PROPOSED UPDATE OF NONLINEAR MODELS FOR REINFORCED MASONRY SHEAR WALLS IN ASCE 41

Jianyu Cheng¹ and P. Benson Shing²

ABSTRACT

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Introduction

Reinforced Masonry (RM) is commonly used for low- to medium-rise residential, commercial, and school buildings. Shear walls are the main seismic load resisting elements in reinforced masonry structures. Under extreme seismic load conditions, the failure behavior of these walls can be governed by flexure, diagonal shear cracking, or shear sliding. A flexure-dominated mechanism is typically characterized by flexural cracking, the yielding, buckling and fracture of vertical reinforcement, masonry toe crushing, and the failure of lap splices if they are present in a plastic-hinge region. A shear-dominated wall shows more brittle behavior governed by the opening of diagonal cracks. The opening of a diagonal crack may be accompanied by the tensile fracture or anchorage failure of the horizontal reinforcement. Shear sliding can occur along the base of a wall or along a bed joint a few courses away from the base.

Much research has been conducted to understand and predict the nonlinear behavior of reinforced masonry structures and components. Ahmadi [1], Sherman [2], Kapoi [3], Shedid et al. [4], Voon and Ingham [5], and Shing et al. [6] have conducted quasi-static cyclic tests on flexure-dominated and shear-dominated wall components. Stavridis et al. [7] have conducted a shake table test to study the system performance of a three-story fully grouted RM building. Mavros et al. [8] have used shell elements with a smeared-crack constitutive model for masonry, and cohesive crack interface elements to model the seismic behavior of RM walls and buildings. However, finite element modeling with shell and interface elements requires significant computational efforts. For

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engineering practice, a simplified modeling approach using frame elements is more desirable.

ASCE 41-13 [9], *Seismic Evaluation and Retrofit of Existing Buildings*, has become the main standard used by engineers to evaluate the seismic performance of existing buildings. It provides backbone curves for the nonlinear static and dynamic analyses of structural components and systems, including RM shear walls. However, the backbone curves adopted for RM in this standard were based on limited experimental data, and have not been extensively verified with test data that are currently available.

This paper presents the results of a study intended to improve the nonlinear analysis of RM shear walls, as part of the ATC-114 [10] efforts. In this study, both flexure-dominated and shear-dominated RM wall components have been considered. The modeling approach proposed and the results obtained can be used to improve ASCE 41. The proposed analysis method has been evaluated with experimental data.

**Flexure-Dominated RM Shear Walls**

**Modeling Approach**

This section describes the modeling approach recommended for flexure-dominated shear wall components. A beam-column element with a predefined plastic-hinge zone at one or both ends is selected to model the axial and flexural behavior of a wall. However, the general methodology presented here can also be applied with other types of beam elements. The main advantage of this element is that it has the computational efficiency of a concentrated-plasticity beam element as well as the generality of a distributed-plasticity element. The element requires that the effective plastic-hinge length be pre-defined to capture the localization of plastic deformation in a wall. The moment-curvature relation for a wall section, including the influence of the axial load, is modeled with a fiber-section approach, in which uniaxial material laws are used to represent the behaviors of masonry and flexural reinforcement. In this study, the effective plastic-hinge length is assumed to be 20% of the effective height of the wall, as suggested in a prior study [11]. The shear behavior of a wall component is modeled with a linearly elastic spring in series of the beam-column element. Appropriate material models are needed to represent the failure behaviors of masonry and reinforcing steel. These models and their calibration are described below.

**Material Model for Masonry**

Figure 1 shows the stress-strain law selected for masonry, which is based on the concrete model developed by Kent and Park [12]. The tensile strength of masonry is ignored. For the compressive behavior, the peak stress and the corresponding strain can be determined from masonry prism tests, and the post-peak residual strength is assumed to be 20% of the peak strength. The post-peak compressive behavior of the model is controlled by the strain parameter $\varepsilon_p$, which is the strain at which the residual strength is first reached. It cannot be directly determined from prism tests because the strain localization in a prism under compression could be different from that in a wall subjected to bending. Hence, the post-peak slope of the curve is calibrated with the wall test data of Ahmadi [1], Sherman [2], Kapoi [3], Shedid et al. [4], and Shing et al. [6], using the aforementioned beam element with the effective plastic-hinge length assumed to be 20% of the
effective wall height. The calibrated model, as shown in Fig. 1, is referred to as the baseline model. Should the effective plastic-hinge length assumed in the element differ from 20% of the wall height, the post-peak slope (i.e., the value of $\varepsilon_r$) should be so changed to ensure the objectivity of the fracture energy and the numerical result, as suggested by Coleman and Spacone [13] based on the original argument of Bazant and Oh [14].

![Normalized baseline stress-strain relation for masonry](image)

**Material Model for Steel**

Under severe seismic loading, the vertical reinforcing bars in a wall may undergo yielding, buckling, and fracture caused by low-cycle fatigue. A phenomenological material law, as shown in Fig. 2(a), is selected to represent the behavior of vertical reinforcing bars, accounting for bar buckling and fracture in an approximate manner. Under a monotonically increasing load, the tensile strength of steel is assumed to be 1.5 times the yield strength, which is typical for Grade 60 steel.

Bar buckling is represented by a sudden drop of the compressive stress in the stress-strain law when the compressive strain reaches a critical value $\varepsilon_b$, which is the strain at which the compressive strength of masonry drops to 40% of the peak stress, as shown in Fig. 1, and thereby, severe crushing occurs. This is based on the assumption that the strain in the bar is the same as that in the adjacent masonry. After buckling, the stress in the bar drops linearly to 10% of the yield strength $f_y$. The strain $\varepsilon_{10}$, at which the residual compressive strength is first reached, is assumed to be -0.01 to represent a rapid strength drop due to buckling.

The fracture strain of a vertical bar depends on the loading history. As a wall is subjected to severe cyclic loading, toe crushing will occur, and the exposed vertical bars may undergo repeated buckling and straightening, which will induce severe bending stresses in the bars. Hence, bar fracture will occur at a lower strain level due to low-cycle fatigue. Hence, for a bar in tension, two types of loading histories are considered. One is monotonically increasing tensile loading. For this case, it is assumed that the peak tensile stress is reached at a strain ($\varepsilon_{ps}$) equal to 0.10, which is typically observed in uniaxial tension tests. After this, bar fracture initiates and the tensile stress drops linearly as the strain increases. The strain ($\varepsilon_0$) at which the tensile stress reaches zero is assumed to be 0.15. The other case is cyclic loading, for which it is assumed that the peak tensile stress is reached at a reduced strain ($\varepsilon_{ps}$) between 0.03 and 0.072 to account for the low-cycle fatigue. The strain ($\varepsilon_0$) at which the tensile stress reaches zero is so determined that the stress drops...
at the same rate as in the monotonic loading case. Walls with a higher amount of vertical reinforcement and a higher axial compressive load are expected to have a lower fracture strain as masonry spalling tends to occur earlier in these walls. Figure 2(b) shows the relation between the strain at the peak stress, $\varepsilon_{ps}$, and these quantities, which are defined as

$$\alpha = \frac{f_y}{f'_m} \rho_v$$

$$\beta = \frac{P}{f'_m A_n}$$

in which $\rho_v$ is the ratio of the vertical steel area with respect to the net cross-sectional area, $A_n$, of the wall, and $P$ is the axial compressive force. The material law shown in Fig. 2 is referred to as the baseline model for steel. It has been calibrated with wall test data. Similar to the masonry material law, this baseline steel model needs to be modified when the effective plastic hinge-length assumed in the element differs from 20% of the effective wall height [10].

Figure 2. (a) Normalized baseline stress-strain relation for steel considering bar buckling and fracture; (b) Tensile strain at bar fracture as a function of steel quantity and axial compressive load for cyclic analysis

**Consideration of Shear Deformation**

For a flexure-dominated wall, shear deformation can be accounted for by using an elastic shear spring connected in series with the lateral degree of freedom of the beam-column model. The elastic shear stiffness can be calculated with Eq. 3.

$$k_v = \frac{A_v G_m}{h}$$

in which $A_v$ is the effective shear area of the wall section, $G_m$ is the shear modulus of masonry, and $h$ is the height of the wall.

However, flexural and shear cracks may develop before the peak shear resistance of a wall has been reached, which can lead to a significant reduction of the shear stiffness. Test data of Shing et al. [15] have shown that the effective shear stiffness of a reinforced masonry cantilever wall can
be reduced to 50% of the theoretical value given by the formula above when the applied lateral force reaches 50% of the shear capacity, and can be as low as 20% of the theoretical value when major shear cracks develop. Based on this observation as well as data from recent tests by Ahmadi [1], Sherman [2], and Kapoi [3], a reduction factor of 0.35 is recommended to be applied to the theoretical shear stiffness to account for wall cracking.

Validation of the Proposed Model

Data from single wall tests performed by Ahmadi [1], Sherman [2], Kapoi [3], Shedid et al. [4], and Shing et al. [6] have been used to validate the modeling method proposed here. A total of 21 cantilever walls have been considered. Eleven of these walls had the vertical reinforcement lap-spliced at the base. These specimens have an aspect ratio ranging from 0.78 to 4.5, and the vertical reinforcement ratio ranging from 0.16% to 1.31%. The axial compression ratio, which is defined in Eq. 2, ranges from 0.0 to 0.125.

The software framework OpenSEES [16] [17] has been selected to conduct cyclic analysis on selected wall specimens. Each wall is represented by a single beam-column element, which is the Beam-with-Hinge model (a beam-column element with a predefined plastic-hinge zone) proposed by Scott and Fenves [14]. The material models for masonry and steel are based on the baseline models presented previously. Masonry is modeled by the Kent-Park model (Concrete01 in OpenSEES), while the vertical reinforcement is modeled by the Hysteretic model in OpenSEES.

Figure 3 shows the results for walls UT-PBS-03 and UT-PBS-04 tested by Ahmadi [1]. The numerical results show a good match with the experimental data. The peak strength, strength degradation, and hysteretic behavior observed in the tests are captured.

Backbone Force-Displacement Curves for Flexure-Dominated Walls

Figure 4 shows an idealized backbone curve proposed to represent the load-displacement relation for flexure-dominated walls. It can be calibrated to represent the first-cycle envelopes for walls subjected to cyclic loading or monotonic pushover curves. The curve is defined in terms of five parameters: the effective initial stiffness \( k \), the expected maximum lateral load resistance \( Q_{max} \),
the displacement \((\Delta_m)\) at which the maximum resistance develops, the displacement \((\Delta_{75})\) at which the post-peak resistance drops to 75\% of \(Q_{max}\), and the capping displacement \((\Delta_c)\) after which the lateral resistance of the wall can be ignored. The capping displacement is taken as the point at which the resistance drops to 50\% of \(Q_{max}\). Furthermore, \(\Delta_c\) is not allowed to exceed 4\%, which is the maximum observed in experimental studies. The construction of the backbone curve using the moment-curvature relation of a wall section is presented below for a cantilever wall. Since the effect of cyclic loading has been accounted for in the steel material model, only monotonic moment-curvature analysis is needed for walls subjected to cyclic loading.

![Backbone load-displacement curve for flexure-dominated reinforced masonry walls](image)

Figure 4. Backbone load-displacement curve for flexure-dominated reinforced masonry walls

Similar to the recommendation in ASCE 41 [9], the effective initial stiffness \((k)\) of a cantilever wall can be calculated with Eq. 4.

\[
k = \frac{1}{\frac{h^3}{3E_mI_e} + \frac{h}{0.35G_mA_v}}
\]

in which \(h\) is the wall height, \(A_v\) is the effective shear area of the wall section, and \(E_m\) and \(G_m\) are the Young’s modulus and shear modulus of the masonry, respectively. To account for the cracking of masonry, the effective moment of inertia \((I_e)\) is assumed to be 15\% of that of an uncracked section, and a reduction factor of 0.35 is applied to the shear stiffness.

A fiber-section model can be used to generate a moment-curvature curve. The lateral shear resistance \((Q_{max})\) can be calculated as \(M_{max}/h\), in which \(M_{max}\) is the peak moment from the moment-curvature curve. Displacement \(\Delta_m\) consists of a flexural component \((\Delta_{fm})\) and a shear component \((\Delta_{vm})\). These two components can be calculated with Eqs. 5 and 6, respectively.

\[
\Delta_{fm} = \frac{M_{max}h^3}{EmI_e} + \left(\phi_m - \frac{M_{max}}{EmI_e}\right)L_p(h - \frac{L_p}{2})
\]

\[
\Delta_{vm} = Q_{max} \frac{h}{0.20A_vG_m}
\]

in which \(L_p\) is the effective plastic hinge length and \(\phi_m\) is the curvature at which maximum moment develops. Equation 6 has a reduction factor of 0.20 applied to the shear stiffness to account for the condition that severe cracking may have occurred at the peak load. Similarly, \(\Delta_{75}\) and \(\Delta_c\) can be calculated with the moment, lateral resistance, and curvature values at corresponding load
Nondimensionalized Moment-Curvature Relation

To reduce the number of independent variables required to construct the backbone curve, one can express the moment-curvature relation in a dimensionless form by assuming that the vertical steel is uniformly distributed along the wall length between the two extreme vertical bars. For a fully grouted symmetric rectangular wall section, the axial load and the bending moment about the centroidal axis can be expressed in terms of the nondimensionalized formulas shown in Eqs. 7 and 8.

\[
\beta = -\int_0^{\frac{1}{2}} \sigma_m'(\epsilon(y')) dy' - \frac{1}{a} \int_0^a \sigma_s'(\epsilon(y')) dy' \quad (7)
\]

\[
M' = -\int_0^{\frac{1}{2}} \sigma_m'(\epsilon(y'))(y' - \frac{1}{2}) dy' - \frac{1}{a} \int_0^a \sigma_s'(\epsilon(y'))(y' - \frac{1}{2}) dy' \quad (8)
\]

in which \( \alpha \) is the vertical reinforcing index, \( \beta \) is the axial compressive load ratio, as defined in Eqs. 1 and 2, respectively. \( M' = \frac{M}{f_m' A_n l_w} \), and \( y' = y/l_w \), with \( y \) being the distance from the centroid axis of the wall section, \( l_w \) the wall length, and \( A_n \) the net area of the wall section. The bending strain \( \epsilon(y') \) is proportional to \( \phi l_w \) and a linear function of \( y' \) based on the plane section remaining plane assumption, and \( \sigma_m' \) and \( \sigma_s' \) are the stresses in the masonry and vertical steel normalized by the masonry compressive strength \( f_m' \) and steel yield stress \( f_y \), respectively. The stresses can be calculated with the baseline material properties shown in Figs. 1 and 2. The variable \( \alpha \) is defined as the ratio of the distance between the two extreme vertical bars to the wall length. Detailed derivation of the above equations can be found in [10].

As the impact of the dimensionless variable \( \alpha \) on the moment-curvature relation is not significant, the number of dimensionless variables governing the moment-curvature relation of a fully grouted rectangular wall section can be reduced to two, which are \( \alpha \) and \( \beta \). For the convenience of constructing backbone load-displacement curves, the values of the dimensionless parameters, namely, \( M' \), \( \phi_m l_w \), \( \phi_{75} l_w \), and \( \phi_c l_w \), are calculated for fully grouted rectangular sections using a fiber-section model in OpenSEES. A wide range of \( \alpha \) and \( \beta \) values are considered to represent typical wall sections. Table 1 shows the values for two wall sections that are used to construct backbone curves for walls UT-PBS-03 and UT-PBS-04 tested by Ahmadi [1].

### Table 1. Nondimensionalized backbone moment-curvature curve values for fully grouted rectangular wall sections subjected to cyclic loading

<table>
<thead>
<tr>
<th>Reinforcement ( \alpha = (f_y/f_m')\rho_v )</th>
<th>Axial Load ( \beta = P/(f_m' A_n) )</th>
<th>( \phi_m l_w )</th>
<th>( \phi_{75} l_w )</th>
<th>( \phi_c l_w )</th>
<th>( M/(f_m' A_n l_w) )</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.01</td>
<td>0</td>
<td>0.0782</td>
<td>0.1002</td>
<td>0.1210</td>
<td>0.0060</td>
</tr>
<tr>
<td>0.05</td>
<td>0</td>
<td>0.0681</td>
<td>0.0904</td>
<td>0.1073</td>
<td>0.0285</td>
</tr>
</tbody>
</table>
Comparison with Experimental Data

Figure 5 shows the comparison of the backbone curves constructed by the method proposed here with the test data for walls UT-PBS-03 and UT-PBS-04. The backbone curves are constructed by interpolating the values of the dimensionless parameters shown in Table 1 for \( \alpha \) equal to 0.01 and 0.05. The backbone curves show a good match with the first-cycle envelopes of the test data. In contrast, the backbone curves based on ASCE 41 show overly brittle behavior.

Shear-Dominated RM Shear Walls

Backbone Force-Displacement Curve

For a fully grouted shear-dominated RM shear wall, the strength and ductility are influenced by the aspect ratio of the wall, the amount of vertical and horizontal reinforcement, and the axial compressive load. The influence of the vertical reinforcement on the shear strength and ductility is through the dowel action. As there is no conclusive data on the influence of these parameters on the ductility of a shear-dominated wall, an empirical backbone curve is proposed based on the test data of Ahmadi [1], Voon and Ingham [5], and Shing et al. [6].

Figure 6 shows the proposed lateral load-vs.-lateral drift ratio curve for fully-grouted shear-dominated walls. For a cantilever wall, the effective initial stiffness \((k)\) can be calculated with Eq. 4. For a wall with fixed-fixed end conditions, the flexure stiffness term in the equation should be modified accordingly. The peak shear strength \((Q_{\text{max}})\) of a wall can be calculated with the formula given in TMS 402 [18], considering the contribution of the masonry \((V_{nm})\) and the contribution of the horizontal reinforcement \((V_{ns})\). The residual strength \((Q_{r})\) is assumed to be equal to \(V_{ns}\). The drift ratio at the peak strength and the drift ratio at which the residual strength develops are taken as 0.5% and 1.0%, respectively. The maximum allowable drift ratio is taken as 2.0%, which is the same as that specified in ASCE 41 [9].

Studies by Minaie et al. [19] and Bolhassani [20] show that partially grouted shear-dominated RM walls have lower strength and displacement capacity than fully grouted shear-dominated walls.
Hence, for partially grouted walls, it is recommended that the backbone curve shown in Fig. 6 be modified as follows. The peak strength ($Q_{\text{max}}$) and residual strength ($Q_r$) should be reduced by a factor of 0.75 as suggested in TMS 402 [18]. The drift ratio at the peak strength should be reduced from 0.5% to 0.2%, and that corresponding to the residual strength be reduced from 1% to 0.4%. The maximum allowable drift is to be capped at 0.8% rather than 2%.

![Figure 6. Backbone lateral load – lateral drift ratio curve for fully grouted shear-dominated reinforced masonry walls](image)

**Comparison with Experimental Data**

Test data of Ahmadi [1], Voon and Ingham [5], and Shing et al. [6] on fully grouted walls are used to evaluate the proposed backbone curve. The shear-span ratios, $M/Vl_w$, of the walls range from 0.5 to 2, with most of the walls having a shear-span ratio of 1. Figure 7 shows the comparison of the proposed backbone curve with the first-cycle envelopes of the test data for walls UT-PBS-01 and UT-PBS-02 tested by Ahmadi [1]. The backbone curves match the test data well, while the backbone curves recommended in ASCE 41 show overly brittle behaviors.

![Figure 7. Comparison of proposed backbone curves with experimental data and the backbone curves recommended in ASCE 41 for fully grouted shear-dominated walls](image)

**Conclusions**

In this paper, new lateral load-vs.-lateral displacement backbone curves are proposed for assessing the seismic performance of reinforced masonry (RM) shear walls using a nonlinear static or
dynamic analysis procedure. For flexure-dominated walls, the backbone curves are derived with a rational analysis method using simplified material laws that accounts the nonlinear behavior of the masonry and the buckling and fracture of the vertical reinforcing bars. For fully grouted RM walls with symmetric rectangular sections, the backbone curves can be readily constructed with a set of nondimensionalized values defining the moment-curvature relations of the wall sections. For shear-dominated walls, the backbone curves are based empirically on test data. For both wall types, the proposed backbone curves show marked improvements over those recommended in ASCE 41. The latter show overly brittle behavior. However, further effort is required to develop a model that can be used to predict the shear behavior of RM walls in a rational manner.

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