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NUMERICAL FRAMEWORK FOR SEISMIC COLLAPSE ASSESSMENT OF RIGID WALL-FLEXIBLE DIAPHRAGM STRUCTURES

M. Koliou[1] , A. Filiatrault[2], D.J. Kelly[3] and J. Lawson[4]

ABSTRACT

This study focuses on the development of a two dimensional (2D) simplified numerical framework of rigid wall-flexible diaphragm (RWFD) structures that can be used to validate seismic design approaches. This type of low-rise industrial buildings, which is widely used in North America, incorporates rigid in-plane concrete or masonry walls and flexible in-plane wood, steel or "hybrid" roof diaphragms. The numerical modeling is detailed enough to capture the nonlinear seismic response of RWFD buildings, but simplified enough to efficiently conduct a large number of nonlinear time-history dynamic analyses. The 2D numerical modeling framework is based on a three step sub-structuring approach including: (1) a hysteretic response database for diaphragm connectors, (2) a 2D inelastic roof diaphragm model incorporating hysteretic connector response and (3) a simplified 2D building model incorporating hysteretic diaphragm model response. The diaphragm connector database (step 1) was developed for both wood and steel deck connectors using cyclic test data available in the literature. Two well-known hysteretic models (Wayne-Stewart and CUREE-SAWS) were used for estimating/fitting hysteretic parameters of each connector type. The analytical model of the inelastic roof diaphragm (step 2) was generated to account for the elastic shear deformation of deck panels, elastic flexural deformations of chord members as well as inelastic deformations of deck-to-frame connectors (from the connector database-step 1). This model includes monotonic and cyclic analysis capabilities. The last step of the proposed analytical framework is a simplified two dimensional model of a RWFD building developed in RUAUMOKO2D to account for the inelastic response of roof diaphragms (based on the analytical roof diaphragm model-step 2) and the out-of-plane walls as well as second order (P-Δ) effects. Both the proposed analytical model of the roof diaphragm and the proposed simplified building model were validated with experimental and analytical studies available in the literature. Furthermore, a sensitivity study was conducted to examine the effect of: (i) analysis time step, (ii) different base fixity of the out-of-plane walls, (iii) P-Δ effects, (iv) inherent viscous damping and (v) direction of shaking on the collapse assessment of RWFD structures.

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This study focuses on the development of a two dimensional (2D) simplified numerical framework of rigid wall-flexible diaphragm (RWFD) structures that can be used to validate seismic design approaches. This type of low-rise industrial buildings, which is widely used in North America, incorporates rigid in-plane concrete or masonry walls and flexible in-plane wood, steel or "hybrid" roof diaphragms. The numerical modeling is detailed enough to capture the nonlinear seismic response of RWFD buildings, but simplified enough to efficiently conduct a large number of nonlinear time-history dynamic analyses. The 2D numerical modeling framework is based on a three step sub-structuring approach including: (1) a hysteretic response database for diaphragm connectors, (2) a 2D inelastic roof diaphragm model incorporating hysteretic connector response and (3) a simplified 2D building model incorporating hysteretic diaphragm model response. The diaphragm connector database (step 1) was developed for both wood and steel deck connectors using cyclic test data available in the literature. Two well-known hysteretic models (Wayne-Stewart and CUREE-SAWS) were used for estimating/fitting hysteretic parameters of each connector type. The analytical model of the inelastic roof diaphragm (step 2) was generated to account for the elastic shear deformation of deck panels, elastic flexural deformations of chord members as well as inelastic deformations of deck-to-frame connectors (from the connector database-step 1). This model includes monotonic and cyclic analysis capabilities. The last step of the proposed analytical framework is a simplified two dimensional model of a RWFD building developed in RUAUMOKO2D to account for the inelastic response of roof diaphragms (based on the analytical roof diaphragm model-step 2) and the out-of-plane walls as well as second order (P-Δ) effects. Both the proposed analytical model of the roof diaphragm and the proposed simplified building model were validated with experimental and analytical studies available in the literature. Furthermore, a sensitivity study was conducted to examine the effect of: (i) analysis time step, (ii) different base fixity of the out-of-plane walls, (iii) P-Δ effects, (iv) inherent viscous damping and (v) direction of shaking on the collapse assessment of RWFD structures.

Introduction

Rigid Wall-Flexible Diaphragm (RWFD) buildings are widely used for light industrial and low-rise commercial construction in the United States and Canada. These buildings are usually framed with exterior precast concrete (plant cast or site cast) or masonry walls, interior columns and horizontal roof members. The horizontal roof framing members are designed to act as diaphragms and can be categorized into wood, steel and "hybrid" roof systems. Steel roof systems are framed with steel deck, steel bar joists and joist girders. Wood deck systems consist of plywood or oriented strand board (OSB) deck fastened to wood framing using common nails, while "hybrid" systems are constructed using wood structural panels (plywood or OSB) fastened to wood cover-plates which are fastened to steel joists and joist girders.

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Rigid Wall-Flexible Diaphragm buildings have performed poorly during past earthquakes including the 1964 Alaska [1], 1971 San Fernando [1], 1987 Whittier Narrows [2], 1989 Loma Prieta [3], 1994 Northridge [1], and the 2010 and 2011 Christchurch [4] earthquakes. The main failure modes observed during these earthquake events can be categorized into: (i) wall panel-to-roof connection failures, (ii) partial out-of-plane collapse of roof and walls, (iii) damage to wall panels with large openings due to insufficient reinforcement or incorrect placement of reinforcement, and (iv) large deformations of flexible roof diaphragms [1-4].

Several experimental and analytical studies on the seismic response and performance of RWFD buildings have been conducted over the last two decades. The main focus of these studies has been the design and seismic response of the flexible roof diaphragm and connections incorporated into wood and/or steel deck systems. The numerical modeling of RWFD buildings has been a main component for these studies. Cohen et al. [5] developed a two degree-of-freedom simplified model as well as a linear elastic detailed finite element model (FEM) in the analysis program SAP2000 to match the response of masonry structures tested under seismic loading. Olund [6] developed a detailed FEM of a single story concrete tilt-up building in the Perform3D analysis program to evaluate the collapse capacity of the structure. Tellier [7] generated a two dimensional (2D) macro-model in the MATLAB platform to evaluate the seismic performance of concrete tilt-up buildings. A detailed deep horizontal plane truss numerical model of flexible roof systems was developed in the OpenSees software platform by Shrestha [8] to reproduce the dynamic characteristics and the elastic and inelastic response of tested steel deck diaphragm specimens. In most of these studies, the flexible roof diaphragm system was modeled with linear elastic elements despite the fact that the roof diaphragm is often the main source of inelasticity in the seismic response of RWFD buildings. Despite the favorable modeling features of detailed finite element models, the computational overhead needed to perform nonlinear time history dynamic analyses of RWFD buildings is an important parameter to be considered when analyzing a large number of these buildings using numerous ground motion records. For this reason, a two dimensional simplified numerical framework is proposed in this study. The proposed numerical approach is detailed enough to capture the nonlinear response of RWFD buildings, but simple enough to efficiently conduct a large number of nonlinear time-history dynamic analyses. The two dimensional numerical framework is based on a three step sub-structuring approach that includes: (1) a hysteretic response database for roof diaphragm connectors, (2) a 2D inelastic diaphragm model incorporating hysteretic connector response, and (3) a 2D building model incorporating hysteretic diaphragm model response. The proposed analytical tool and a sensitivity/parametric study on the effects of the 2D building modeling assumptions (step 3) are presented in this paper.

Numerical Framework

Each of the three modeling steps of the two dimensional numerical framework is presented in the following sections.

Roof Diaphragm Connector Database-Step 1

The first step of the proposed numerical framework involved the generation of a deck-to-framing and panel-to-panel side-lap connector database for connectors used for wood, “hybrid” and steel deck diaphragms. This database includes the hysteretic response of the connections, which is the main source of inelasticity in the flexible diaphragm in-plane seismic response. The connectors’ hysteretic properties are incorporated into numerical models of the flexible roof diaphragm. The connector database was developed for wood and steel deck connectors using cyclic test data available in the literature [9-14]. The wood connectors include common nails, cut and rolled

thread screws. The steel deck connectors include pins (powder-actuated fasteners), screws, button punches, and welds. The Wayne-Stewart [15] and CUREE-SAWS [16] hysteretic models were used for estimating the hysteretic parameters of each connector type. A set of MATLAB [17] codes was developed to optimize the hysteretic parameters associated with both models. The optimal hysteretic parameters were computed through an identification process to fit the numerical model with the experimental data of each connector test by minimizing the differences in force and deformation. The identification process was set as a constrained least squares problem that was solved using the Trust Region Reflective Algorithm. Details on the optimization procedure followed can be found in [18].

Inelastic Roof Diaphragm Model-Step 2

An analytical 2D model of the roof diaphragm was generated in the MATLAB platform. Each deck panel was modeled as a deep shear beam, while all the deck-to-framing and sidelap connectors around each panel were modeled using the Wayne-Stewart hysteretic model. The Wayne-Stewart hysteretic properties of the connectors were obtained from step 1 (connector database) of the sub-structuring modeling approach. This analytical model was developed with displacement controlled monotonic and cyclic analysis capabilities. A constant cyclic force applied at mid-span of the diaphragm was considered for the analysis on the diaphragm model in order to induce constant shear forces in the diaphragm. The total in-plane flexible diaphragm displacement, as schematically illustrated in Fig. 1, was computed as the sum of: (i) the elastic shear deformation of each individual panel, (ii) the inelastic deformations (slippage) of connectors and (iii) the elastic flexural deformations of the chord members. The algebraic equations of equilibrium developed for this analytical model were solved using the Newton-Raphson method. Analysis convergence was satisfied when the maximum load imbalance was less than a specified tolerance. The inelastic response of the roof diaphragm generated using the analytical model was further incorporated into the simplified building model of step 3.

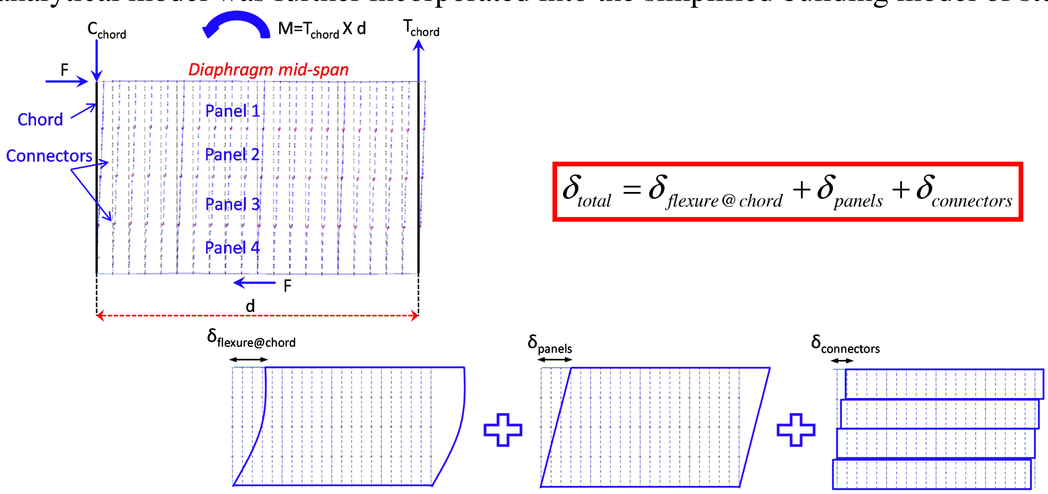


Figure 1. Illustrative representation of the analytical inelastic roof diaphragm model developed in MATLAB.

Simplified Building Model-Step 3

This modeling step introduces a two dimensional simplified model of a RWFD building developed in the RUAUMOKO2D [19] platform. The inelastic response of the roof deck diaphragm, the out-of-plane walls, the in-plane walls and the second order (P-Δ) effects were incorporated into the simplified building model. The modeling assumptions are presented in the

following sections.

Geometry, Mass and Element Properties

The geometry of a typical RWFD building consists of the rigid in-plane walls, the out-of-plane walls, the flexible roof diaphragm and the interior columns. Half of a typical RWFD building was modeled in the RUAUMOKO2D platform accounting for the building's symmetry. The total number of degrees of freedom (DOFs) incorporated into the proposed simplified building model can be computed according to Eq. (1).

$$DOFs = \left(4 \times N_{opwhb}\right) + 2 \tag{1}$$

where N_{opwhb} is the number of out-of-plane wall slices or wall panels on one side of half of the RWFD building.

An illustrative example of a simplified building model is provided in Fig. 2. This building is of dimensions 400ft x 200ft, therefore the respective DOFs for excitation along the short direction of the building computed according to equation (1) are equal to 34.

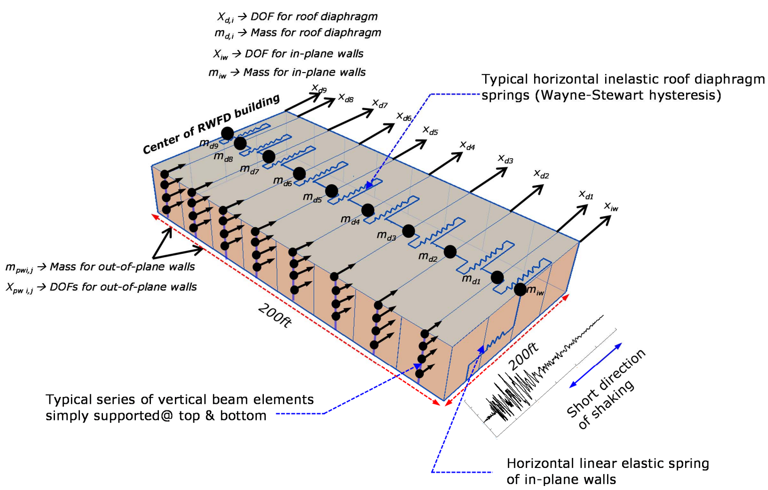


Figure 2. Illustrative example of a simplified RWFD building model developed in RUAUMOKO2D.

The in-plane inelastic shear and elastic flexural response of the flexible roof diaphragm were accounted for in the simplified model. The roof diaphragm, represented by inelastic horizontal springs, was discretized by horizontal slices coordinated with the locations of the out-of-plane wall panels. The inelastic horizontal springs were modeled to follow the Wayne-Stewart hysteretic rule implemented into the RUAUMOKO2D platform. The hysteretic properties of these springs were obtained from the diaphragm's inelastic response modeled in step 2. The tributary diaphragm mass was lumped at the center of each diaphragm slice. The out-of-plane wall panels were modeled by vertical beam elements located at the centerline of each wall panel with four masses lumped along the height. The locations of the lumped masses were at: (i) one-third height, (ii) mid-height, (iii) two thirds height, and (iv) top of the wall. The mass of the wall parapet (if applicable) was also lumped at the roof level. The wall beam elements were modeled to be simply supported at the top and bottom. A modified Takeda [20] moment-curvature hysteresis rule was implemented at each beam sub-element end to represent the inelastic flexural response of the out-of-plane wall panels. The moment curvature relationship properties were based on the reinforcement details of the out-of-plane wall panels. The post yield stiffness factor of the modified Takeda hysteresis was considered equal to the ratio of the cracked moment of inertia to the gross moment of inertia of the out-of-plane wall panels. The plastic hinge length of

the inelastic wall beam elements was set equal to walls' cross sectional thickness. The elastic flexural and shear responses of the rigid in-plane wall panels were modeled by a single horizontal linear elastic spring element and a horizontal lumped mass. The principle of virtual work and the unit load method were used to compute the equivalent linear stiffness accounting for both shear and flexural deformations. The in-plane wall panels were considered as a cantilever element with a force applied at its free end for deriving the equivalent linear stiffness as presented in Eq. (2).

$$k_{in-plane-walls} = \frac{1}{\left(h^3/3EI\right)+\left(h/GA\right)} \tag{2}$$

where E is the modulus of elasticity of the wall panels, G is the shear modulus of the wall panels, I is the moment of inertia of the wall panels, A is the cross sectional area of the wall panels and h is the height to the roof.

Damping Properties

For modeling purposes, damping is usually considered in the form of equivalent viscous damping as a percentage of the critical damping in one or more modes of vibration. Based on numerous studies and viewpoints available in the literature, the initial stiffness Rayleigh damping was selected be used in the simplified modeling accounting on its simplicity and robustness compared to the tangent stiffness proportional damping. Essential component of the damping matrix formulation for nonlinear analyses is the target damping values. Current guidelines recommend values of viscous damping considered for nonlinear analyses of structures under earthquake shaking to vary between 2% and 5% of critical [21]. A target damping value of 2% of critical at the first and second mode of vibration was selected for the simplified building model, considering that RWFD buildings are expected to have lower damping values compared to tall buildings mainly because there are fewer non-structural components.

Additional Analysis Options

Second order (P-Δ) effects were accounted into the simplified modeling of the RWFD buildings by including leaning columns under each diaphragm degree of freedom and using the large displacement analysis option in RUAUMOKO2D. The Newmark-Beta implicit direct integration method was used to solve the dynamic equation of equilibrium. The Newmark's average acceleration method (β=1/4) was considered for the nonlinear analyses, since it is unconditionally stable [22, 23] compared to the linear acceleration method (β=1/6). The analysis time step for conducting nonlinear time history analyses was set constant and equal to 0.001sec. This time step value was defined based on a trial-and-error procedure to achieve numerical convergence of the displacement and acceleration values at the center of the roof diaphragm.

Model Validation

The inelastic roof diaphragm model and the simplified building model were validated with experimental and numerical studies available in the literature. Both models matched the results of existing studies with relatively good accuracy. Due to space limitations the validation study results are not presented in this paper but can be found in [18].

Sensitivity Study

A sensitivity study was conducted to investigate the effects of different modeling assumptions of the proposed simplified building model (step 3) on the collapse assessment of a typical RWFD building. The collapse assessment of the RWFD building was evaluated by conducting Incremental Dynamic Analyses (IDA) [24] using the FEMA P695 Far Field Ground Motion

ensemble [25] and computing the median collapse intensity defined as the median 2% damped spectral acceleration at the fundamental period of the building archetype for which 50% of the earthquake motions cause its sidesway collapse. To monitor the state of the structure at the end of each nonlinear time history analysis, the building drift ratio (*BDR*) (see Eq. (3)), which is the combination of the diaphragm drift ratio (*DDR*) [26] with the in-plane wall drift ratio (*WDR*), was considered as the Damage Measure (DM). The Intensity Measure (IM) was defined by the 2% damped spectral acceleration at the fundamental period of the RWFD building.

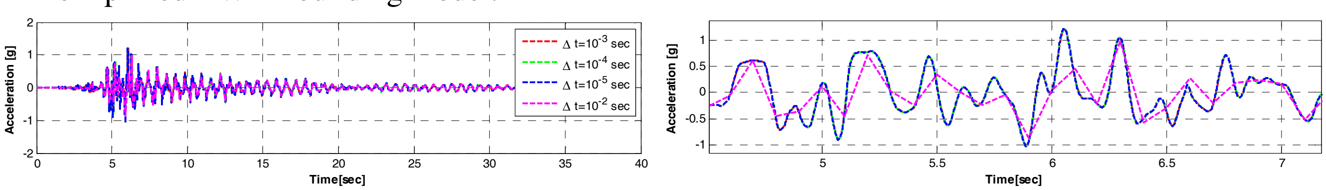
$$BDR(\%) = DDR(\%) + WDR(\%) \Rightarrow BDR(\%) = \left[\frac{x_{mid,roof}}{\left(L_{roof}/2\right)} + \frac{x_{in-plane\ walls}}{h_{wall}} \right] \times 100 \qquad (3)$$

where $x_{mid,roof}$ is the displacement at the middle of the flexible roof diaphragm, L_{roof} is the horizontal span of the flexible roof diaphragm, $x_{in\text{-}plane\ walls}$ is the displacement at the top of the rigid in-plane walls and h_{wall} is the height of roof above the foundation.

A single story concrete tilt-up building, of dimensions 200ft x 400ft, incorporating a flexible roof diaphragm was considered in this sensitivity study. A panelized "hybrid" roof system with oriented strand board was used for the horizontal diaphragm system. The tilt-up concrete wall panels had a height of 30ft to the roof plus a 3ft tall parapet. A response modification factor, R=4.0 representing intermediate precast shear walls was used for the design. This RWFD building archetype was designed according to 2012 IBC [27], ASCE 7-10 [28], 2008 NDS SDPWS [29] and ACI 318-11 [30] code provisions. Seismic design category D and risk group II were applicable. Details on the roof diaphragm nailing pattern as well as the seismic exposure design characteristics are provided in [18]. The effects of the modeling assumptions studied are associated with the: (i) analysis time step, (ii) second order (P-Δ) effects, (iii) inherent viscous damping, (iv) response of out-of-plane walls and (v) direction of shaking.

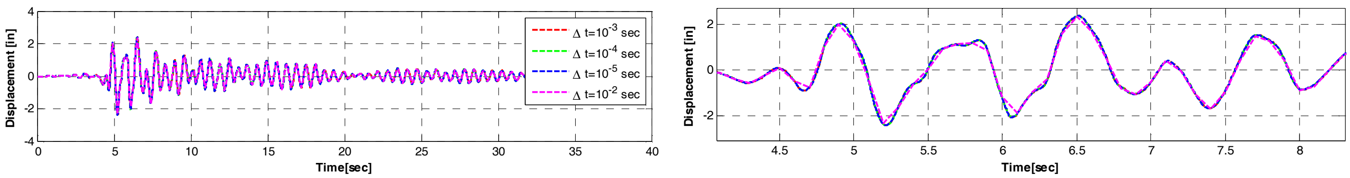
Analysis Time Step

Four different integration time steps were considered to evaluate the nonlinear time history analysis convergence. The displacement and acceleration at the center of the flexible roof diaphragm were the response parameters considered to check the analysis convergence. The four analysis time steps were selected to be integer factors of the time steps of all the FEMA P695 ground motions. The selected time step values are: 10^{-2}sec, 10^{-3}sec, 10^{-4}sec and 10^{-5}sec. The results obtained using the four different integration time steps to evaluate the response of the RWFD building archetype under one representative earthquake record (due to space limitations in this paper) are shown in Figs. 3 and 4. It is observed that the convergence of displacement was achieved with a time step of 0.01sec (see Fig. 4), while a smaller time step (0.001sec) was needed to achieve convergence of accelerations (see Fig. 3). Hence, an integration time step equal to 0.001sec was selected to perform nonlinear time history dynamic analyses of the simplified RWFD building model.



(a) (b)

Figure 3. Effect of integration time step on the acceleration at the center of the roof diaphragm for Loma Prieta earthquake record: (a) time history and (b) close up view at the peak response.



(a) (b)

Figure 4. Effect of integration time step on the displacement at the center of the roof diaphragm for Loma Prieta earthquake record: (a) time history and (b) close up view at the peak response.

Second Order (P-Δ) Effects

The influence of second order (P-Δ) effects on the collapse capacity of the RWFD building archetype was also investigated. As noted earlier in this paper, the large displacement analysis option of RUAUMOKO2D was used to account for P-Δ effects, while the small displacement analysis option was activated to omit the influence of second order effects. As shown in Fig. 5, the median collapse does not differ significantly (≈0.06g) for the two analysis cases (with and without the P-Δ effects). However, the implementation of second order effects into the modeling of RWFD buildings is necessary because it influences their collapse capacity at higher intensities.

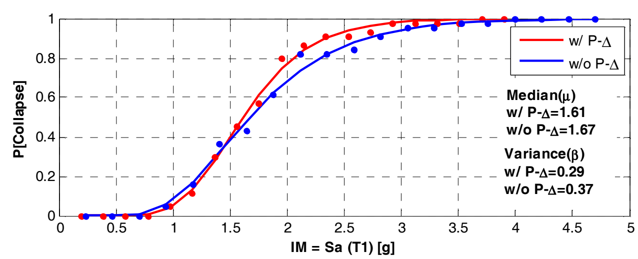
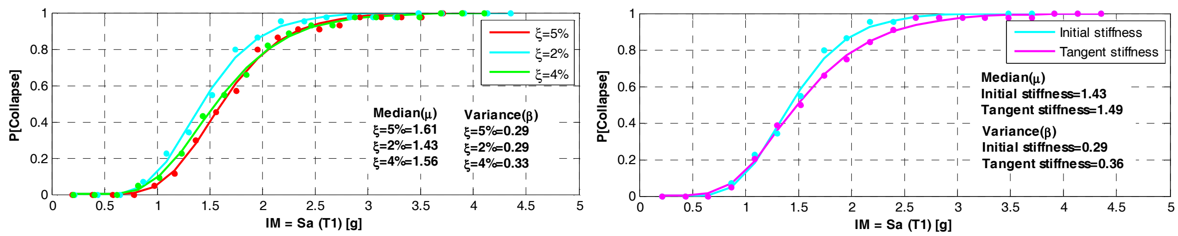


Figure 5. Influence of P-Δ effects on the collapse capacity of the RWFD building archetype.

Inherent Viscous Damping

The effects of the type of inherent viscous damping as well as the target damping values were investigated. Three different target values (2%, 4% and 5% of critical) were selected to evaluate their influence on the collapse capacity of the RWFD building archetype as shown in Fig. 6a. The median collapse of the building was improved for higher values of equivalent viscous damping, while a damping ratio of 2% of critical was detrimental to the building's collapse performance. A lower damping value is more realistic for RWFD buildings that can be characterized as "naked" since they incorporate fewer non-structural components. Therefore, the 2% of critical damping assumption was a representative value for the simplified modeling of RWFD buildings. Note that the analyses for evaluating the target damping values were conducted accounting for initial stiffness equivalent viscous damping. Furthermore, the collapse capacity of the RWFD archetype was evaluated for both initial stiffness and tangent stiffness proportional damping as presented in Fig. 6b. The target damping value of 2% of critical was considered in this study. The median collapse of the building archetype did not differ significantly for the two cases, while improved performance was observed for the tangent stiffness proportional damping case at higher intensities. Considering that the initial stiffness case is a conservative and numerically robust assumption compared to the tangent stiffness damping, the initial stiffness damping was considered an appropriate modeling assumption.

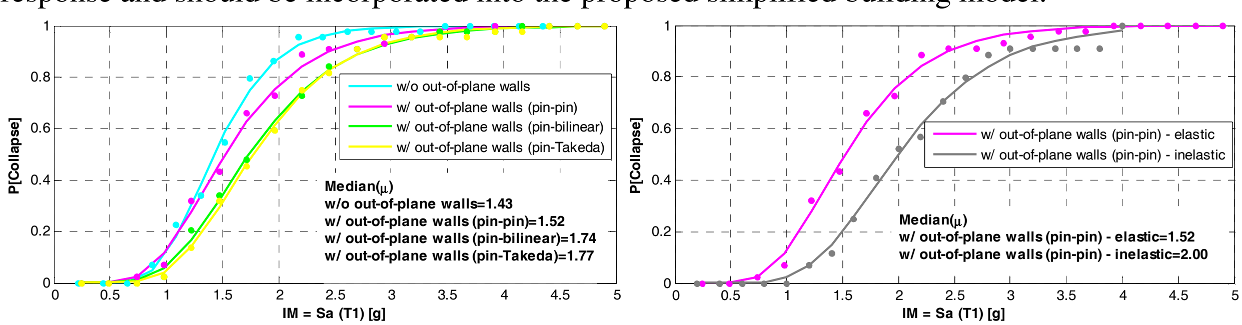


(a) (b)

Figure 6. Influence of: (a) target damping values and (b) damping type on the collapse capacity of the RWFD building archetype.

Out-of-Plane Wall Panels

Collapse of the out-of-plane loaded wall panels of RWFD buildings has been observed as a main failure mode in past earthquakes. Therefore, their effect on the collapse capacity of the RWFD building archetype was investigated herein. The effect of modeling the out-of-plane wall panels as part of the 2D simplified building model was examined along with different base fixity conditions of these walls. Three different fixity cases were studied including: (i) pin (top)-pin (bottom), (ii) pin (top)-bilinear hysteresis (bottom) and (iii) pin (top)-modified Takeda hysteresis (bottom). The properties of both bilinear and modified Takeda hysteretic models were obtained based on the corresponding moment-curvature relationships of the out-of-plane walls. The moment-curvature relationships were generated from the reinforcement design details of the wall panels. Comparing the fragility curves of Fig. 7a for the analysis cases with and without the out-of-plane walls and the various wall fixity conditions, it is observed that the collapse capacity of the RWFD archetype improved when the walls were accounted into the modeling. Furthermore, as shown in Fig. 7a, the median collapse intensity of the RWFD building incorporating a modified Takeda base fixity hysteresis was not significantly improved compared to the bilinear base fixity case. However, it appears that the pin-pin fixity case assumption conservatively represents the collapse capacity of RWFD buildings when out-of-plane walls are incorporated into the modeling approach. Moreover, the inelastic response of the out-of-plane wall panels was investigated by implementing a modified Takeda [20] moment-curvature hysteresis at each sub-element end of these panels. Based on the analyses, the wall panels were found to yield and that is also verified by their fragility curve presented in Fig. 7b. According to the results of this study, it was concluded that the modeling of inelastic out-of-plane wall panels affects the building's response and should be incorporated into the proposed simplified building model.



(a) (b)

Figure 7. Influence of : (a) out-of-plane wall panels' base fixity conditions and (b) the inelastic response of the out-of-plane wall panels on the collapse capacity of the RWFD building archetype.

Direction of Shaking

The direction of shaking was the last parameter investigated in this study. The RWFD building archetype incorporating “hybrid” flexible diaphragm was subjected to ground excitation along its long and short direction, respectively. As shown in Fig. 8, the collapse capacity of the building archetype was considerably improved when excited along the long direction, with an increase of the median collapse capacity of approximately 50%. One influencing factor in this result was that the diaphragm design necessary for the short direction yielded excessive capacity in the long direction.

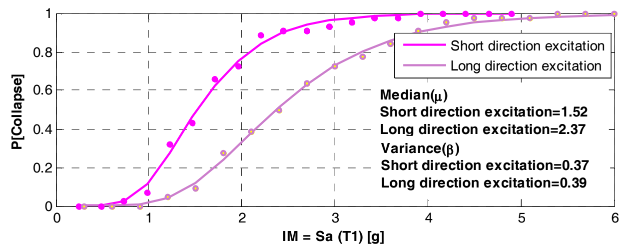


Figure 8. Influence of direction of shaking on the collapse capacity of the RWFD building archetype.

Conclusions

This paper introduced an efficient numerical framework for nonlinear seismic analysis of Rigid Wall-Flexible Diaphragm (RWFD) buildings. This numerical tool is based on a three step sub-structuring approach including the hysteretic response database for diaphragm connectors, the two dimensional inelastic diaphragm model incorporating hysteretic connector response, and the two dimensional building model incorporating hysteretic diaphragm model response. The numerical model of the flexible roof diaphragm as well as the two dimensional simplified building model demonstrated very good correlation with experimental and analytical studies available in the literature. Furthermore, a sensitivity study was conducted and presented in this paper, on the effects of different modeling assumptions on the collapse performance of a typical RWFD building. This study showed that the assumptions considered in the simplified building model (step 3 of sub structuring approach) were realistically representing the seismic response of typical RWFD buildings. Therefore, the proposed numerical framework can be used as an efficient tool for the collapse assessment of RWFD buildings. This numerical tool is detailed enough to capture the nonlinear response of RWFD buildings, but simplified enough to efficiently conduct a large number of nonlinear time-history dynamic analyses in a timely manner and for designers to better investigate new and existing RWFD buildings in a more accessible computer model.

Acknowledgments

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