear truss model · Fiber model · Shear–flexure interaction · Inelastic shear · Reinforced concrete wall · Numerical simulation

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1 Introduction

The nonlinear truss model (NLT model) (Park and Eom 2007; Panagiotou et al. 2012) was developed on the basis of the strut-and-tie model first developed in Europe (Ritter 1899; Morsch 1922) for simulating beams failing in shear and later to simulate disturbed regions of deep reinforced concrete (RC) members (Schlaich and Weischede 1982; Schlaich and Schäfer 1991). The NLT model was further developed and programmed in OpenSees (McKenna et al. 2000) by Lu and Panagiotou (2014). It was extensively tested by Lu et al. (2014) to simulate the pseudo-static and dynamic response of shear walls of slender and squat geometry, with several reinforcement characteristics, and failure modes, demonstrating its capabilities and limitations. The NLT model uses overlapping vertical, horizontal, and diagonal elements to represent the discontinuous strain field of reinforced concrete in its cracked state. Shear–flexure interaction is modeled explicitly through the geometric interaction of vertical, horizontal, and diagonal elements while the compression field of concrete (Vecchio and Collins 1986) is directly accounted for in the latter. It is worth noting that inelastic shear modeling of RC wall is a complicated task (Fischinger et al. 2014) and only a few macro-models available in the literature can reproduce the shear–flexure interaction (Massone et al. 2006; Jiang and Kurama 2010; Fischinger et al. 2012; Kolozvari et al. 2014). Due to the overlapping elements in the NLT model, the model overestimates the axial and flexural stiffness in the elastic (uncracked) state. Compared to the commonly used force-based fiber-beam–column element with distributed plasticity (Fiber model), the NLT model better simulates the response of shear-dominated midrise and squat walls by explicitly modeling shear–flexure interaction. Nevertheless, it has the disadvantage of higher computational cost due to the larger number of nodes and elements required for representing a bidimensional wall panel.

The paper proposes a hybrid NLT-fiber model (HyNLT-F) for reinforced concrete walls that couples the NLT model, which is implemented in critical nonlinear areas to model shear–flexure interaction, with a Fiber model to achieve faster running times, maintaining the accuracy. First, the efficacy of the default NLT model (Lu and Panagiotou 2014) is evaluated to model global and local response of reinforced concrete walls, by comparing the analytically computed results to the experimentally reported results of two isolated wall specimens: one flexure-dominated wall and one shear-dominated squat wall. The experimental data serves as benchmark for the detailed evaluation of the NLT model at the global and local level of response. Two features of the NLT model not well documented in the literature that are tested here are its capability of simulating nonlinear shear–flexure interaction, and nonlinear distributions of normal strain profiles in walls, which is a deviation from the classical Euler–Bernoulli theory which most macro models are programmed upon.

Next, the analytical results of the HyNLT-F are compared against the experimental results of three isolated wall specimens of different aspect ratio. A main variable studied is the heights of the NLT (h_{HyNLT}) component compared to the total length of the specimen (l_w). The HyNLT-F model is evaluated in terms of its capacity to model the global force–displacement response of the wall specimens, and local features such as shear–flexure interaction and nonlinear distributions of normal strain profiles at the critical section. Additionally, the time needed to run the models in comparison with the default NLT and fiber model is also evaluated. Based on the aspect ratio, a minimum height of the NLT component is determined, which allows modeling the response of the walls adequately. Finally, the proposed HyNLT-F is employed in studying the seismic behavior of a

multistory frame-wall building subjected to ground motion excitation with the purpose of evaluating how the variation in the inertial forces along the wall height might trigger different response characteristics.

2 Modeling capabilities of the NLT model

This section focuses on testing two singular features of the default NLT model: (i) its capability of modeling shear–flexure interaction in walls, and (ii) its capability of simulating a non-linear normal-strain distribution at their critical section (e.g. the Euler–Bernoulli assumption does not necessarily apply). Compared to previous studies (Lu and Panagiotou 2014; Lu et al. 2014, 2017; Kolozvari et al. 2018) on the NLT, this paper discusses and compares the contributions of shear distortions to total displacements, nonlinear shear–flexural interaction, and nonlinear normal strain distributions along the length of the critical section. This allows a more detailed comparison between the model and experimental behavior and enhances the understanding of the NLT model performance. Other studies focused on modeling failure mechanism (Lu et al. 2014), which is out of the scope of this paper.

Two benchmark experimental wall panels are used for this purpose: one flexure-dominated wall as specimen RW2 from the Thomsen and Wallace (2004) test set, and one shear-dominated squat wall as specimen RW-A15-P10-S78 (referred to as RW-A15 in this paper) from the Tran and Wallace (2015) experimental set. Specimens RW2 and RW-A15 have aspect ratios, defined as the quotient between the height where the lateral load is applied and the wall length (h_w/l_w), of 3.1 and 1.5, respectively. Both specimens have low axial load ratio of approximately 7% of $A_g f'_c$, where A_g is the gross cross-section of the web, and f'_c is the actual unconfined concrete strength at the day of the test. The elevation view and reinforcement characteristics of specimens RW2 and RW-A15 are shown in Fig. 1. Table 1 summarizes main characteristics of the specimens such as the cross-section aspect ratio (l_w/t_w), as-tested material properties (f'_c and $f_{y,BE}$), reinforcement ratios at the boundary element ($\rho_{v,BE}$) and the web ($\rho_{h,web}/\rho_{v,web}$), shear–span (M/Vl_w) and axial-load ratios ($P/A_g f'_c$). Other features of the test may be found in the respective published experimental reports (Thomsen and Wallace 2004; Tran and Wallace 2015).

2.1 NLT wall discretization

Analytical models for the wall specimens are discretized using vertical, horizontal and diagonal elements as shown in Fig. 2 for specimen RW-A15. The extreme vertical elements at the edges of the walls represent the boundary elements; these elements are continuous nonlinear fiber-section beam–column elements implemented with the Gauss–Lobatto integration scheme (Spacone et al. 1996) and model the resistance to lateral expansion of the wall provided by the confined boundary zones. Other vertical and horizontal elements are modeled as truss elements with a fiber section assigned to account for tributary area of concrete and steel. The fiber sections represent cyclic, uniaxial stress–strain relationships for the different materials in every section. Tension response of concrete was ignored in the horizontal elements to mitigate the over-prediction of un-cracked stiffness because of the diagonal elements double-counting the available area of concrete. Diagonals are special elements whose constitutive material model is coupled with its geometry in the deformed configuration, allowing it to model the compression field of concrete (Vecchio and Collins

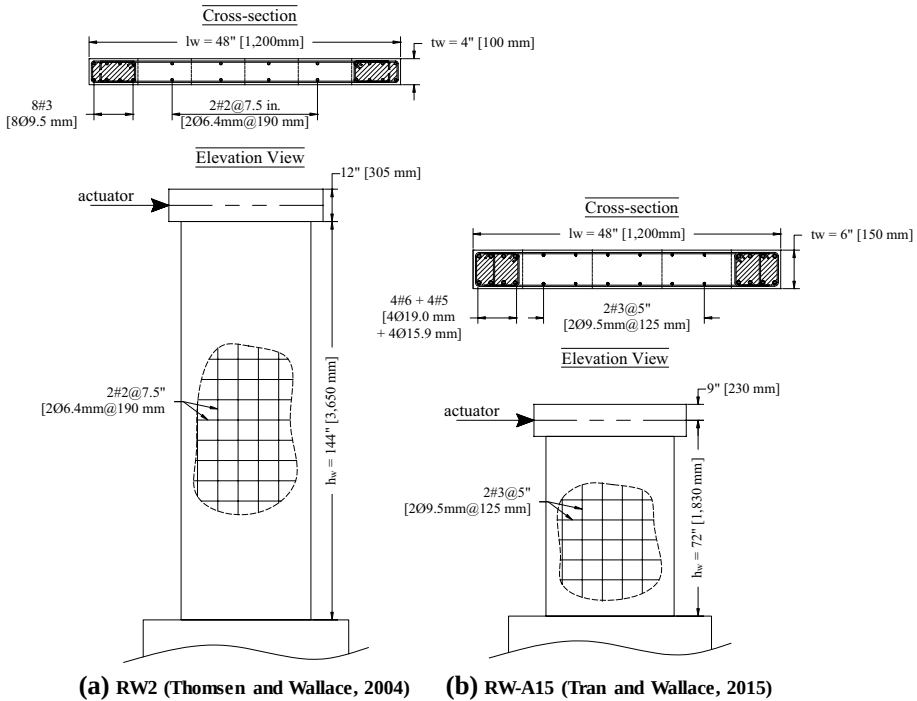


Fig. 1 Geometry and reinforcement detailing of specimens RW2 and RW-A15

Table 1 Experimental data for planar wall test specimens

Specimen ID	l_w/t_w	h/l_w	f'_c (MPa)	$f_{y,BE}$ (MPa)	$\rho_{v,BE}$ (%)	$\rho_{h,web}/\rho_{v,web}$ (%)	$M/(Vl_w)$	$P/(A_g f'_c)$
RW2	12	3.00	43.7	434	2.93	0.33/0.33	3.13	0.07
WSH6 ^a	13	2.02	45.6	576	1.54	0.25/0.46	2.26	0.11
RW-A15	8	1.50	55.8	477	6.06	0.73/0.73	1.50	0.07

^aDazio et al. (2009). This specimen is used to evaluate HyNLT-F model capabilities in Sect. 3

1986). To achieve this, the diagonals are modeled using the nonlinear biaxial truss element as developed by Lu and Panagiotou (2014) in OpenSees. Each panel of concrete diagonals disposed in an X-pattern comprises two 4-node elements. In each element, two of the diagonal nodes connect the main truss, while the others allow connecting a virtual strain-gage element to monitor the strains normal to the diagonal of interest (Fig. 3a). Inclinations of the diagonal is defined by angle θ_d which can be estimated with Eq. (1) [see (Lu and Panagiotou 2014)]:

$$\theta_d = \tan^{-1} \left(\frac{V_{max}}{f_{y,t} \rho_t t_w d} \right) \leq 65^\circ \quad (1)$$

where V_{max} is the maximum resisted lateral force; $f_{y,t}$ and ρ_t are the yield strength and reinforcement ratio of the horizontal steel, respectively; t_w is the thickness of the panel, and d

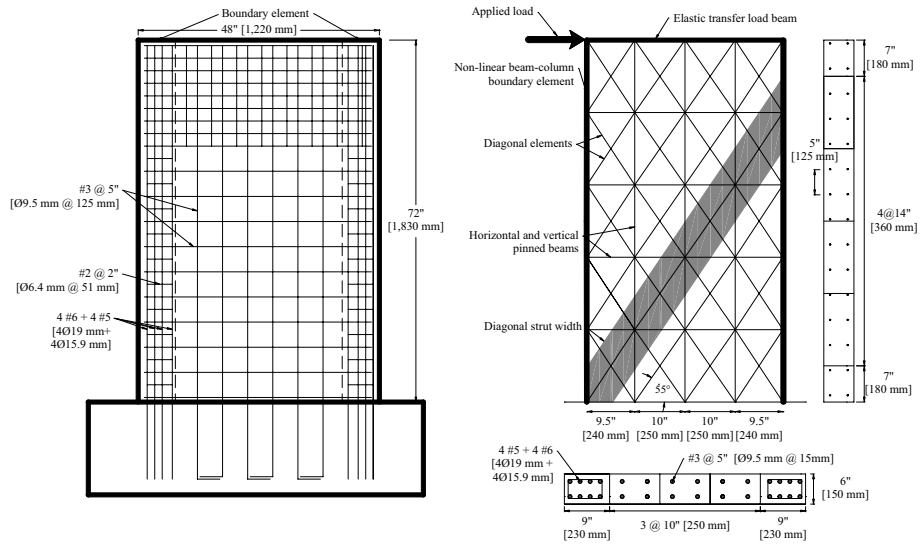


Fig. 2 Discretization example, specimen RW-A15

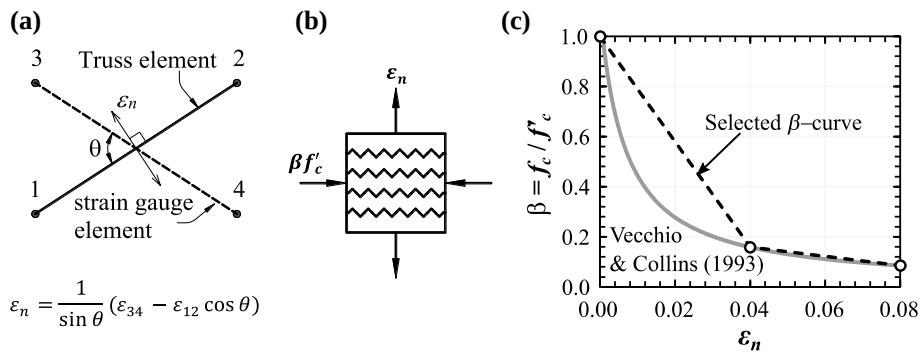


Fig. 3 **a** Four-node truss element formulation for concrete diagonal (struts) modeling; **b** relationship between concrete compressive stress reduction factor, β , and normal strain, ϵ_n [after Lu and Panagiotou (2014)]; **c** calibration of the β -curve parameters compared to the concrete compressive strength reduction factor, as proposed in Lu and Panagiotou (2014)

is the distance between the outer vertical lines in the direction of loading. Recommended lower limit of angle θ_d is 45° for walls with aspect ratio larger than 1.0, except for the case of walls subjected to large tension forces (Lu et al. 2014).

2.2 Calibration of material models

2.2.1 Concrete parameters

Concrete in the vertical and horizontal elements is modeled using the *Concrete02* material in OpenSees using confined or unconfined properties depending on location

within the cross section. The model uses the stress–strain relationship proposed in Kent and Park (1971) and modified by Scott et al. (1982) to include the tensile behavior of concrete. Tensile strength is only used for the vertical elements, while the horizontal elements tensile capacity is set to zero. The hysteretic behavior of concrete under stress reversal is modeled according to Mohd Yassin (1994), including gradual degradation of stiffness under unloading and reloading in compression (Spacone et al. 1996). The linear softening post peak branch of concrete model is regularized according to Eq. (2) to enforce mesh objectivity [see (Coleman and Spacone 2001)]:

$$\varepsilon_{20} = \max \left[\left(\frac{G_f^c}{0.6f'_c L_e} - \frac{0.8f'_c}{E_c} + \varepsilon_{c0} \right), 0.006 \right] \quad (2)$$

where ε_{20} is the post-peak strain of the concrete model at strength $0.2f'_c$; G_f^c is the compression fracture energy (Jansen and Shah 1997), defined as 25 N/mm and 150 N/mm for unconfined and confined concrete, respectively; L_e is the element length; E_c is the concrete modulus; and ε_{c0} is the strain at peak strength for the concrete, set in all cases to 0.2%. A key feature of the NLT model is the diagonal elements, which are modeled using the *Truss2* element in OpenSees. This truss element allows the behavior of the compressive strength of a concrete diagonal to be dependent on the instantaneous tensile strain in the normal direction (Vecchio and Collins 1986, 1993). The compressive stress–strain relationship is parabolic up to f'_c , and is based on the Fujii concrete model [see Hoshikuma et al. (1997)]. The tension stress–strain relationship during loading is linear until it reaches the tensile strength, f_t . After this point, the concrete softens according to Stevens et al. (1991) in an exponential manner. Further discussion of its behavior can be found in Lu and Panagiotou (2014).

The implementation of the concrete model by Lu and Panagiotou (2014) in OpenSees (*ConcretewBeta*) allows the specification of a β -curve (see Fig. 3c), which defines the relationship between concrete compressive strength reduction factor, $\beta = f_c/f'_c$, and the normal strain, ε_n (see definition in see Fig. 3b) at each step. The β -curve is a trilinear representation of the concrete compressive strength reduction factor as proposed in Vecchio and Collins (1993). Parameters to define the β -curve, that is $(\beta_{int}, \varepsilon_{int})$ and $(\beta_{res}, \varepsilon_{res})$, were calibrated to find a single set that simulated adequately the global response across all wall specimens studied herein. The parameters used for all models in this paper are shown in Fig. 3c. These values are defined for gage length $GL = 600$ mm (the size of the panels tested by Vecchio and Collins (1986)), and require further regularization following Eqs. (3) and (4) to accommodate the various diagonal length in the models:

$$\varepsilon_{int,diag} = \frac{600}{L_e} \varepsilon_{int} = \frac{24}{L_e} \quad (3)$$

$$\varepsilon_{res,diag} = \frac{600}{L_e} \varepsilon_{res} = \frac{48}{L_e} \quad (4)$$

where $\varepsilon_{int,diag}$ and $\varepsilon_{res,diag}$ are the regularized parameters of the β -curve; L_e is the diagonal length in mm.

2.2.2 Reinforcing steel parameters

The longitudinal reinforcing steel stress–strain behavior was assumed to be bilinear with isotropic strain hardening (Filippou et al. 1983). The implementation in OpenSees is named *Steel02*, which is a uniaxial Giuffrè–Menegotto–Pinto steel material. Yield strength and initial elastic tangent were taken from the reported material properties in the database. Parameters to control the transition from elastic to plastic branches were $R0=20$, $CR1=0.925$ and $CR2=0.15$ as recommended by Filippou et al. (1983); Lu and Panagiotou (2014). Bar buckling, and/or bar rupture is not accounted for by this model.

2.3 Load–deformation responses

The axial load in the models includes the experimentally applied axial load ratio, as reported in the databases, plus the self-weight of the specimen. The axial load is applied as a distributed load along the top horizontal element at the beginning of the analysis using a Newton Raphson strategy solution with 10 load steps. Then, the vertical load is kept constant. Reverse cyclic horizontal loading was applied using a displacement control strategy, with target displacements obtained from the experimental data. For displacement control analysis, the Modified Newton algorithm was used. Figure 4 presents the experimentally measured and numerically computed total top horizontal drift ratio versus base shear for the two wall specimens considered.

Table 2 compares key simulated structural response parameters with their corresponding experimental value. Variables of interest include the drift and strength at yielding (Δ_y and V_y , respectively), maximum strength V_{max} , and flexural and shear secant-to-yielding stiffness (EI_{eff} and GA_{eff} , respectively). The global response computed with the NLT model is in good agreement with the experimental response, although a larger un-cracked stiffness of the model is observed at early cycles of loading for the reason discussed previously. The displacement at the onset of yielding occurs in the model at around 80% of the measured values; yielding strength is predicted within 5% accuracy for both specimens. Post-yielding strength is under-predicted for RW2; nevertheless, in the inelastic range (after yielding),

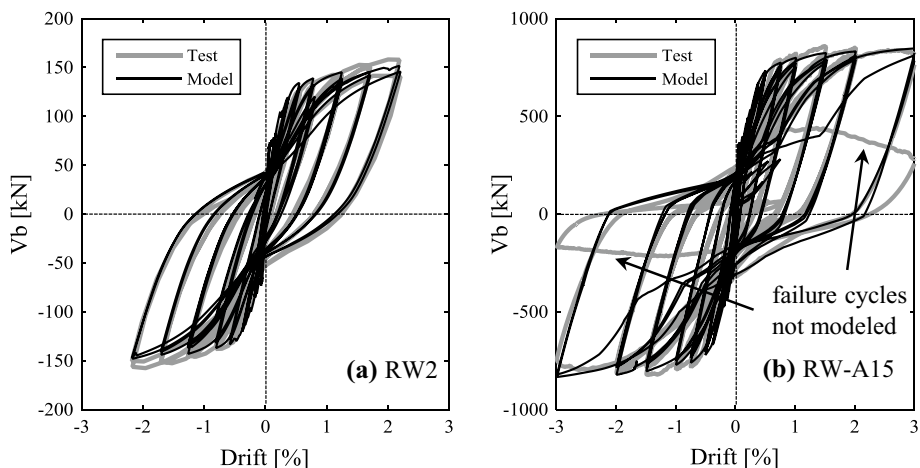


Fig. 4 Load–displacement responses simulated by the NLT model: **a** RW2; **b** RW-A15

Table 2 Comparison of measured and simulated response quantities as modeled by the NLT model

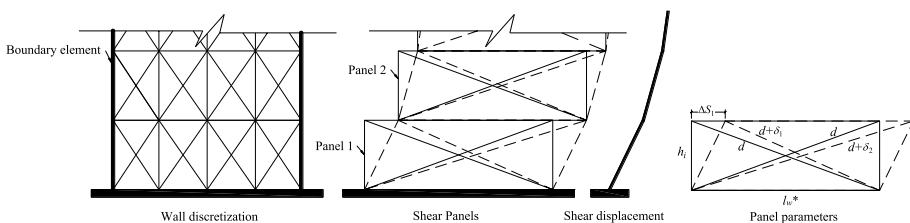
Specimen ID	Δ_{y_sim}/Δ_y	V_{y_sim}/V_y	V_{max_sim}/V_{max}	EI_{eff_sim}/EI_{eff}	GA_{eff_sim}/GA_{eff}
RW2	0.79	0.96	0.92	1.07	0.44
RW-A15	0.78	0.95	1.00	1.25	1.18

discrepancies in terms of strength for a given displacement are smaller than 8%. On the other hand, post-yielding strength is accurately modeled for RW-A15. The under-prediction for the slender specimen is associated, in part, to the shorter arm of the axial forces carried by the extreme elements, which is an inherent issue of the discretization process. For the discretization scheme used, the external vertical portions of the walls are represented by beam–column elements located, in plan, at the centroid of the boundary elements, which effectively produces a model of shorter length.

The global energy dissipation represented by the hysteretic response shape also shows a good agreement for all specimens, except for cycles after the onset of collapse which were not modeled. The flexural-dominated responses of specimen RW2 computed by the NLT model is in excellent agreement with the experimental data (Fig. 4a). The NLT model also simulates well the pinched-flag-shaped response of specimens with shorter shear–span ratios (RW-A15), although pinching is slightly more pronounced in the simulated curves (Fig. 4b). In the NLT model presented herein, the effect of bar buckling, rupture, and the subsequent loss of strength of confined concrete is not modeled and the effect of diagonal crushing is controlled by the strut strength degradation model defined by the β -curve, which was set to degrade at a low rate for all specimens. The authors selected a single set of β parameters for all specimens, as would be done in engineering-or blind-prediction-type of applications where the actual performance of structural components is unknown prior to the simulation. Simulation of failure was out of the scope of this study, and is reported elsewhere (Lu and Panagiotou 2014; Lu et al. 2014). Therefore, the hysteresis loops with progressive loss of strength that are observed after the onset of failure in specimens RW-A15 are not simulated well.

2.4 Shear and flexural deformation contributions

Shear displacement profiles were estimated by integration of panel shear distortions over the height of interest. To estimate shear distortion, a panel in a NLT model refers to the rectangle bounded by the extreme vertical boundary element, and two adjacent sets of horizontal RC elements (see Fig. 5). The shear displacement contribution of each panel is


Fig. 5 Shear displacement estimation model

estimated with Eq. (5) [see (Hiraishi 1984; Massone and Wallace 2004; Beyer et al. 2011)], which is then integrated over the height of the wall:

$$\Delta_s = \frac{1}{4l_w^*} \left[(d + \delta_2)^2 - (d + \delta_1)^2 \right] - (\eta - 0.5)\theta(h_i)h_i; \eta = \frac{\int_0^{h_i} \theta(z)dz}{\theta(h_i)h_i} \quad (5)$$

where l_w^* is the model wall length, d is the length of the diagonal in the undeformed configuration of the panel, δ_1 and δ_2 are the change in length of these diagonals, and $\theta(h_i)$ is the difference of rotations at top and bottom sections of a panel of height h_i . The inclusion of parameter η in Eq. 5 accounts for the difference in diagonal deformations because of the variability of curvature over height that cannot be attributed to shear behavior.

Comparison of flexural and shear response for simulated and experimental data are presented next for specimens RW2 (Fig. 6) and RW-A15 (Fig. 7). For specimen RW2 the flexural and shear displacements were measured over the first story height (gauge length -GL- of 36 in. [914 mm]), whereas for specimen RW-A15 those displacements were measured along the entire wall height (GL = 144 in. [3658 mm]). An overall good agreement between the experimental and simulated response is observed in terms of strength, displacement, and hysteretic shape for both specimens. The contribution to total displacement provided by shear displacements in specimen RW2 ranges from 0 to 10% (considering all levels of displacement demand) for the experimental data, and 0–18% for the analytical data. In contrast, shear displacement contribution for specimen RW-A15 is in the range 24–46% for both the experimental and the simulated data. This confirms that the response of specimen RW2 (slender RC wall) with shear-span ratio of 3.1 is flexure-dominated, while that of RW-A15 (mid-rise RC wall) has larger influence from shear distortions thanks to its smaller shear-span-ratio of 1.5. For specimen RW2, the model slightly overpredicts the initial secant flexural stiffness (up to yielding), while underpredicts the initial shear stiffness (Table 2). Ultimate shear displacement is overpredicted by 36% on average, with respect to the experimental data. The model for specimen RW-A15 also overpredicts the initial secant flexural stiffness, but predicts better the initial shear stiffness. The load–shear displacement

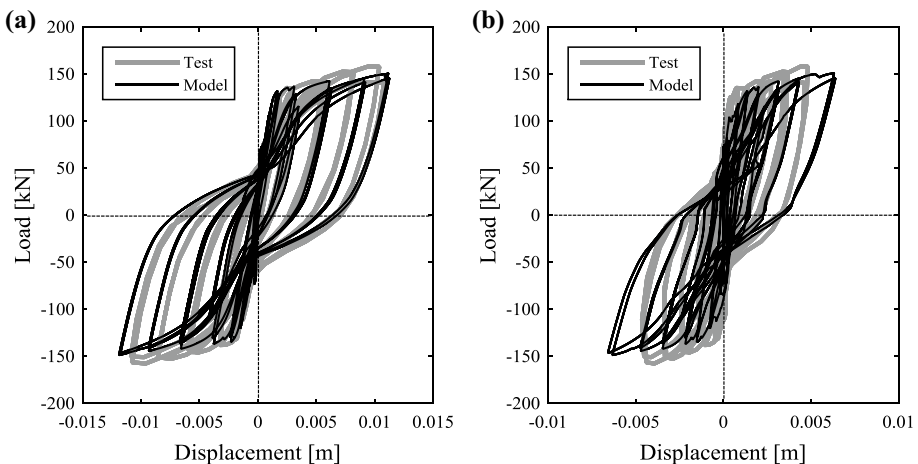


Fig. 6 First-story load-deformation responses for specimen RW2 simulated by the NLT model: **a** flexure, **b** shear

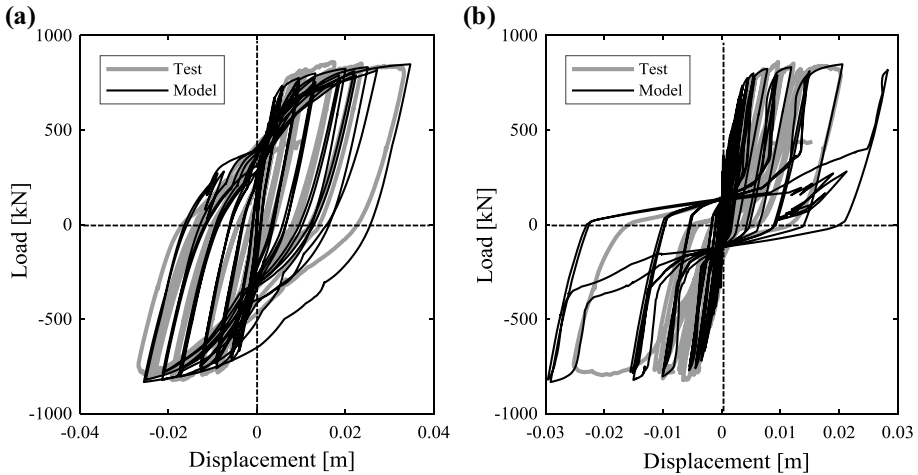


Fig. 7 Load-deformation responses for specimen RW-A15 simulated by the NLT model: **a** flexure, **b** shear

interaction is adequately predicted, though ultimate capacity is over-predicted as strength loss is not modeled.

The experimental and analytical curves in Fig. 8 show the relationship between the flexure- and shear-contributed displacements for both, specimens RW2 and RW-A15. For specimen RW2, the shear–flexure interaction is linear because of its flexure-dominated response. The average shear–flexure slope of the model is approximately 1.23-times that of the experimental data, confirming that the model overpredicts shear displacements. Compared to RW2, RW-A15 has a larger contribution from shear and the elliptical shape of the interaction curves reflects shear–flexural yielding (Fig. 8b). Precise simulation of shear–flexural interaction is a difficult task, especially with models having nonlinear shear spring in series with nonlinear flexural elements (Koložvari et al. 2018). This is especially

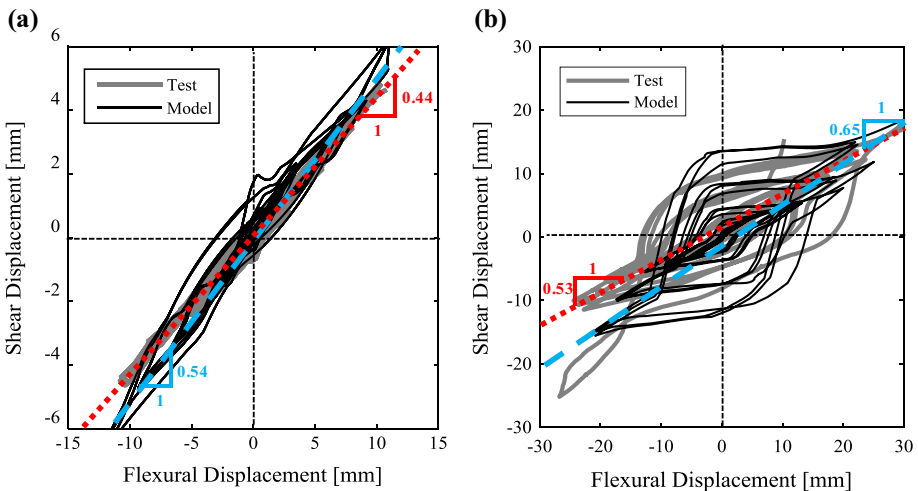


Fig. 8 Shear–flexure interaction curves simulated by the NLT model: **a** RW2 (first story), **b** RW-A15

true when the shear and/ or the flexural springs are modeled with an elastic–perfectly-plastic constitutive relation, or are programmed to soften after their maximum strength is achieved. Nonetheless, the NLT model does simulate coupled shear–flexure yielding with some deviations from the experimental data. In the numerical response, shear or flexural inelastic displacements do not necessarily occur simultaneously, which is clear from the box-shaped loops (i.e. inelastic shear and flexural displacements take turns to develop). The approximately vertical branches of the curves show that inelastic shear displacement is favored, while the flexural displacement rate is low; on the other hand, the flatter portions of the response show that flexural displacements are favored, while the shear displacement tend to remain constant. For specimen RW-A15, the inclination of the shear–flexure curves in the linear region match well, while deviating when inelastic shear–flexural interaction behavior occurs. Before failure, the average shear–flexure slope of the model is also 1.23-times larger than that of experimental data, again confirming the model tends to overpredict shear displacements.

2.5 Nonlinear vertical strain profile at the critical section

Analytical and experimental vertical strain distributions along the wall length are presented in Fig. 9. The experimental strains are measured over a gauge length, measured from the base, of 229 mm and 356 mm for RW2 and RW-A15, respectively, whereas the analytical data was obtained from the strains of the vertical elements of the bottom panels, with similar gage lengths of 208 mm and 366 mm for specimens RW2 and RW-A15, respectively. A notable feature of the NLT model is its capability of simulating the nonlinear vertical strain distribution along the wall length, which is a major difference from most macro-element-type of models that are based on the Euler–Bernoulli bending theory, where plane sections remain plane after the deformation (see Kolozvari et al. (2018) for a comparison of wall macro models capabilities). On the other hand, the NLT model underestimates strains at the extreme fiber in compression. This occurs because the cross-section concrete area is concentrated at discrete positions along the wall length. Therefore, the extreme monitoring location (to the right of the figures) has the boundary element area assigned, which is large enough to prevent compressive strain demand growth with increasing tip drift ratio.

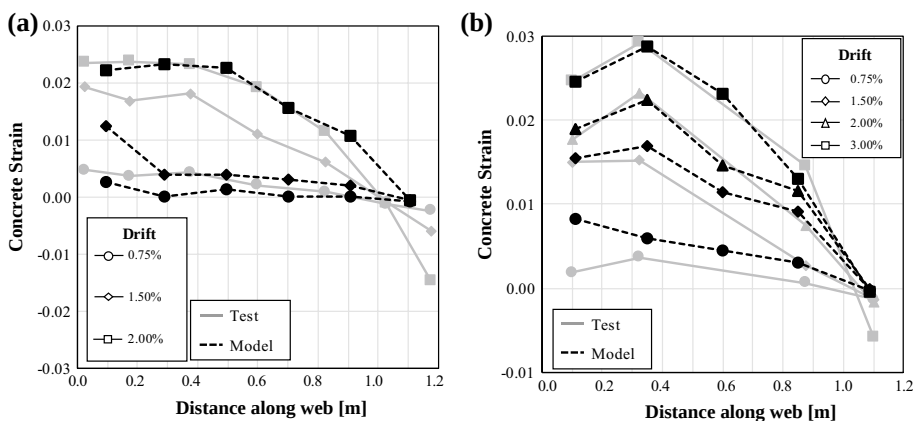


Fig. 9 Distribution of vertical strains along the wall base simulated by the NLT model for Specimen: **a** RW2, **b** RW-A15

Furthermore, part of the force at the extreme in compression is also carried by the closest diagonal strut (see Fig. 14).

For specimen RW2, vertical strains are underestimated for drift levels of 0.75% and 1.50%, by a factor of 2 in average. For levels of drift close to 2.5%, where peak load is reached, strains in tension match well, and are underestimated for compression by 2 to 3 times. For specimen RW-A15, the NLT model overestimates tensile vertical strains by factor 1.5 and 2.0 for drifts between 0.75% and 1.50%. For levels of drift greater or equal than 2.0%, simulated tensile vertical strains match well with experimental data. This level of drift also matches with the instant in which peak load is reached. The relation between the improvement in accuracy and the instant in which peak load is reached can be explained by the angle of the strut diagonals. This angle is defined using Eq. (1), whose main input is the peak load. Compressive strains are also underestimated.

3 Alternative hybrid NLT-fiber-beam-column model

The previous section showed that the NLT model can model the global and local response of slender and squat walls with acceptable accuracy. However, it may be a computationally expensive model when compared to typical fiber-beam-column models, because the greater number of degrees of freedom required to model a bidimensional panel increases the size of the tangent stiffness matrix. The greater number of elements and nodes required to construct a panel also increases the time required to implement the model. In contrast, a forced-based beam-column element only requires two nodes to construct a wall panel that usually requires three to five integration points for proper formulation of its stiffness matrix depending on the quadrature rule employed.

This section proposes a hybrid NLT-Fiber model (HyNLT-F model) as presented in Fig. 10, with the intention of reducing the modeling and computational time when using the NLT model approach. The hybrid model comprises a NLT panel of height $h_{HyNLT} = \alpha l_w < h_w$ at the bottom of the wall, where l_w and h_w are the actual length and height of the wall, respectively, and α is the aspect ratio of the NLT panel, which is connected in series with a fiber-beam-column element for the remaining height of the wall. In the case implemented herein, a force-based fiber-element with 3 Gauss-Lobatto integration points was considered, but other integration schemes may be used as well. The fiber-section element accounts for the same material properties assigned to the vertical elements of the NLT panel. The interface between the NLT panel and the fiber-section element is modeled with a rigid elastic beam. The alternative proposed here is mainly conceived for modeling moderate-aspect-ratio and slender walls, where a predominantly flexure-dominated response is expected, but where a large shear demand may be also expected. Nonetheless, squat walls are also well simulated.

Three specimens previously tested by other authors are modeled using the HyNLT-F model. The specimens are RW2, WSH6 [from Dazio et al. (2009)] and RW-A15. The WSH6 specimen was selected to evaluate the capabilities of the HyNLT-F model to simulate the response of walls with moderate aspect ratio (e.g. $h_w/l_w = 2.3$ for this specimen). Values of α ranging from 1/4 to 1 were considered to determine the minimum height of the NLT panel that lead to an adequate response simulation, while reducing the computational cost. Figure 11 presents the response of the HyNLT-F model versus the experimental data, also contrasting it to that obtained previously with the default NLT model (see top portion of Fig. 11), and with a forced-based fiber-beam-column model (see bottom portion

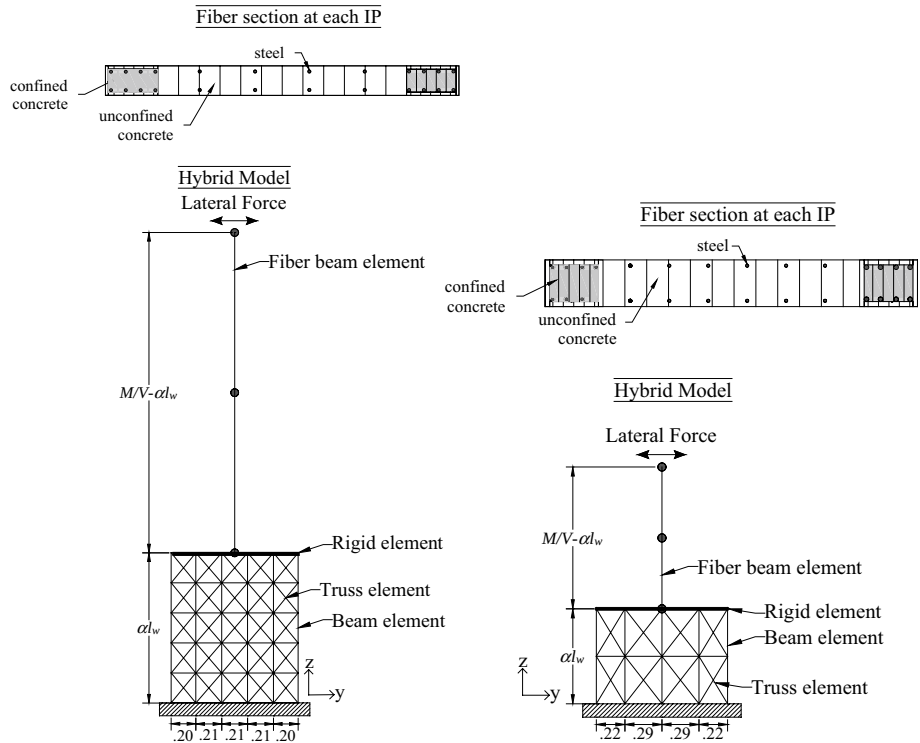


Fig. 10 Hybrid NLT-Fiber model discretization for RW2 and RW-A15

of Fig. 11). For the specimen RW2, the HyNLT-F model global response is in good agreement with the experimental observations for $\alpha \geq 1/4$, although a larger yield strength and un-cracked stiffness is observed for decreasing values of α . This feature is inherited from the NLT model, as previously discussed. The HyNLT-F model estimates better post-yielding strength in comparison with the default NLT model because the response of this specimen is flexure-dominated, and the top fiber-beam-column element is a better approach to model the distribution of moment and curvature in the section. When compared to the fiber-beam-column model, the HyNLT-F model simulates better the pinching of the hysteretic curves.

In the case of specimen WSH6, the HyNLT-F approach models the global response in reasonable agreement with the experimental observations for $\alpha \geq 1/2$. In the positive direction, the HyNLT-F model simulates adequately the elastic behavior of the specimen, estimating accurately the yield strength and displacement. Post-yield strength and pinching in the positive direction are also adequately estimated. In the negative direction, the model underestimates post-yield strength. Although the HyNLT-F better models the strength of the specimen in both directions, the directional strength bias is inherited from the default NLT model, which degrades the strength of the compressive struts for the negative direction, when they are put into tension during the positive cycles. Other models capable of simulating shear-flexure interaction also exhibit this feature for this specimen [see (Kolozvari et al. 2018)]. The fiber-beam-column model does not simulate the post-yield strength capacity nor the hysteretic behavior well.

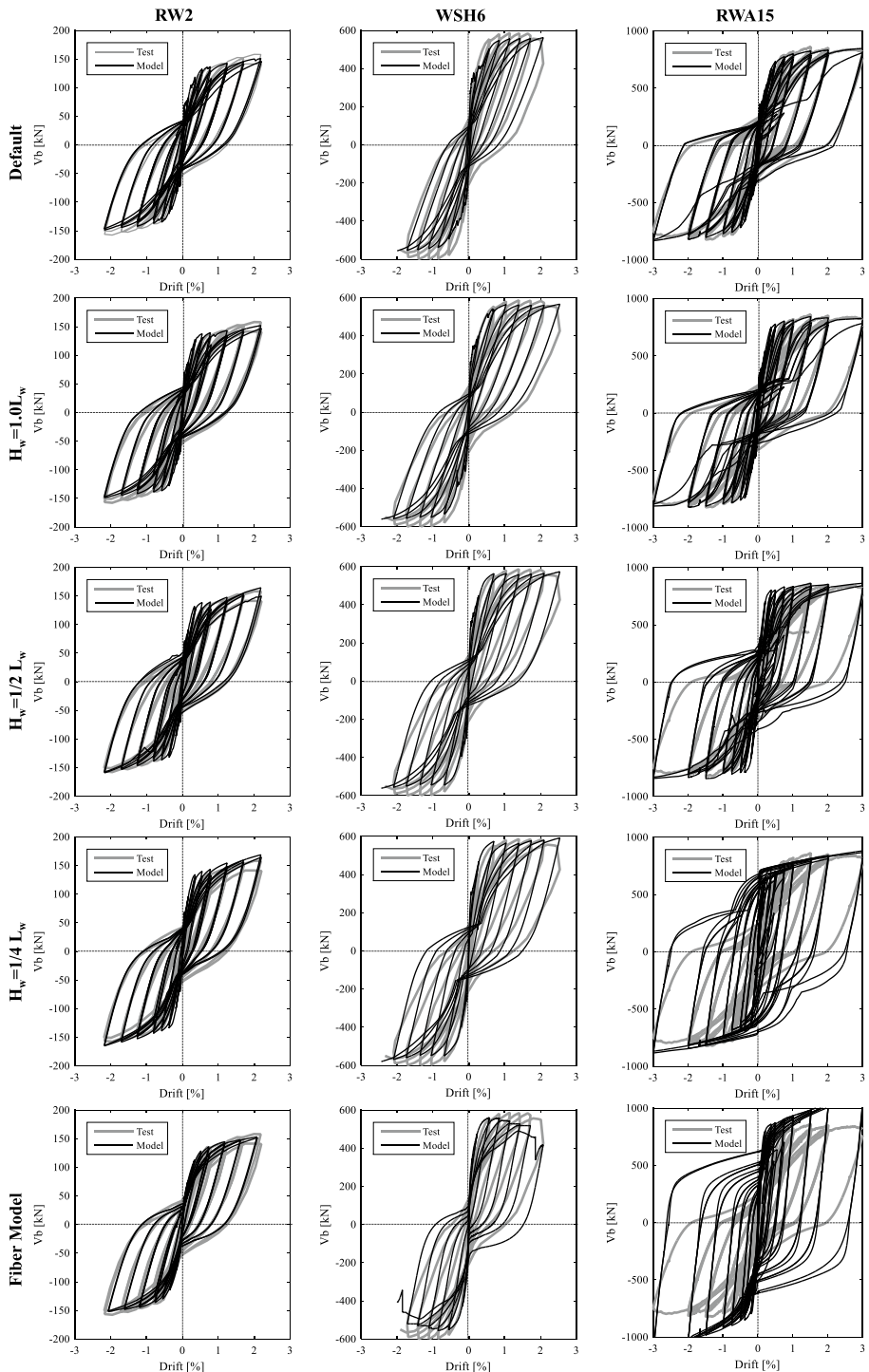


Fig. 11 Load-displacement responses for specimens RW2, WSH6 and RW-A15 using the default NLT, hybrid NLT-Fiber and forced-based fiber-beam-column model

For the specimen with lower shear-span ratio, RW-A15, the HyNLT-F model global response agrees with the experimental results for $\alpha \geq 3/4$, that is, for a panel height at least equal to 75% the wall length (although the simulation shown in Fig. 11 is for $\alpha \geq 1$ due to formatting restrictions). Post-yield strength and hysteretic response is adequately predicted. The value of α needed to simulate adequately the response of the specimen is 1/2 the shear span-ratio of the specimen, being the largest among the three models due to its shear-dominated behavior. Nevertheless, it is worth noting that the hybrid approach is a viable alternative to model walls whose displacements have large contributions from shear distortions. As expected for a shear-dominated specimen, degradation of simulation quality is apparent for decreasing values of α .

As noted above, the minimum acceptable α -values for specimens RW2, WSH6 and RW-A15 are 0.25, 0.50 and 0.75, respectively, hence they can be related to the wall aspect ratio using Eq. (6). Similarly to the results presented in Table 2 for the default NLT model, Table 3 quantifies the predictive capabilities of the HyNLT-F model for $\alpha=1$, in terms of global response parameters such as Δ_y and V_y , V_{max} , EI_{eff} and GA_{eff} . It is noted that results in Table 3 only vary marginally with respect to those reported in Table 2, hence confirming the capabilities of the HyNLT-F model to simulate the global response with the same level of accuracy as the default NLT model. Results indicate that overestimation of secant stiffness (both shear and flexural) increases in the HyNLT-F, resulting in an underestimation of yield displacement Δ_y . However, yield strength and capacity are better estimated, and match almost exactly the values obtained in laboratory tests.

$$\alpha \geq \frac{6}{5} - \frac{1}{3} \frac{h}{l_w} \quad (6)$$

3.1 Shear–flexure interaction and nonlinear vertical strain profile

The hybrid model does simulate inelastic shear and the shear–flexure interaction, although with a deviation from the experimental data given the reduced height over which shear distortions are accommodated. Comparison of flexural and shear response for simulated and experimental data are presented for specimens RW2 and RW-A15 in Fig. 12. For both specimens, the simulated flexural response slightly varies from that obtained with the default NLT model and is still in good agreement with the experimental response in terms of strength, displacement, and hysteretic shape (see Fig. 12a, b). On the other hand, shear displacement is overpredicted by 90% with respect to the experimental results for specimen RW2 (Fig. 12b) and between 15 and 54% for specimen RW-A15 (Fig. 12d). This contrasts with the results obtained with the default NLT model, where the overprediction was about 36% for RW2, and between 10 and 40% for

Table 3 Comparison of simulated and measured response quantities simulated by the HyNLT-F model

Specimen ID	$\Delta_{y, \text{sim}}/\Delta_y$	$V_{y, \text{sim}}/V_y$	$V_{\text{max, sim}}/V_{\text{max}}$	$EI_{\text{eff, sim}}/EI_{\text{eff}}$	$GA_{\text{eff, sim}}/GA_{\text{eff}}$
RW2	0.72	0.98	0.93 ^a	1.09	0.45
WSH6	1.12	0.96	0.94	N/A ^b	N/A ^b
RW-A15	0.60	0.98	0.99	1.48	1.85

^a V_{max} is better simulated for $\alpha \leq 1/2$, but this value does not allow to assess shear and flexure response independently, as these displacements were measured at first story height

^bExperimental results not available

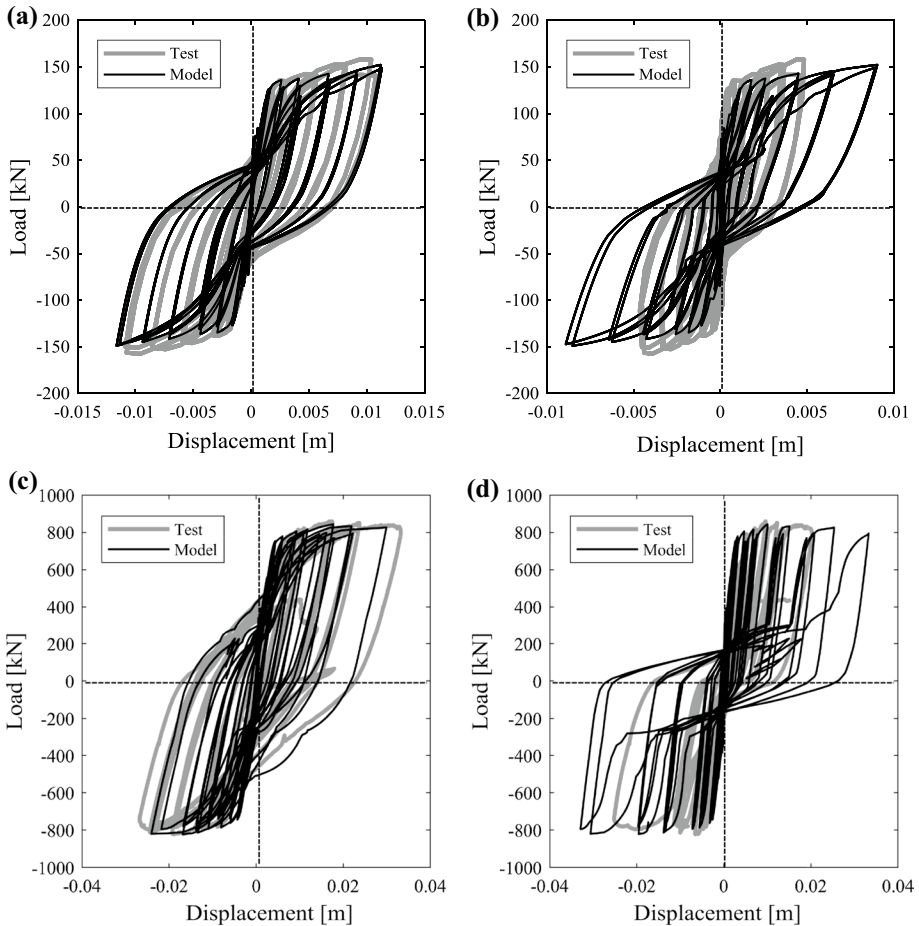


Fig. 12 First-story load-deformation responses for specimen RW2 simulated by the HyNLT-F model: **a** flexure, **b** shear. Load-deformation responses for specimen RW-A15 simulated by the HyNLT-F model: **c** flexure, **d** shear

RW-A15. Figure 13 shows the flexure–shear interaction for a bottom NLT panel with aspect ratio of $\alpha = 1.0$. Similar to the comparison presented in Fig. 8, the average slope of the predicted and observed flexure–shear behavior is included in Fig. 13. The average slope of the simulated curves is 1.25- and 1.32-times larger than the slope of the experimental results for RW2 and RW-A15, respectively. This indicates shear displacement is overestimated by the HyNLT-F model, a feature also observed in the default NLT model. Equation (5) indicates that the accuracy of the estimation of shear displacements is dependent on the quality of the simulated distribution of rotation and curvature over height (represented by the variable η). Both, the default NLT and the HyNLT-F models tend to overestimate shear displacements because of the inherent mesh-size effect, as rotation and curvature are discretized along the height of the NLT panel. For example, for a height equal to that of the bottom NLT panel, actual rotations can be approximated as a straight line from zero at the base to certain value at the top of the panel; nevertheless, when implementing Eq. (5), rotations assigned to the bottom panel cannot be

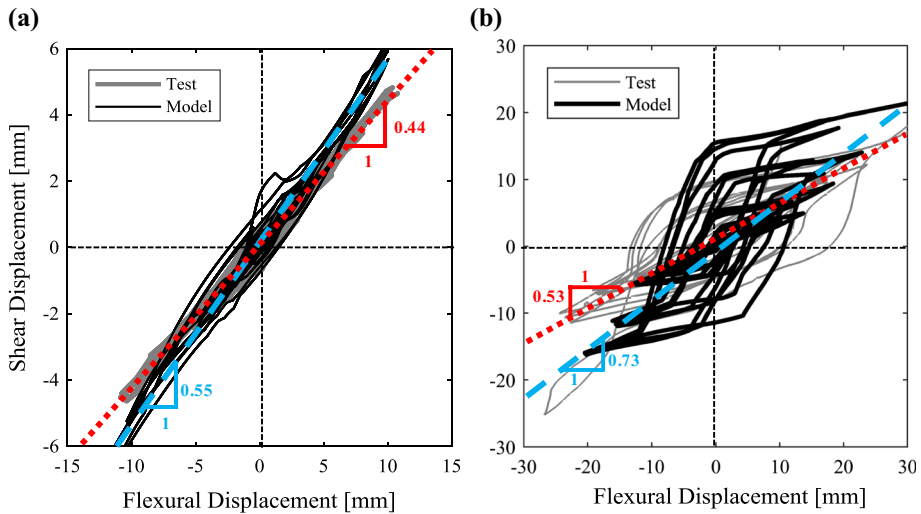


Fig. 13 Estimated flexure-shear interaction with $\alpha=1.0$

set equal to zero at the base due to the discrete nature of this distribution. This issue can be overcome using a finer mesh for the HyNLT-F, but the authors consider results obtained are within acceptable margin for most engineering problems. Results show that the HyNLT-F model approach stands among the few available models capable of simulating coupled shear-flexure interaction, while allowing reduction in running time, as will be shown in Sect. 5.

Simulating the nonlinear nature of the vertical strain profile within the critical section (Thomsen and Wallace 2004) is a notable feature of the NLT model because it may prevent overestimations of tensile strain demands if the Euler-Bernoulli hypothesis of linear strain distribution is employed. Figure 14 presents the axial strains field for a tip drift ratio of 1.5%, which is close to the instant when the peak load is reached for both specimens. At this instant, the base section is mostly in tension, with the extreme wall-boundary element being the only element in compression. For the direction of loading shown, deformations in the NLT panel show a distinctive feature: the flexure- and

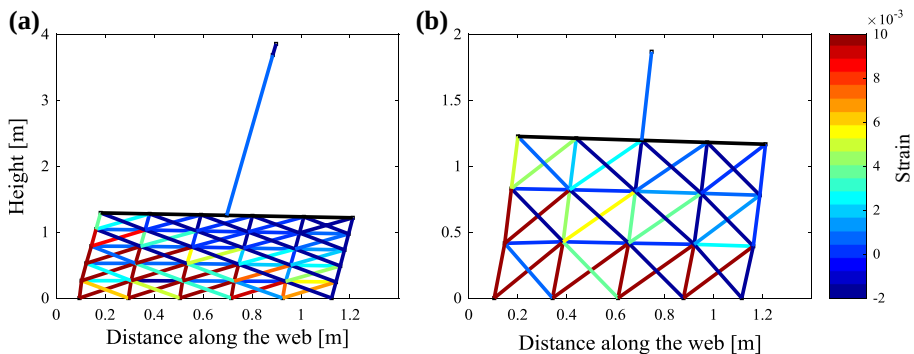


Fig. 14 Distribution of Axial Strains simulated by the HyNLT-F model at drift of 1.5% for Specimen: **a** RW2, **b** RW-A15

shear-dominated displaced shapes are apparent from the difference in concavity for specimens RW2 (e.g. concave to the right) and RW-A15 (e.g. concave to the left), respectively.

Figure 15 shows the distribution of vertical strains along the wall base simulated by the HyNLT-F model for the two specimens. For specimen RW2 the vertical tensile strains are overestimated by factor 1.20 for all levels of drift but in general show a good agreement with the experimental results. Indeed, in comparison with the default NLT model, the simulated behavior in tension of the hybrid model agrees better with test results for the lower levels of drift demand (Fig. 9). For specimen RW-A15, tensile strains are overestimated by factor 1.5 to 2.0 for levels of drift demand lower than 1.5% but reduces to approximately 1.25 for greater levels of drift demand. As occurs for the default NLT model, compressive strains in the hybrid models are underestimated in both specimens, more so for the flexure-dominated specimen. This feature that can be partially corrected if a finer mesh is employed, but the penalty of requiring larger computational time to prepare and run the models might not compensate for a marginal improvement.

4 Case study: multistory frame-wall system response

An implementation example of the hybrid NLT-Fiber wall (HyNLT-F model) model is presented next. Figure 16 shows an 8-story 5-bay RC frame, representative of an older non-ductile building, coupled to a lightly reinforced wall of 2.5 m in length, and aspect ratio of approximately 11. The geometry, the materials and the reinforcement characteristics of the frame and the wall can be found in Galanis and Moehle (2015) and Parra et al. (2019), respectively. The building system shown was analyzed under the ATC-78 project (Holmes et al. 2017) to develop a simple procedure for assessing the collapse risk of non-ductile RC frame-wall buildings. Nonlinear force-based beam-column elements are used to model the response of the structural elements of the frame. Specifically, the *beam-with-hinges* model by Scott and Fences (2006) was implemented in OpenSees for this purpose, with plastic hinge length equal to one-half the member depth. Three different models of the wall were

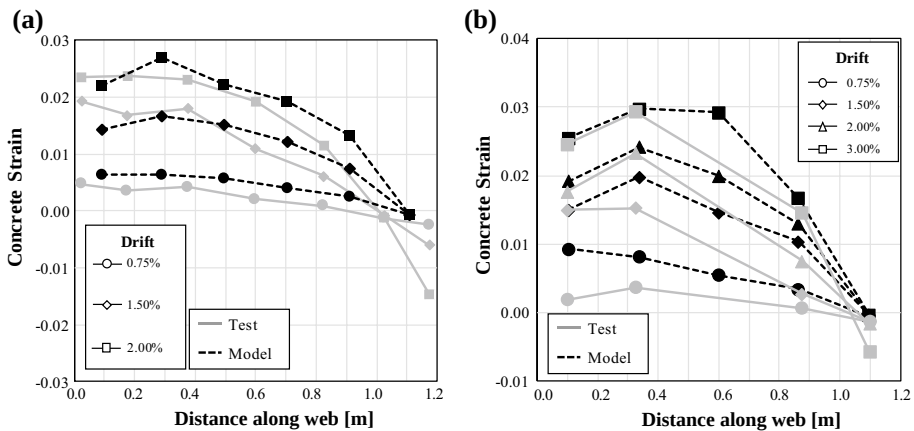


Fig. 15 Distribution of Vertical Strains along the Wall Base simulated by the HyNLT-F model for Specimen: **a** RW2, **b** RW-A15

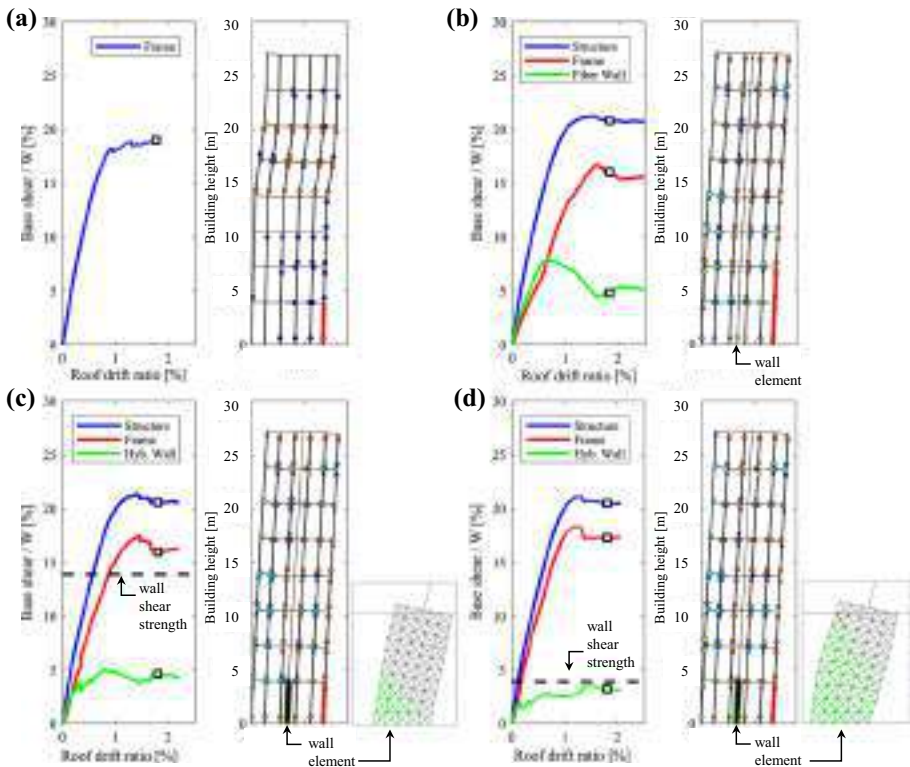


Fig. 16 Pushover curves for different wall model formulation: **a** bare frame; **b** frame with Fiber wall; **c** frame with HyNLT-F wall; **d** frame with HyNLTW-F wall

implemented: (i) a force-based fiber beam–column model with distributed plasticity (Fiber model), (ii) a hybrid model with a NLT implementation for the first story panel and a Fiber model for the wall segments above the first story (HyNLT-F model), and (iii) a HyNLT-F model with a deliberately weaker shear strength at the first story panel, to excite nonlinear shear–flexure interactions at the critical section (HyNLTW-F model).

4.1 Pushover response

The frame and the wall were coupled by a horizontal displacement constraint at each floor and are incrementally pushed with an inverted triangular load pattern, resembling the first mode shape of the system. For context, Fig. 16a shows the response of the non-ductile bare frame, which exhibits an upper-story mechanism at a maximum roof drift ratio (RDR) of 1.8%, approximately, and a base shear demand (V_b) of approximately 19% of the seismic weight (W). Figure 16b–d show the response of the non-ductile bare frame coupled to the wall at RDR=1.8%. A main feature of the coupled response is that the wall is able to modify the non-ductile mechanism that formed in the bare frame, allowing the system to achieve larger RDRs. Figure 16b shows the global response of the building with the Fiber model wall. Two quadrature rules were used: (i) for the first-story element, the Gauss–Radau scheme with 2 integration points, including one at the base, was

used, while (ii) the Gauss–Legendre integration scheme with 3 integration points was used elsewhere. Parra et al. (2019) proved that the so-called GRa2GLe3 mixed formulation was efficient and accurate for modeling the global response of slender walls interacting with frames. Because of its short length and light reinforcement, addition of the wall increases the system strength only marginally to 21.2%. The peak capacity of the system is recorded at 1.4% RDR, where the wall takes 6.2% of normalized base shear (V_b/W). Maximum wall demand is $V_{bw}/W=7.8\%$ at 0.65% RDR. Figure 16c shows the global response of the building with the HyNLT-F model wall. A similar global response is observed, with maximum strength $V_b/W=21.3\%$ also at $\text{RDR}=1.4\%$. At this instant, the HyNLT-F wall takes 4.0% of the normalized base shear. The force–displacement relationship of the wall shows an apparent stiffness softening after the onset of cracking achieving a maximum strength of $V_{bw}/W=4.9\%$ at 0.80% RDR. It is also shown that the shear demand on the wall is well below its capacity. Figure 16d shows that the implementation of the hybrid model with reduced shear strength increases the demand on the frame, but the overall response is similar to the previously described model because the shear demand on the HyNLTW-F wall is still below its capacity. The blown-up figures around the NLT walls show how their deformed shape differ, making apparent a flexure-dominated displacement of the NLT panel in Fig. 16c while the weaker wall exhibits shear deformations (Fig. 16d). A question that arises is whether variations in the lateral force distributions may trigger shear yielding of the walls. This issue is explored next in the dynamic response domain, where distribution of inertial forces along the building height may not agree with the inverted triangular load tested, hence generating a lateral load pattern over the wall with a lower resultant arm, that may preclude flexural yielding but instead may trigger shear failure.

4.2 Dynamic response

The dynamic analyses on the models are performed using the Conditional Scenario Spectra (CSS) ground-motion set selected by Arteta and Abrahamson (2019). The CSS set is hazard-consistent with a site in the Bay Area of California, United States. It is organized in 9 hazard levels of increasing intensity, representing the expected seismic demand at the site from 100 to 100,000 return period. The CSS set comprises 804 horizontal-component ground motions, whose spectral shapes can be found in (Arteta 2015). Figure 17 compares the maximum roof drift ratio demand of the frame-wall systems with the three wall configurations considered. Data shown is for 335 analyses with the CSS ground motions, comprising five return periods $\text{TR}=[100, 220, 500, 1000 \text{ and } 4000]$ years, to ensure the system is demanded elastically and within its plastic range of response; the corresponding median spectral coordinates at the fundamental period of the frame-wall structure are $S_a(T_1)=[0.11, 0.16, 0.24, 0.31, 0.50]$ g, respectively.

Displacement demand at the roof level is consistent among the three models, with apparent departures from the 1:1-line for ground motions with $\text{TR}=4000$ years, where deviations from a flexure-dominated response in the walls started to occur, and some convergence problems were evidenced. This indicates that the HyNLT formulation is stable, producing reasonable results, which are comparable to the well-known fiber-based model.

The study of the displaced shape of the structure helps to further explore the impact of the NLT formulation at the critical section of the wall. Under a given ground motion run, parameter λ is defined as the ratio between the maximum story drift ratio of each story ($\text{SDR}_{\max}$) and the maximum roof drift ratio ($\text{RDR}_{\max}$) of the structure. Figure 18 depicts how the three wall models modify the global displacement response of the bare frame.

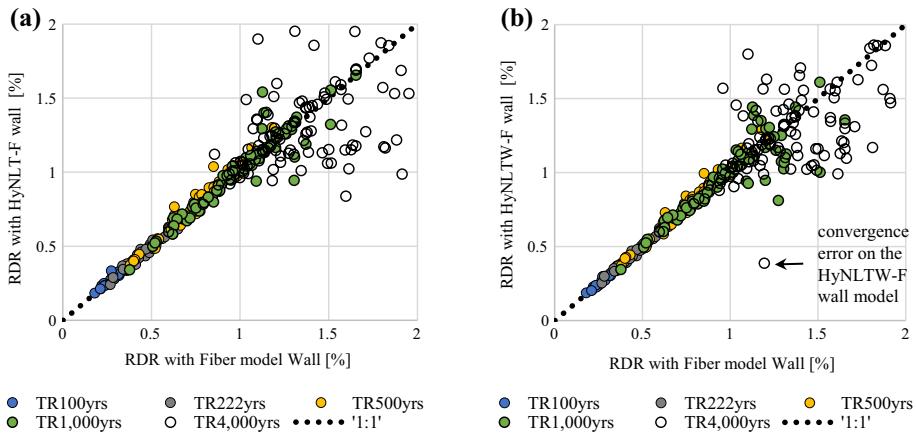


Fig. 17 Comparison of dynamic roof drift ratio demanda among models: **a** model with Fiber wall versus model with HyNLT-F wall; **b** model with Fiber wall versus model with HyNLTW-F wall

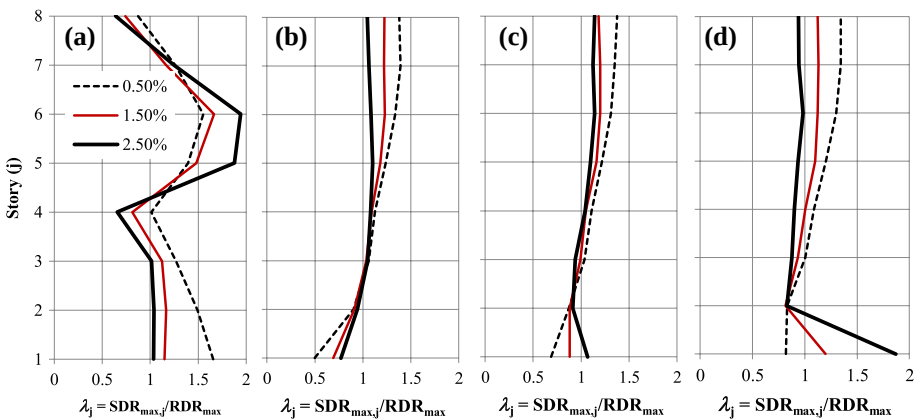


Fig. 18 λ -values at different roof drift ratio levels: **a** bare frame model; **b** model with Fiber wall; **c** model with HyNLT-F wall; **d** model with HyNLTW-F wall

Data shown correspond to median λ -values of runs previously binned by $\text{RDR}_{\max}$ at three discrete levels of increasing demand. For instance, the 0.5% series were gathered from runs that demanded the structure elastically, while the 2.5% series were gathered from runs that demanded the structure within its inelastic range of response. The λ -value distribution allows assessing the location along the building height where lateral distortion concentrations are likely. Figure 18a confirms the previous observations that an upstairs mechanism is likely for the bare frame. In this case, the dynamic analyses confirm that the distortion concentration may occur at the 5th but would also occur at the 6th story. Figure 18b shows that the addition of the Fiber wall prevented a lateral distortion concentration over the height, generating a uniform cantilever-like λ -value distribution, which resembles the shape of the first derivative with respect to height of the elastic deformed shape of a cantilever wall. The implementation of the HyNLT wall does not exhibit large variations of the story drift distribution for lower intensity demand (i.e. for the $\text{RDR}_{\max} = 0.5\%$ series) as

compared to the Fiber model implementation (Fig. 18c). Nevertheless, because of dynamic higher-mode effects and frame wall interaction, a λ -value increment is observed at the first story for larger-demand scenarios. This is possible thanks to the capability of the NLT panel to accommodate shear distortions. This observation becomes more evident for the wall model with weaker shear strength (Fig. 18d). The HyNLTW-F implementation shows larger concentration of drift demand at the first story, consequently not improving the bare-frame response, but just modifying the location of the weak story. It is worth noting that this response feature may be modeled with a shear spring, connected in series with the force-based element at the first story, to simulate a shear distortion. However, this uncoupled approach may only work properly for strain hardening response in flexure and shear. Otherwise, if either deformation component strain softens or behave elastic-perfectly-plastic, the two-element system will behave as the weaker component alone, producing an unlikely structural behavior.

5 Time of execution of the analyses

The time of analyses execution denoted here as *running time* is evaluated next to compare the efficiency of the hybrid approach against the default NLT model. The evaluation is done for the cyclic analyses in Sect. 3, and for the dynamic analyses of the frame-wall building in Sect. 4.

5.1 Nonlinear static analyses running times

Running times of the reversed cyclic loading analyses are evaluated in terms of the reduction in the number of elements and α -values. The baseline of the comparison is given by the running times of the default NLT model. Table 4 summarizes the number of elements required to construct the NLT-based models in Sect. 3, for α -values that simulate the global response properly. For example, for $\alpha=3/4$, which ensures the proper modeling of the global response of all specimens, the number of elements needed to reproduce the response of specimens RW2, WSH6, and RWA15 reduces by 75%, 70%, and 51%, respectively, with respect to the number of elements required to implement the default NLT. A major reduction in the number of elements required to construct a model is accompanied by a reduction in the time required to construct the model in OpenSees, which will facilitate the use of the NLT modeling approach by the engineering community. Figure 19a presents the relative computational time required to run the quasi-static analyses using the α -values in

Table 4 Number of elements required to construct the NLT models

Default-NLT	RW2	WSH6	RWA15
	338	290	85
<i>Hybrid</i>			
$\alpha = 1$	107 (68%)	146 (50%)	52 (39%)
$\alpha = 3/4$	86 (75%)	88 (70%)	42 (51%)
$\alpha = 1/2$	65 (81%)	59 (80%)	–
$\alpha = 1/4$	23 (93%)	–	–

Values in parenthesis represent the reduction, in percentage, of the number of elements with respect to the default NLT model

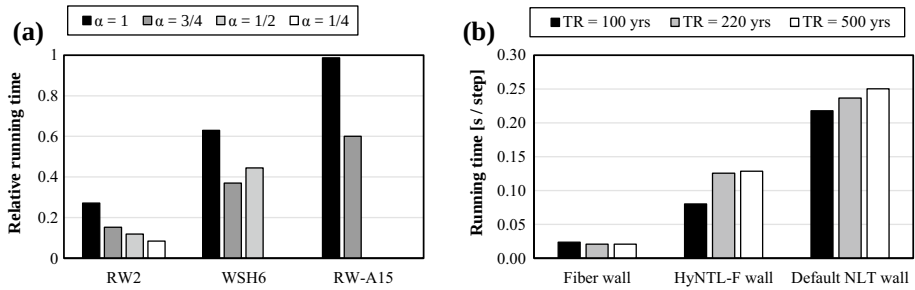


Fig. 19 Computational time evaluation: **a** time required by the hybrid NLT-Fiber model to estimate the pseudo-static global response of specimens RW2, WSH6 and RW-A15 for different values of α ; **b** time required for frame-wall structure with different wall model implementations to run ground motions of different intensity levels

Table 4. For a given experimental specimen, the relative computational time is estimated as the ratio between the running time of a hybrid model and that of the default NLT model. For $\alpha = 3/4$, the hybridization of specimens RW2, WSH6, and RWA15 represents, respectively, an 85%, 63%, and 40% reduction in running time with respect to the default NLT model. It is noted that the implementation of the HyNLT-F model reduces the computational cost more efficiently for the flexure-dominated specimens.

5.2 Nonlinear dynamic analyses running times

Running times of the dynamic analyses are evaluated in terms of wall model implementation, ground motion intensity level, and number of data points of the selected accelerograms. Three frame-wall models selected here for comparison comprise those on Fig. 16b and c (i.e., frame with *Fiber wall* and with *HyNLT-F wall*, respectively), and a frame with a full NLT model implemented for the wall of each floor (frame with default *NLT wall*). Three different intensity levels were selected from the CSS set: TR = [100, 220, 500]. From each hazard level, a subset of ground motions which do not offer converge problems to any model were chosen. A total of 30 accelerograms discretized into 378,720 data points with a total of 1830 s of analysis were employed in the assessment. The analyses were performed in a 6-core Intel® Xenon® E5-2620 server machine with maximum frequency of 2.5 GHz, 16 GB of RAM, and 7200 RPM hard drive, dedicated solely for these analyses. For each hazard level, the three models ran concurrently under the same set of ground motions, and the PC was left alone until the last accelerogram of the slowest model finished running. Then, the following hazard level was run in the same manner after restarting the computer. The entire process took approximately 1 week to complete.

Figure 19b presents a summary of the average time spent solving each step of dynamic analysis for different intensities and wall model implementations. It is noted that increasing the hazard level intensity increases the running times marginally. On average, for all intensity levels, the model with the *Fiber wall* spent 0.2 s per step [s/step], while those with the *HyNLT-F* and the default *NLT* spent 0.11 and 0.23 s/step, respectively. In summary, the model with the *HyNLT-F* implementation was between 3.4 and 6.2-times slower than that with the *Fiber wall* implementation and was between 1.9 and 2.7-times faster than that with the default *NLT* wall implementation, confirming the efficiency of the approach proposed herein.

6 Final comments

In this paper experimental data of three reinforced concrete wall panels of different aspect ratio was used to evaluate the capabilities of the nonlinear truss model (NLT model) to simulate their global and local response. Two key distinctive features of the NLT model tested were (i) its capability to model inelastic shear accounting for coupled flexure and shear interaction, and (ii) its capability of simulating the nonlinear nature of normal strain distribution in deep members. This feature makes it the only available macro-model that is not programmed based on the Euler–Bernoulli assumption. The numerical computation with the NLT model showed a good agreement with the experimentally recorded responses in terms of the force–displacement relationships, shear–flexure interaction, and vertical strain distribution at the base, under different levels of drift demand. A disadvantage of the NLT model is that it may be time consuming in its construction and is computationally expensive as compared to nonlinear beam–column elements.

For mitigating this disadvantage, a hybridization of the default NLT model is proposed by coupling a NLT panel at the critical section of the wall (where most shear–flexure interaction would occur) with a force-based fiber-beam–column element for other sections (HyNLT-F model). The NLT panel height was defined in terms of the wall length (e.g. $h_{\text{HyNLT}} = \alpha l_w$). Numerical results of the implementation of the HyNLT-F model with various NLT panel heights were compared against the global and local response of three wall panel experiments having aspect ratio of 1.5, 2.3 and 3.1, and tested under reversed cyclic loading. It was shown that minimum α -values to reasonably model the global response of the squat, midrise and slender wall were 3/4, 1/2 and 1/4, respectively. Features of the expected response of slender and squat walls are properly simulated by the hybrid NLT model, including shear distortion contributions to total displacement, inelastic shear–flexure interaction, and nonlinear normal strain distributions.

A multistory building case study was implemented with a HyNLT-F model to evaluate the response under static and dynamic loading of a non-ductile RC frame coupled to a lightly reinforced wall. The results suggested that under certain demand conditions, shear–flexure interaction of the wall may trigger response features on the wall that may impact the overall frame-wall coupled behavior, and that cannot be modeled with fiber beam–column elements representative of the wall.

The efficiency of the HyNLT-F model with respect to the default NLT model was measured in terms of running time cost in the static and dynamic domain of analysis. For the case study of a multistory frame-wall system, the building with the HyNLT-F model ran between 2- and 3-times faster than the building with the default NLT wall for various levels on seismic demand. A reduced programing time is also part of the benefits of the hybridization because of the reduced number of elements and nodes required to construct a HyNLT-F model with respect to the default NLT model.

Future research with the HyNLT-F model should include recommendations on modeling failure mechanisms of walls. This will require calibrating different β -curves for individual specimens based on the expected failure mechanism and including reinforcing steel models that account for bar rupture and buckling. One way of implementing such a model, is using an asymmetric backbone for steel model to include the effects of reduction in compressive strength due to rebar buckling (Arteta 2015) and bar rupture. The Opensees uniaxial *Hysteretic* material model developed by Scott and Filippou (2016) may be implemented for this purpose using Mazzoni et al. (2007) recommendations.

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References

- Arteta CA (2015). Seismic response assessment of thin boundary elements of special concrete shear walls. (10188056 Ph.D.), University of California, Berkeley, Ann Arbor. Retrieved from. <https://search.proquest.com/docview/2031098506?accountid=41515> ProQuest Dissertations & Theses A&I database
- Arteta CA, Abrahamson NA (2019) Conditional scenario spectra (CSS) for hazard-consistent analysis of engineering systems. *Earthq Spectra* 35(2):737–757. <https://doi.org/10.1193/102116eqs176m>
- Beyer K, Dazio A, Priestley MJN (2011) Shear deformations of slender reinforced concrete walls under seismic loading. *ACI Struct J* 108(2):167–177
- Coleman J, Spacone E (2001) Localization issues in force-based frame elements. *J Struct Eng* 127(11):1257–1265. [https://doi.org/10.1061/\(ASCE\)0733-9445\(2001\)127:11\(1257\)](https://doi.org/10.1061/(ASCE)0733-9445(2001)127:11(1257))
- Dazio A, Beyer K, Bachmann H (2009) Quasi-static cyclic tests and plastic hinge analysis of RC structural walls. *Eng Struct* 31(7):1556–1571. <https://doi.org/10.1016/j.engstruct.2009.02.018>
- Filippou FC, Popov EP, Bertero VV (1983) Effects of bond deterioration on hysteretic behavior of reinforced concrete joints. Report UCB/EERC-83/19. Retrieved from Berkeley
- Fischinger M, Rejcek K, Isakovic T (2012) Modeling inelastic shear response of RC walls. Paper presented at the 15th world conference on earthquake engineering, Lisbon, Portugal
- Fischinger M, Rejcek K, Isaković T (2014) Inelastic shear response of rc walls: a challenge in performance based design and assessment. In: Fischinger M (ed) Performance-based seismic engineering: vision for an earthquake resilient society, vol 32. Springer, Dordrecht
- Galanis PH, Moehle JP (2015) Development of collapse indicators for risk assessment of older-type reinforced concrete buildings. *Earthq Spectra* 31(4):1991–2006
- Hiraiishi H (1984). Evaluation of shear and flexural deformations of flexural type shear walls. Paper presented at the 8th world conference on earthquake engineering, San Francisco, US
- Holmes WT, Liel AB, Mehrai M, Moehle JP, Somers P (2017) Seismic evaluation of older concrete frame, frame-wall, and bearing wall buildings for collapse potential, ATC-78-6. Preliminary report. Retrieved from Redwood City
- Hoshikuma J, Kawashima K, Nagaya K, Taylor AW (1997) Stress-strain model for confined reinforced concrete in bridge piers. *J Struct Eng* 123(5):624–633. [https://doi.org/10.1061/\(ASCE\)0733-9445\(1997\)123:5\(624\)](https://doi.org/10.1061/(ASCE)0733-9445(1997)123:5(624))
- Jansen DC, Shah SP (1997) Effect of length on compressive strain softening of concrete. *J Eng Mech* 123(1):25–35
- Jiang H, Kurama YC (2010) Analytical modeling of medium-rise reinforced concrete shear walls. *ACI Struct J*. <https://doi.org/10.14359/51663812>
- Kent DC, Park R (1971) Flexural members with confined concrete. *J Struct Div* 97(7):1969–1990
- Kolozvari K, Orakcal K, Wallace JW (2014) Modeling of cyclic shear–flexure interaction in reinforced concrete structural walls. I: theory. *J Struct Eng* 141(5):04014135. [https://doi.org/10.1061/\(ASCE\)ST.1943-541X.0001059](https://doi.org/10.1061/(ASCE)ST.1943-541X.0001059)
- Kolozvari K, Arteta C, Fischinger M, Gavridou S, Hube M, Isakovic T, Wallace J (2018) Comparative study of state-of-the-art macroscopic models for planar reinforced concrete walls. *ACI Struct J* 115(6):20. <https://doi.org/10.14359/51710835>
- Lu Y, Panagiotou M (2014) Three-dimensional cyclic beam-truss model for nonplanar reinforced concrete walls. *J Struct Eng* 140(3):04013071. [https://doi.org/10.1061/\(ASCE\)ST.1943-541X.0000852](https://doi.org/10.1061/(ASCE)ST.1943-541X.0000852)
- Lu Y, Panagiotou M, Koutoumanos I (2014) Three-dimensional beam-truss model for reinforced-concrete walls and slabs subjected to cyclic static or dynamic loading, PEER Report 2014/18, Pacific Earthquake Engineering Research Center. University of California, Berkeley
- Lu Y, Henry RS, Gultom R, Ma MT (2017) Cyclic testing of reinforced concrete walls with distributed minimum vertical reinforcement. *J Struct Eng* 143(5):04016225. [https://doi.org/10.1061/\(ASCE\)ST.1943-541X.0001723](https://doi.org/10.1061/(ASCE)ST.1943-541X.0001723)
- Massone LM, Wallace JW (2004) Load-deformation responses of slender reinforced concrete walls. *ACI Struct J* 101(1):103–113
- Massone LM, Orakcal K, Wallace JW (2006) Shear–flexure interaction for structural walls. Special Publication, New York. <https://doi.org/10.14359/18215>

- Mazzoni S, McKenna F, Fenves GL (2007) Steel02 & hysteretic—material behavior. OpenSees comparison of modelling tools. Retrieved from <http://opensees.berkeley.edu/OpenSees/manuals/comparisonManual/2773.htm>
- McKenna F, Fenves GL, Scott MH, Jeremic B (2000) Open system for earthquake engineering simulation (OpenSees) (version 2.4.3.). Pacific Earthquake Engineering Research Center, University of California, Berkeley
- Mohd Yassin MH (1994) Nonlinear analysis of prestressed concrete structures under monotonic and cyclic loads. Retrieved from
- Morsch E (1922) *Der Eisenbetonbau-Seine Theorie und Anwendung* (Reinforced concrete construction—Theory and application) (5th ed. vol. 1). Stuttgart, Germany
- Panagiotou M, Restrepo JJ, Schoettler M, Kim G (2012) Nonlinear cyclic truss model for reinforced concrete walls. *ACI Struct J* 109:205
- Park H, Eom T (2007) Truss model for nonlinear analysis of rc members subject to cyclic loading. *J Struct Eng* 133(10):1351–1363. [https://doi.org/10.1061/\(ASCE\)0733-9445\(2007\)133:10\(1351\)](https://doi.org/10.1061/(ASCE)0733-9445(2007)133:10(1351))
- Parra PF, Arteta CA, Moehle JP (2019) Modeling criteria of older non-ductile concrete frame-wall buildings. *Bull Earthq Eng* (in press)
- Ritter W (1899) *Die Bauweise Hennebique*. Schweizerische Bauzeitung 33(7):59–61
- Schlaich J, Schäfer K (1991) Design and detailing of structural concrete using strut-and-tie models. *Struct Eng* 69(6):113–125
- Schlaich J, Weischede D (1982) Detailing of concrete structures: first draft of a design manual (in German), bulletin d'Information 150. Comité Euro-International du Béton, Paris
- Scott MH, Fenves GL (2006) Plastic hinge integration methods for force-based beam–column elements. *J Struct Eng* 132(2):244–252. [https://doi.org/10.1061/\(Asce\)0733-9445\(2006\)132:2\(244\)](https://doi.org/10.1061/(Asce)0733-9445(2006)132:2(244))
- Scott M, Filippou FC (2016) Hysteretic material. Retrieved from. http://opensees.berkeley.edu/wiki/index.php/Hysteretic_Material
- Scott BD, Park R, Priestley MJN (1982) Stress–strain behavior of concrete confined by overlapping hoops at low and high strain rates. *ACI J Proc* 79(1):13–27. <https://doi.org/10.14359/10875>
- Spacone E, Filippou FC, Taucer FF (1996) Fibre beam–column model for non-linear analysis of R/C frame: part I formulation. *Earthq Eng Struct Dyn* 25:711–725
- Stevens NJ, Uzumeri SM, Collins MP, Will TG (1991) Constitutive model for reinforced concrete finite element analysis. *ACI Struct J* 88:49
- Thomsen JH, Wallace JW (2004) Displacement-based design of slender reinforced concrete structural walls-experimental verification. *J Struct Eng Asce* 130(4):618–630. [https://doi.org/10.1061/\(Asce\)0733-9445\(2004\)130:4\(618\)](https://doi.org/10.1061/(Asce)0733-9445(2004)130:4(618))
- Tran TA, Wallace JW (2015) Cyclic testing of moderate-aspect-ratio reinforced concrete structural walls. *ACI Struct J* 112(6):653–665
- Vecchio FJ, Collins MP (1986) The modified compression-field theory for reinforced concrete elements subjected to shear. *ACI J* 83(2):219–231
- Vecchio FJ, Collins MP (1993) Compression response of cracked reinforced concrete. *J Struct Eng* 119(12):3590–3610