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Is it feasible to use stabilized earth masonry for earthquake-resistant single-story houses in the U.S.?

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Abstract. Stabilized earth masonry (SEM) made with unfired blocks is emerging as an attractive option for sustainable houses that are built using locally available materials. However, it is unclear whether SEM is suitable for low-rise buildings in seismic regions of the US, and establishing feasibility is essential to guide research and development toward generating guidance for practitioners. Drawing from the authors' experience in SEM prototyping and mechanical characterization, this paper presents an analytical investigation on the feasibility of using SEM for earthquake-resistant dwellings in the US. A structural analysis and design framework is enlisted to examine the case study of a representative single-story house located in New Madrid (MO), and thus in one of the most seismic-prone areas in the US.

Keywords: Earthquake-resistant house, feasibility evaluation, New Madrid seismic zone, stabilized earth masonry, structural analysis.

1 Introduction

Earthen materials offer compelling environmental benefits in construction, in line with the growing preference for minimally processed and low-toxicity building materials [1]. The ability to naturally regulate indoor humidity and temperature has been demonstrated in earth masonry buildings compared to structures built using conventional construction materials [2, 3, 4], also including conventional clay masonry. In addition, by using locally available materials, earth masonry makes it possible to substantially reduce carbon footprint [2, 5]. In the US, the presence of suitable soils for earth masonry makes this building solution attractive and viable for houses in rural and even suburban areas.

Based on a study conducted by the United States Geological Survey (USGS), in the US more than 143 million individuals are at risk of death, injury, or homelessness, should a major earthquake occur near their home [6]. This situation is critical in the Southern and Midwest US and along the East Coast and West Coast where the

population density is particularly high. Studies on earthen structures—including adobe, rammed earth, and cob—have covered solutions to enhance the structural performance of shear walls under in-plane loads [7, 8, 9]. Based on large-scale in-plane load tests on SEM wall specimens, some evidence has been presented on the feasibility of using unreinforced SEM for residential structures in low seismic-hazard zones [10]. However, to the best of the authors' knowledge, no research has been published on the feasibility of using reinforced SEM in zones that pose more severe concerns for seismic hazard. This paper intends to contribute to filling this gap by presenting a case study on feasibility for the most critical seismic-hazard area in US.

Feasibility is evaluated by verifying the in-plane flexure and shear resistance of structural walls for a representative case study based on: (a) a traditional single-story floorplan (two bedrooms, one bathroom) and layout of structural walls (Fig. 1); (b) a nominal thickness of the structural walls of 254 mm; (c) a SEM nominal compressive strength of 3.50 MPa; and (d) New Madrid Seismic Zone (NMSZ) location and the correspondent Seismic Design Category in conformance with ASCE/SEI 7-22 [11].

2 Methodology

This section outlines: (a) the selected case study of a low-rise dwelling located in the NMSZ; (b) the method to determine the design loads; and (c) the method to establish the nominal and design strength of reinforced SEM under seismic loads, for the cases of in-plane flexural and shear failure, following mechanics-based equations [12] together with related empirical factors (*e.g.*, friction coefficient) applicable to earth masonry as specified in the NZS 4297:2020 standard [13]. The provisions specified in ASCE/SEI 7-22 [11] and TMS 402/602-22 [14] were followed to define the nominal and design loads and the reinforcement layout, respectively.

2.1 Case Study

A typical square floorplan of a one-story dwelling was selected for this case study. The floorplan, which was previously used in Matta *et al.* [15], offers two bedrooms and one bathroom, and has a nearly symmetric distribution of structural walls in the two main directions (x and y in Fig. 1). A rigid diaphragm that is anchored into the walls is assumed to ensure sufficient distribution of the horizontal seismic loads. Table 1 summarizes the wall layout (L_i = length of wall i ; x_i and y_i = coordinates of centroid of wall i along x and y direction, respectively). The walls are assumed to have a height $h = 2.50$ m and a thickness $t = 254$ mm. Such thickness is higher than the minimum of 152 mm and 250 mm specified for reinforced masonry in TMS 402/602-22 [14] and NZS 4297:2020 [13], respectively.

Since the case study is intended for a SEM application, a masonry compressive strength $f_{SEM} = 3.50$ MPa was assumed, which is consistent with the range between 3.13 and 4.05 MPa that is found in the literature [16–18].

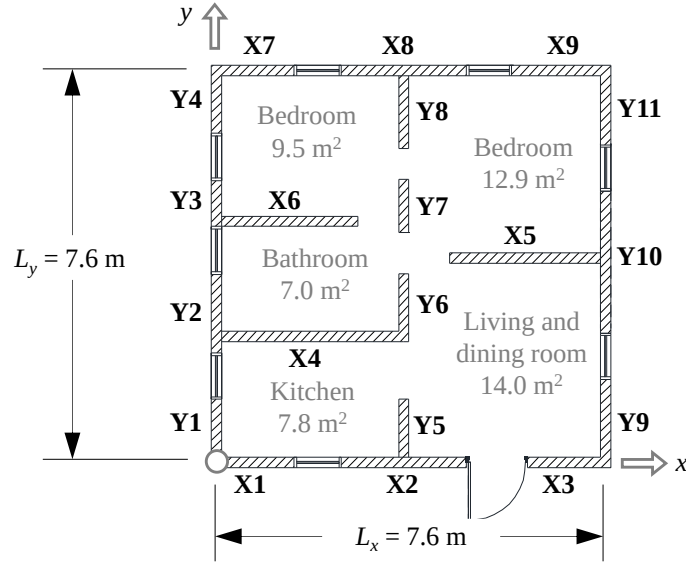


Fig. 1. Floorplan of single-story dwelling in case study.

Table 1. Layout of walls.

x direction				y direction			
Wall	L_i [m]	x_i [m]	y_i [m]	Wall	L_i [m]	x_i [m]	y_i [m]
X1	1.52	0.76	0	Y1	1.22	0	0.61
X2	2.44	3.66	0	Y2	1.52	0	2.90
X3	1.52	6.86	0	Y3	0.91	0	5.03
X4	3.66	1.83	2.44	Y4	1.22	0	7.01
X5	3.05	6.01	3.96	Y5	1.22	3.66	0.61
X6	2.44	1.22	4.57	Y6	1.22	3.66	3.05
X7	1.52	0.76	7.60	Y7	0.91	3.66	5.03
X8	2.44	3.66	7.60	Y8	1.52	3.66	6.86
X9	1.83	6.71	7.60	Y9	1.52	7.60	0.76
-	-	-	-	Y10	2.74	7.60	3.81
-	-	-	-	Y11	1.52	7.60	6.86

The NMSZ is located between the states of Missouri, Arkansas, Tennessee, and Kentucky. It is one of the highest seismic-hazard areas in the US (Fig. 2) [19]. This area offers a valid context also because it provides abundant availability of soil that is suitable for the manufacturing of compressed and stabilized earth blocks [17].

Table 2 presents the peak ground acceleration and the Seismic Design Category (SDC) corresponding to the selected location in New Madrid (MO), following the

provisions in ASCE/SEI 7-22 [11]. The level of seismic-hazard, which is translated into SDCs A, B, C, D and E, serves to determine the seismic force-resisting system (SFRS) that is required in TMS 402/602-22 [14].

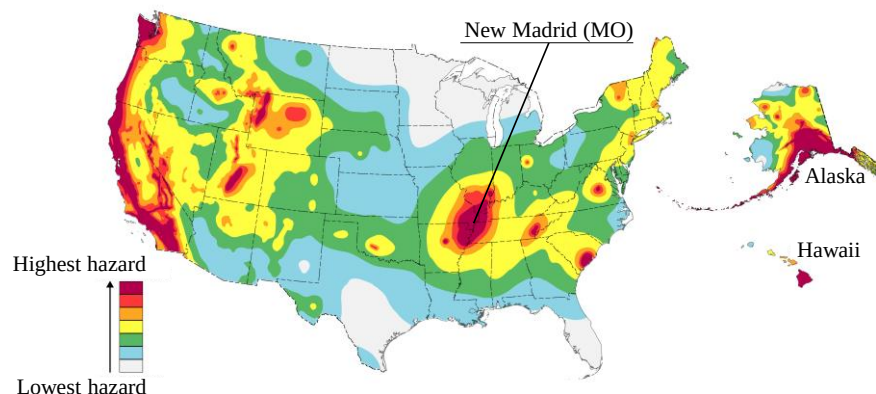


Fig. 2. Location of New Madrid (MO) in USGS seismic-hazard map [19].

Table 2. Peak ground acceleration and SDC for New Madrid (MO).

Location	Peak ground acceleration	Seismic Design Category	SDC based on Modified Mercalli Intensity [11]
	PGA_M [g]	SDC	MMI
New Madrid (MO)	1.43	E	MMI IX: Hazardous damage to robust structures

For structural walls, TMS 402/602-22 [14] mandates specific reinforcement ratios and layouts, which are associated with different levels of ductility that are required as a function of the Seismic Design Category (SDC), as outlined in Table 3. These provisions ensure that the minimum requirements for the reinforcement layout are suitable for the seismic hazard exposure. Table 3 shows the minimum requirements for each SFRS and the corresponding response modification factor, R , which accounts for the ductility to be considered in the in-plane design load calculations (as presented in Section 2.2).

Table 3. SFRSs and associated SDCs [14].

SFRS	SDC	Response modification factor
		R
Ordinary Reinforced Masonry (ORM)	B, C	2.0
Intermediate Reinforced Masonry (IRM)	B, C	3.5
Special Reinforced Masonry (SRM)	B, C, D, E	5.0

For the SDC E that is specified for New Madrid (MO), only the Special Reinforced Masonry (SRM) SFRS is permitted [14] to provide the necessary ductility. Fig. 3 shows the associated minimum reinforcement layout for SRM walls, where ρ_v is the vertical reinforcement ratio, and ρ_h is the horizontal reinforcement ratio.

2.2 Design loads

This section presents the calculation of the vertical loads due to the self-weight of the walls, the dead and live loads of the roof system and snow, and the in-plane seismic loads. The design loads are determined in accordance with ASCE/SEI 7-22 [11].

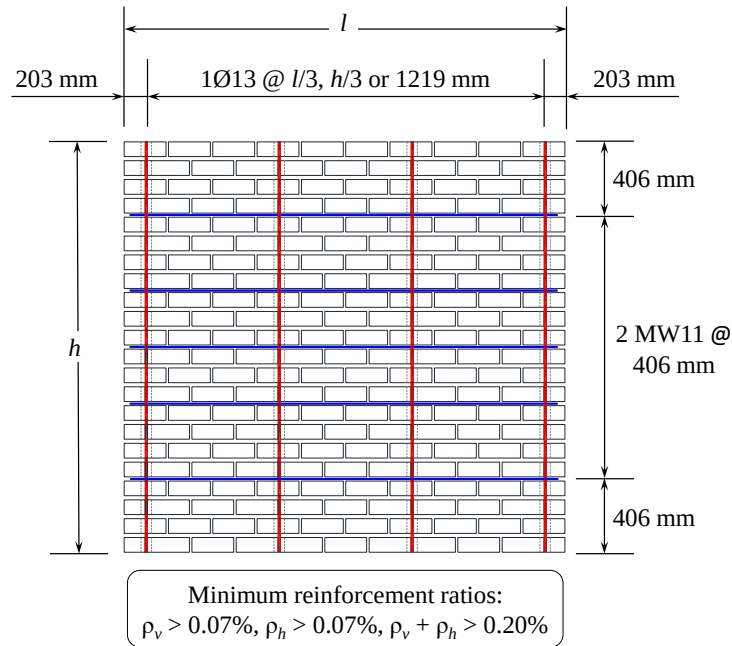


Fig. 3. Minimum reinforcement layout for SRM per TMS 402/602-22 [14].

A specific weight $\gamma_{SEM} = 15.89 \text{ kN/m}^3$ (1620 kg/m^3) [15] was used to estimate the self-weight of the walls. A nominal dead load of 0.71 kN/m^2 for a traditional roof, made of wooden trusses and panels, and asphalt shingles, was assumed. The live load acting on the roof structure was assumed equal to 0.96 kN/m^2 considering a roof area that is not intended for occupancy [11]. The snow load for New Madrid (MO) is equal to 1.05 kN/m^2 [11]. All vertical loads were distributed according to the tributary area for each load-bearing wall.

The seismic load demand was estimated following the Equivalent Lateral Force (ELF) procedure in ASCE/SEI 7-22 [11]. This procedure is applicable to structures for residential construction under any SDC.

The seismic base shear was calculated as:

$$V = \frac{S_{DS}}{\left(\frac{R}{I_e}\right)} W \quad (1)$$

where:

- V = seismic base shear [N]
- S_{DS} = design spectral response acceleration parameter in short-period range (= 2.37 for New Madrid [11]) [g]
- R = response modification factor (= 5 for SRM as noted in Table 3) [-]
- I_e = importance factor (= 1 for residential structures [11]) [-]
- W = effective seismic weight (as defined in ASCE/SEI 7-22 [11]) [N]

The seismic base shear was distributed proportionally to the lateral stiffness of the walls. The resulting in-plane shear force acting on the top of wall i was calculated following the methodology demonstrated in Matta *et al.* [15].

Following the load combinations recommend in ASCE/SEI 7-22 [11], the governing load combination, with seismic load effect, is:

$$1.2D + E + L + 0.15S \quad (2)$$

where:

- D = dead load resisted for each structural wall [N]
- E = earthquake load resisted for each structural wall [N]
- L = live load resisted for each structural wall [N]
- S = snow load resisted for each structural wall [N]

2.3 Design Method

Mechanics-based equations were used to determine the nominal capacity of reinforced SEM walls. Given that shear walls serve as the key structural components to withstand horizontal forces during a seismic event, the potential in-plane flexure and in-plane shear failure modes were analyzed based on the schematics presented in Fig. 4, where: σ_c is the axial compressive stress produced by the snow load and the self-weight of the roof in wall i ; V_i is the shear load on wall i as produced by the seismic forces; W_i is the self-weight load for wall i ; and F_i is the shear force reaction at the base of wall i .

In-plane flexure. Failure under in-plane flexure is due to toe crushing of the masonry or yielding of the vertical reinforcement, as shown in Fig. 4(a). The mechanism is described by the equilibrium between the overturning moment due to the shear load

V_i , and the stabilizing moment generated by the axial loads and the tensile forces resisted by the vertical reinforcement. The resultant compressive force at the toe is calculated assuming the classical rectangular stress block distribution [12]. As a result, the nominal in-plane load associated with flexural failure is calculated as:

$$V_{nf} = \frac{1}{h} \left[\gamma f_{SEM} a t \left(\frac{l-a}{2} \right) + \sum_{i=1}^n A_{si} f_{si} \left(d_i - \frac{l}{2} \right) \right] \quad (3)$$

where:

- V_{nf} = nominal in-plane load associated with flexural failure [N]
- h = height of wall [mm]
- γ = equivalent compression block height factor ($= 0.80$) [-]
- f_{SEM} = compressive strength of masonry [MPa]
- a = depth of compression stress block [mm]
- t = thickness of wall cross-section [mm]
- l = length of wall cross-section [mm]
- n = number of vertical reinforcement bars [-]
- A_{si} = cross-sectional area of bar i [mm²]
- f_{si} = tensile or compressive stress in bar i [MPa]
- d_i = on-center distance of bar i from extreme compression fiber [mm]

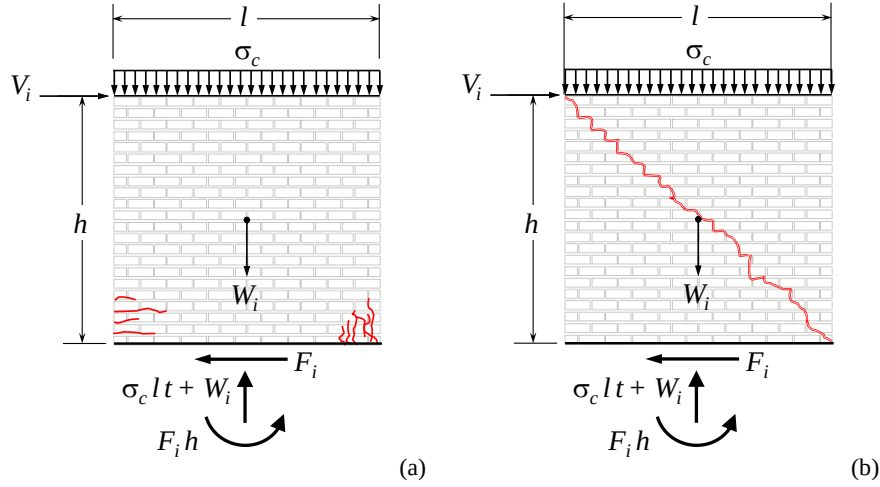


Fig. 4. Failure modes: (a) in-plane flexure, and (b) in-plane shear.

In-plane Shear. In-plane shear failure occurs when the interaction between axial loads and horizontal forces leads to diagonal cracking, as shown in Fig. 4 (b) [12]. As conveyed in Eq. (4), the nominal shear capacity of a reinforced SEM wall, V_{ns} , includes the shear strength contribution of plain masonry (V_{SEM}), acting axial forces (V_p), dowel action of vertical reinforcement (V_{vs}), and horizontal reinforcement (V_{hs}):

$$V_{ns} = V_{SEM} + V_p + V_{vs} + V_{hs} \quad (4)$$

The contribution of the plain SEM and axial forces is defined based on Coulomb's friction theory, and assuming a friction coefficient of 0.30 as specified in NZS 4297:2020 [13] for earth mortar joints. The dowel action provided by the vertical reinforcement and the contribution of the horizontal reinforcement were estimated based on the analytical method presented by Priestley and Bridgeman [20] and Tomaževič [12]. The contribution of the horizontal reinforcement was estimated under the assumption that a 45° crack may not intercept the uppermost and lowermost horizontal reinforcement. This modification represents a conservative assumption recommended by Shing *et al.* [21]. As a result, the nominal in-plane load associated with shear failure is calculated as:

$$V_{ns} = f_{vo} A_n + (0.30 \sigma_i) A_n + 0.619 n d_{vs}^2 \sqrt{f_m f_y} + \left(\frac{d_v}{s} - 1 \right) A_{hs} f_{yh} \quad (5)$$

where:

- V_{ns} = nominal in-plane load associated with shear failure [N]
- f_{vo} = shear strength under zero compressive stress [MPa]
- A_n = cross-sectional area of wall [mm²]
- σ_i = axial stress produced by snow and self-weight of SEM and roof [MPa]
- n = number of vertical reinforcement bars [-]
- d_{vs} = diameter of vertical reinforcement bar [mm]
- f_m = compressive strength of mortar or grout (assumed $\approx f_{SEM}$) [MPa]
- f_y = yield strength of vertical reinforcement bar [MPa]
- d_v = effective shear depth of cross section [mm]
- s = on-center vertical spacing between horizontal reinforcement wires [mm]
- A_{hs} = steel cross-sectional area in one horizontal reinforcement layer [mm²]
- f_{yh} = yield strength of horizontal reinforcement wires [MPa]

Since diagonal cracking is expected to initiate at the middle height of the wall, the axial stress contribution at the critical section is:

$$\sigma_i = \sigma_c + \frac{W_i}{lt} \quad (6)$$

where:

$$W_i = \frac{\gamma_{SEM} h l t}{2} \quad (7)$$

The shear strength under zero compressive stress is assumed equal to 1/25 times the compressive strength of the SEM. This value is more conservative than the ratio of 1/15 specified in NZS 4297:2020 [13]. This choice is consistent with evidence

gained through triplet tests for SEM with compressive strength in the range 3.66-4.05 MPa [16, 17], which applies to this study.

Strength Reduction Factors. The strength reduction factors specified for reinforced masonry in TMS 402/602-22 [14] and presented in Table 4 are used to determine the design in-plane load capacity associated with flexural and shear failure from the nominal values calculated using Eqs. (3) and (5). Design strength values will be compared with design loads to evaluate feasibility.

Table 4. Strength reduction factors for reinforced masonry [14].

Action	ϕ
Flexural	0.65 – 0.90 (depending on net tensile strain of vertical reinforcement)
Shear	0.80

3 Results and discussion

Considering the distribution of vertical and in-plane loads and the governing load combination in Eq. (2), wall Y10 was found to be the one with the critical combination of horizontal and vertical design load demand. The reinforcement layout that was designed for wall Y10 to withstand the design loads is presented in Fig. 5. This design is based on earth block length and depth (height) of 260 mm and 90 mm, respectively, and a thickness of 15 mm for the mortar bed and head joints. According to the minimum requirements for SRM [14], the reinforcement layout for the critical wall consists of: four vertical bars of Ø16 mm spaced at 827 mm on center ($\rho_v = 0.11\%$), and nine pairs of MW32 horizontal reinforcement wires spaced 310 mm and 210 mm as is presented in Fig. 5 ($\rho_h = 0.09\%$). The yield strength of the vertical and horizontal reinforcement (f_y and f_{yh}) was assumed as 420 MPa and 485 MPa, respectively.

The reinforcement layout for the critical wall (Fig. 5) was designed using customary sizes and yield strength values for the reinforcement bars and welded wires. The total vertical and horizontal reinforcement ratio ($\rho_v + \rho_h = 0.20\%$) satisfies the minimum requirement in TMS 402/602-22 [14]. In addition, the ductility requirement is satisfied by ensuring that flexural failure is governed by yielding of the vertical reinforcement. For the critical wall Y10, Fig. 6 shows the design flexural and shear capacity curves as a function of the vertical (axial) and in-plane design loads, assuming a wall thickness $t = 254$ mm, a compressive strength $f_{SEM} = 3.50$ MPa, and the reinforcement layout shown in Fig. 5. It is noted that the axial load in Fig. 6 only accounts for the load more than that due to the self-weight of the wall. The resulting design axial (vertical) load due to the weight of snow and roof structure is 3.38 kN, with an associated average compressive stress of 5 kPa. The design in-plane (seismic) load is 63.3 kN, with an associated average shear stress of 91 kPa.

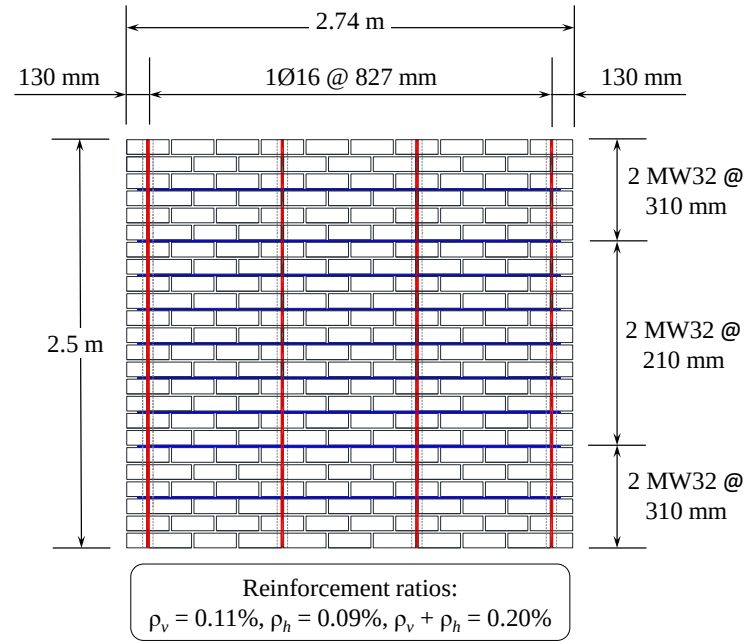


Fig. 5. Reinforcement layout for critical wall Y10.

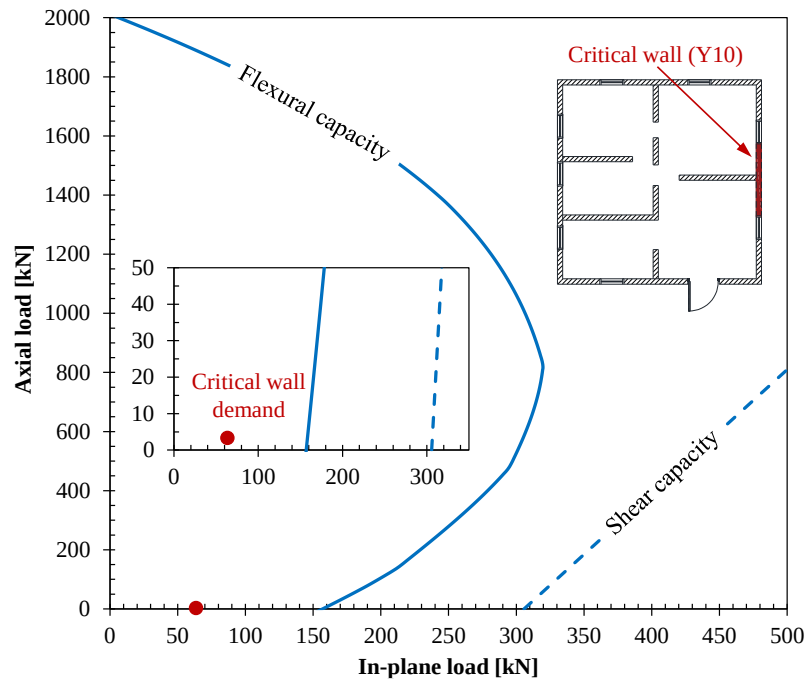


Fig. 6. Design capacity and demand for critical wall with design shown in Fig. 5.

Based on the results of the analytical assessment, the ratio of design strength-to-design load for flexural and shear failure is equal to 2.6 and 4.6, respectively. These ample margins of safety allow for a satisfactory assessment of feasibility. Due to the relatively large amount of horizontal reinforcement mandated for SRM [14] (*i.e.*, around twice the amount required for ordinary reinforced masonry), the governing failure mode for the critical wall (Y10 in Fig. 1) is flexure (reinforcement yielding) rather than shear (diagonal cracking).

4 Conclusions and future work

For the representative case study of a single-story dwelling located in New Madrid (MO), this paper demonstrates that it is feasible to design the load-bearing walls using reinforced SEM. In addition, based on the calculated margins of safety against flexural and shear failure (equal to 2.6 and 4.6, respectively), the results suggest that while seismic force resisting systems other than SRM are not permitted for SDC E [14], smaller amounts of reinforcement might be feasible for SEM structures in high seismic-hazard zones in the US.

Ongoing collaborative research between the University of South Carolina, Columbia (USC), and the University of California, Davis, aim to experimentally verify the findings of this feasibility assessment through physical experiments. Fig. 7 shows the full-scale wall specimens that were designed and built for this purpose.

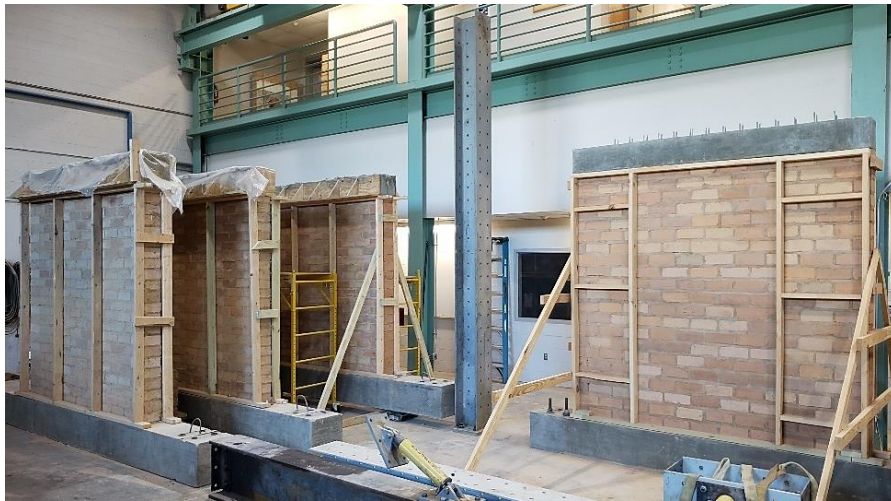


Fig. 7. Full-scale SEM wall specimens for upcoming in-plane cyclic load tests at USC.

These specimens were designed with two different reinforcement layouts based on the analytical method presented in this paper. They are slated for testing under cyclic in-plane loads at the USC Structures and Materials Laboratory.

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