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Accuracy of Analytical Model in a Performance Based Seismic Design of a 35 Storey RC Building

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ABSTRACT

Performance Based Seismic Design (PBSD) is a design concept that is currently being applied in seismic design of several buildings and bridges in the U.S. and in Canada. For typical reinforced concrete highrise buildings, the design objective is to achieve collapse prevention at the Maximum Considered Earthquake (MCE) intensity and this requires reliable, accurate predictions of structural behavior given large inelastic deformations. For modeling buildings with reinforced concrete shear walls, the current state of practice is to use fiber-section elements which model the interaction of a wall's axial and flexural forces and provide reliable predictions of drift and lateral strength.

However, recent studies have shown that some fiber section models produce inaccurate predictions of drift capacity if the wall element experiences softening prior to reaching flexural failure. For those cases, it is recommended to apply regularization to the fiber section's material models.

For this study a nonlinear model of a 35 storey RC coupled shear wall system is developed using the commercial software Perform-3D and using fiber section wall elements to model RC shear walls. This model is subjected to time history analyses for ground motions spectrally matched to the MCE design level earthquake.

The accuracy of this model's predictions is evaluated by running a sensitivity study focusing on wall discretization and its effects on predicted maximum strains, softening and localization. This study also presents comparison of analysis results using fiber sections with and without regularized material models in order to determine ways to reduce localization and improve accuracy.

INTRODUCTION

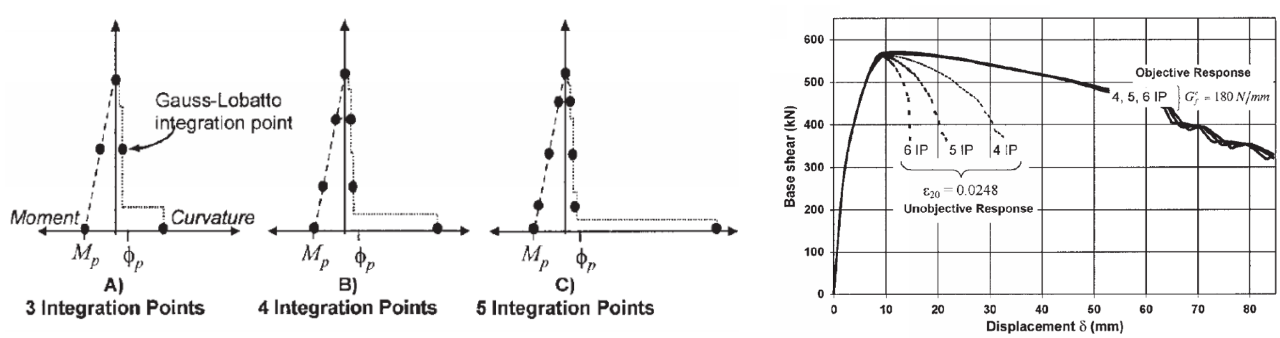
As part of a performance based seismic design of a tall reinforced concrete building, a nonlinear structural model is developed to evaluate its seismic performance at the design MCE level earthquake. This structural model must follow current state-of-practice techniques to ensure its dependability. It is good practice to evaluate modeling assumptions and how they affect the structural analysis predictions.

One aspect of structural modeling that presents limited recommendations is how to discretize a structural wall along the height of the building. The engineer can choose to discretize it using one element per storey; or two or more elements per storey -particularly if the inter-storey height to wall length aspect ratio, h_i/L, is greater than a value of 2. This choice in discretization affects the number of integration points in a fiber-section force-based wall element. Recent research studies [1-4] have reported that using a higher number of integration points can lead to localization effects which affects the accuracy of the predictions in nonlinear deformation capacity of structural elements.

The motivation for this study is to determine if tall building models can develop localization due to the engineer's choice in number of wall elements per storey and if regularization can be used to correct this behavior.

LITERATURE REVIEW

A sensitivity study by Coleman and Spacone [1] observed the effects of varying the number of integration points in frame models. They observed that, for frame models with a softening behavior (i.e. frame specimens that exhibited compression failures), the inelastic behavior would become localized at one critical integration point when a high number of integrations points was used (see Figure 1a). This effect is known as "localization" because all inelastic behavior concentrates in one integration point. The authors also noted that no localization effects occurred on frame elements with hardening sections (i.e. specimens that experience strain hardening prior to failure). The explanation for this is that the integration point under highest moment demand has high stiffness after yielding and that enables the global system to further increase in moment demands, which in turn enables the inelastic curvatures to spread to adjacent integration points. In contrast, a frame element with softening behavior will cease to gain strength for strains greater than peak strain, $\boldsymbol{\varepsilon_0}$, which causes the forces in the global system to drop or plateau and inelastic behavior is unable to spread.
The authors then studied how adjusting the material stress-strain assigned to the model's integration points could reduce localization and mesh dependency in frame elements and proposed regularization equations for both concrete and steel stress-strain material curves [1]. The regularization method is used for reducing a numerical model's dependency on the number of integration points. The method consists in modifying the material stress-strain curve as a function of the integration length assigned to the critical integration point and the energy dissipation capacity of the material between yielding and its failure strain.



a) **b)**

Figure 1. Localization of Inelastic Models with Varying Number of Integration Points (Coleman and Spacone, 2001), a) curvature profiles b) pushover predictions using unregularized and regularized material models

Studies by Pugh [2] and Welt [3] compared laboratory test results of 21 RC wall specimens to analytical results from single forced-based nonlinear beam-column models using the analysis software OpenSees [5]. These studies found that for walls with softening behavior, regularization eliminated the model's sensitivity to the number of integration points. However, regularization in walls with hardening behavior worsened drift capacity predictions.

MODEL DESCRIPTION

The building prototype consists of a 35-story reinforced concrete residential tower with three levels of parking below grade. The tower has a height of 359 ft. above grade and a rectangular shape with maximum dimensions of 180 ft. (East to West direction), and 85 ft. (North to South direction).

The gravity load-carrying system of the tower consists of reinforced concrete post-tensioned flat slabs and a series of rectangular reinforced concrete columns. The below-grade structure, including grade level, has reinforced concrete flat slabs supported by perimeter basement walls and concrete columns. The entire structure is supported by a continuous mat foundation.

The seismic load-carrying structure consists a central core of special reinforced concrete shear walls. It was designed to the prescriptive requirements of the 2013 California Building Code (CBC) [6] except for the structural height limit defined for this system in Table 12.2.-1 of the ASCE 7-16 [7].

The shear wall elements were modeled using fiber sections for flexural behavior with one integration point per element, and elastic elements with cracked section properties for shear and out-of-plane stiffness. Coupling beams were modeled using shear hinge elements for nonlinear shear behavior with other properties modeled with elastic elements. For modeling of the gravity system, columns consisted of fiber section frame elements connected to beam elements with moment-hinges which represented the moment connection to the floor slabs. Basement walls were modeled using elastic elements for flexural and shear behavior. Floor levels above L4 were modeled assuming rigid diaphragm conditions, with floor mass concentrated at the center of mass, while lower levels (L1-L4, mezzanine and basement levels) were modeled using shell elements with distributed mass. The analytical model was assembled in the commercial analysis software Perform-3D [8]. An illustration of the plan view of the structural model is shown in Figure 2.

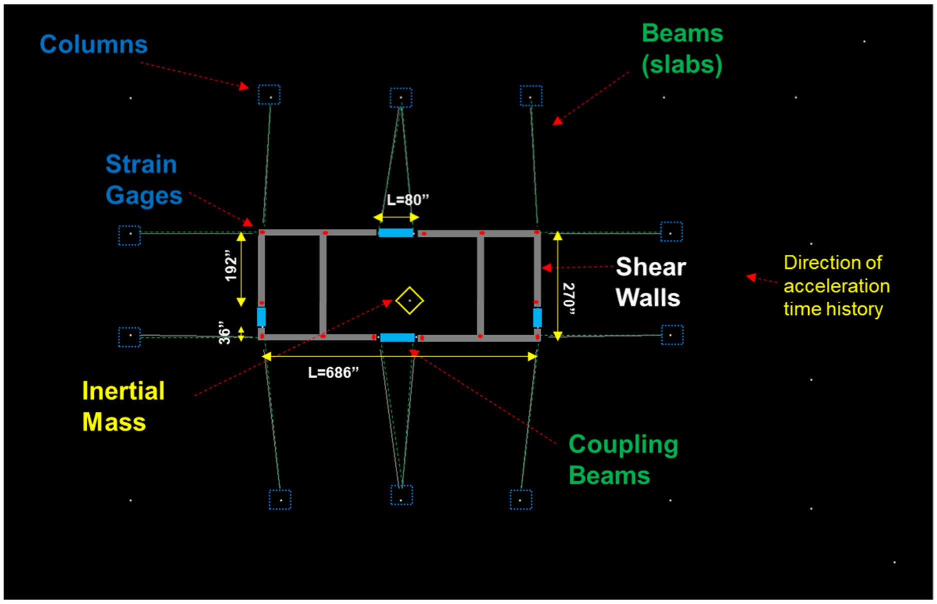


Figure 2. Structural plan view of reinforced concrete shear wall coupled wall system.

ANALYSIS OF SENSITIVITY

The sensitivity study compares two structural models of the 35 storey structure, illustrated in Figure 3, which differ only in the number of elements used between levels P4 to L7 for the four RC shear walls along the cantilever direction of the building: Model 1 uses one element per storey and Model 2 uses two elements per storey. The resulting wall aspect ratios, h_i/L, are 0.43 (h_i =117in and L =270in) and 0.22 (h_i =58.5in and L =270in), respectively. In each model, strain gage elements were added to the sides of every wall element.

The two models are subjected to MCE intensity level seismic demands using 11 spectrally matched time histories. The acceleration time histories are applied in one direction only, along the structural core's uncoupled direction, in order to ensure strain demands correspond strictly to flexural demands.

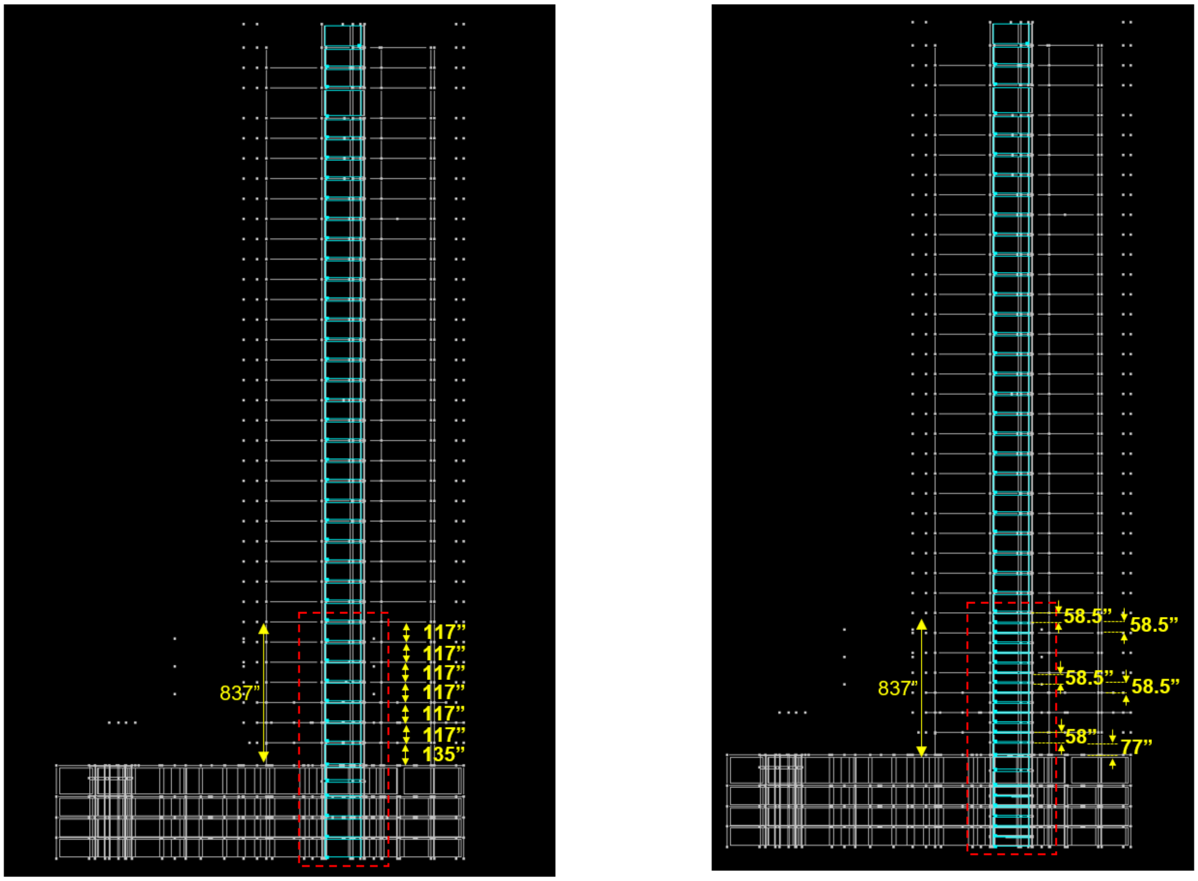
Strain Demands

The average compression strain demand profiles obtained from the two models, shown in Figure 4a, present similar values and no localization of strains. The maximum strain, ε_{cx}, is observed at L1 with a value of approximately 0.001 in/in. These findings suggest that for this structure, compression strain demands are not sensitive to the number of wall elements per storey and that regularization of the concrete material stress strain curves would not be necessary.

This study found significant differences between the tensile strain demand profiles of the two models, seen in Figure 4b, where strain demands in Model 2 are greater than for Model 1. The highest difference is found at grade level, where tensile strain for Model 2, e_{sx2} is equal to 0.0038 in/in while for Model 1 shows a corresponding tensile strain, e_{sx1}, equal to 0.0029 in/in. In addition, the comparison shows that localization effects occur for Model 2, between levels L1 and L7, with levels above the mezzanine exhibiting strain reversals.

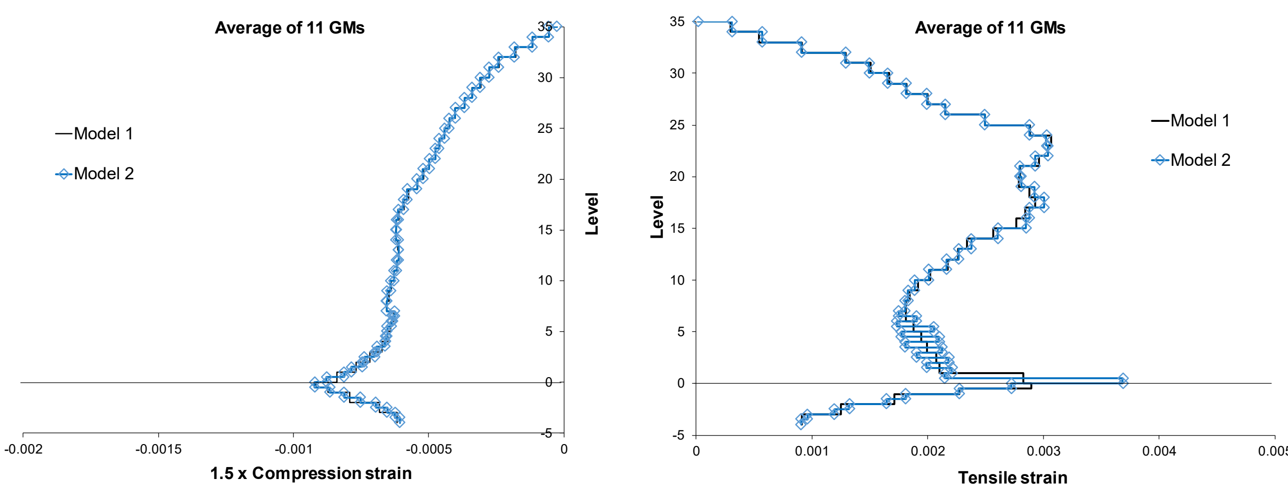
Localization occurs for tensile strains and not compression strains because of the magnitude of the strain demands developed during the structural response, as shown in Figure 5. Concrete strain demands are within the elastic range of the confined concrete stress-strain backbone curve and therefore do not result in localization. In contrast, tensile strain demands are past the yield point in the stress-strain backbone, as seen in Figure 5b, and this leads to localization.

The findings from this comparison showed that Model 2 with wall elements with aspect ratio, $h_i/L=0.22$, showed localization effects while Model 1 with aspect ratio, $h_i/L=0.43$, did not show localization effects. It is necessary to evaluate if regularization can reduce localization effects in Model 2 while maintaining the wall discretization aspect ratio at $h_i/L=0.22$.



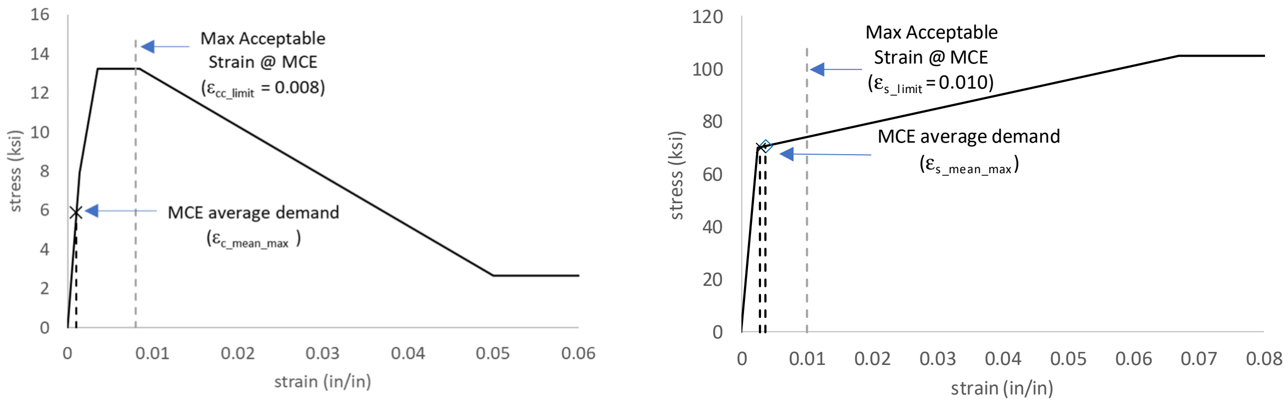
a) b)

Figure 3. Models for sensitivity study of RC shear wall discretization: a) Model 1, b) Model 2.



a) b)

Figure 4. Average demand profiles for corner strain gage: a) Compression strains, b) Tensile strains.



a) b)

Figure 5. Maximum strain demands vs stress strain backbone curves: a) Confined concrete, b) Reinforcing steel.

REGULARIZATION

Following the recommendations in [1], localization of strains can be reduced by modifying the material backbone curves using regularization. This technique intends to make the two models have an equal value of post peak energy dissipation, G, at the plastic hinge region. In this study, we modified the material models in the wall elements in Model 2, that had lengths, L_2, equal to 58.5in, to result in energy values equal to those wall elements in Model 1, with lengths, L_1, equal to 117in. Although localization was observed only for reinforcing steel material and not concrete, regularization was applied on both material stress strain curves for consistency.

Regularization of concrete material curve

The equation for post-peak energy dissipation for concrete, G_c, is shown in Equation (1) below:

$$\frac{G_c}{L} = 0.6f'_c * \left([\varepsilon_{20} - \varepsilon_0] + \frac{0.8f'_c}{E_c}\right) \quad (1)$$

Where:

G_c: concrete post peak energy dissipation

L: length of integration of wall element

f'_c : concrete peak strength

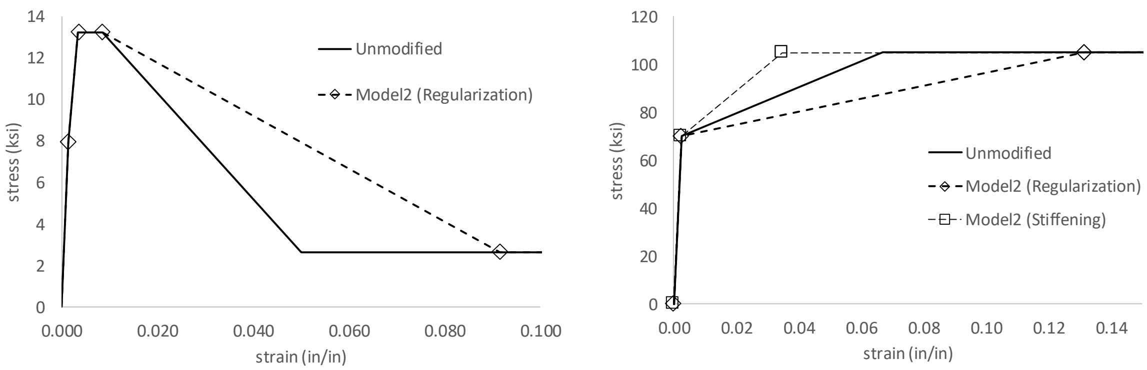
E_c : concrete elastic modulus

ε_0: peak concrete strain, at 100% of concrete strength, f'_c

ε_{20}: post-peak concrete strain, at 20% of concrete strength, f'_c .

The modified concrete post-peak segment for Model 2 is shown in Eq. (2) derived by setting G_{c2} equal to G_{c1} and neglecting the term $\frac{0.8f'_c}{E_c}$. The modified concrete stress strain backbone curve using regularization is shown in Figure 6a.

$$[\varepsilon_{20} - \varepsilon_0]_2 = [\varepsilon_{20} - \varepsilon_0]_1 * \frac{L_1}{L_2} \quad (2)$$



a) b)

Figure 6. Modified stress strain curves using Regularization: a) Confined concrete stress strain, b) Reinforcing Steel

Regularization of reinforcing steel material curve

Similar to the concrete material curve, regularization of the reinforcing steel's stress strain curve is based on the post-peak energy dissipation, G_s, which can be calculated using Eq. (3). The modified post-peak segment for Model 2 shown in Eq. (4) is derived by setting G_{s2} equal to G_{s1} and neglecting the term $\frac{1}{2}\frac{(f_u)^2}{E_s}$

$$\frac{G_s}{L} = \frac{1}{2}\left(f_u + f_y\right) * \left[\varepsilon_{su} - \varepsilon_{sy}\right] - \frac{1}{2}\frac{(f_u)^2}{E_s} \tag{3}$$

$$\left[\varepsilon_{su} - \varepsilon_{sy}\right]_2 = \left[\varepsilon_{su} - \varepsilon_{sy}\right]_1 * \frac{L_1}{L_2} \tag{4}$$

Where:

G_s: reinforcing steel post peak energy dissipation

f_y : reinforcing steel yield strength

f_u : reinforcing steel ultimate strength

E_s : steel elastic modulus

ε_{sy}: steel strain at yield strength.

ε_{su}: steel strain at ultimate strength.

The regularization of reinforcing steel described above results in a softening of the post-yield slope in the material's backbone curve, as shown in Figure 6b. To further evaluate solutions for localization, the authors propose a second approach, referred herein as "stiffening", consisting of increasing the post-yield slope of the material's backbone curve. Stiffening is calculated using Eq. (5) as shown below:

$$\left[\varepsilon_{su} - \varepsilon_{sy}\right]_2 = \left[\varepsilon_{su} - \varepsilon_{sy}\right]_1 * \frac{L_2}{L_1} \tag{5}$$

Analysis Results of Modified Models

Nonlinear analyses were run using modified versions of model 2, using regularization and stiffening methods. Both methods result in similar strain predictions than those previously obtained for the unmodified model 2, as shown in Figure 7, with only minor differences observed at L1. In general, both methods, regularization and stiffening, were not effective in reducing localization effects for tension strains. This observation has been observed in other studies when modeling frame structures under low axial loads [1].

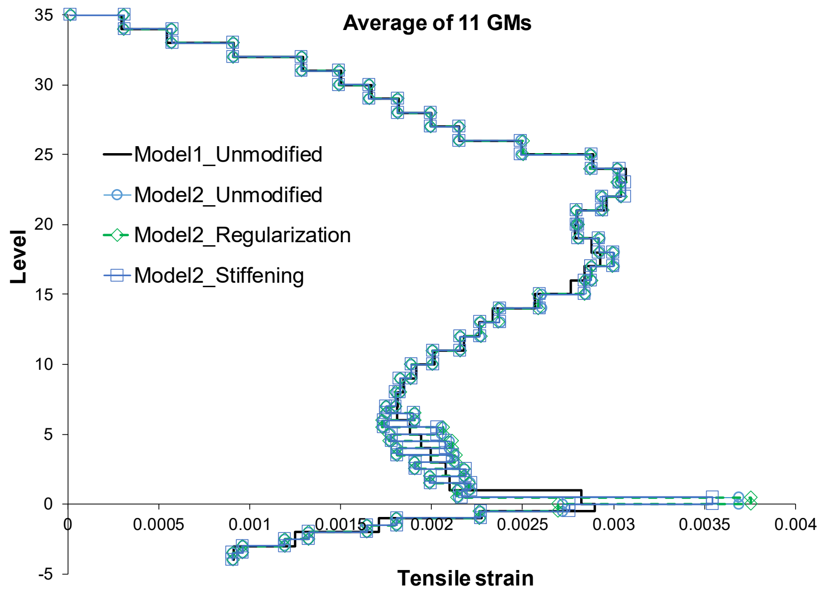


Figure 7. Average tensile strain demand profiles of modified and unmodified wall models.

CONCLUSIONS

This paper aimed to evaluate how wall discretization affects the accuracy of average seismic demands obtained from nonlinear analysis of force-based fiber-section wall elements. This study compared wall strain demand results of a 35 storey RC coupled shear wall building modelled in Perform-3D. Compression strains at the MCE level were not sensitive to discretization of wall elements and did not develop localization because average strain demands were lower than the peak concrete strain. In contrast, tensile strains at the MCE level were affected by discretization because demands were greater than the reinforcing steel's yield strain which resulted in localization of strains in wall elements. Regularization of the reinforcing steel material's stress strain curve was ineffective in reducing localization of strain demands. To avoid localization, inter-storey height to length aspect ratio of wall elements should be equal or greater than 0.4.

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