Steel Supported Masonry Walls

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Structural steel beams and columns are often used to support masonry walls subjected to transverse and axial loads. Warehouse and gymnasium type buildings often have large floor-to-ceiling heights, such that allowable masonry wall heights are exceeded. Steel framing, tied to the masonry wall by shear connections, tie bars or other devices, works compositely with the wall to resist flexural and compressive loads.

Standard design practice considers the steel framing as a plane grid or as simply supported framing providing lateral support to the wall. The lateral load on the contributory wall area is assumed to be transmitted directly to the steel framing. Composite action is often neglected.

This study considers the steel framing stiffness requirements for composite action to stiffen non-bearing masonry walls subjected to transverse loading.

METHOD OF ANALYSIS

A masonry wall stiffened by a steel beam and column framing system subjected to transverse load is compared to a non-stiffened wall. A section of an infinitely long stiffened wall is shown in Fig. 1. The beams are at wall mid-height and the columns are spaced at distance a. The wall height, h, selected as 26 ft, is based on an assumed 24-ft ceiling height; the transverse load is based on wind loading.

Both the stiffened and non-stiffened walls are analyzed by the finite element model shown in Fig. 2. The top and bottom of the wall are assumed simply supported, as per current design practice, although masonry walls on footings have some flexural restraint.[1] The steel framing is assumed pin-connected. The model assumes a strip of wall, of width equal to the column spacing, from an infinitely long wall; the slope of the horizontal deformed shape at the edge of the wall strip is equal to zero.

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No studies have been conducted to determine the effective flange width, b_E , either vertically or horizontally, for composite action. Two block courses horizontally and four courses vertically, as shown in Fig. 3, are assumed to act compositely as a T-section. Elements are modeled with thickness as determined by the standard transformed area analysis.

For comparison of stiffened vs. non-stiffened walls, one load condition is used. A uniformly distributed lateral load of 20 psf is used to approximate the wind load. Wall dead load and roof load are neglected for the non-bearing wall analysis. This will result in maximum flexural tensile stress in the wall.

The stiffened wall finite element model consists of 25 elements, as shown in Fig. 2. This grid is coarse, but when considering the variability of material properties and construction practices in masonry construction, is deemed adequate for comparison purposes.

An existing finite element computer program[2] is used. For each plate element there are twelve local degrees of freedom, and a cubic element displacement function is assumed.

The modulus of elasticity of masonry is 2,100,000 psi and that of steel is 29,000,000 psi, resulting in a transformation constant of 14. The moment of inertia of the wall corresponds to an 8-in. thick wall; moments of inertia for beams and columns vary from 100 to 300 in.^{4}, corresponding to various W8 and W16 sections commonly used in design.

Column spacing corresponding to a/h ratios of $1/2$, 1, and $1\,1/2$ are used. Beams are at wall mid-height. By changing one parameter while keeping the others constant, individual stiffening effects are evaluated.

The results for maximum moment for the vertical span are used as a basis for the following discussion. The maximum stiffened and non-stiffened wall moment as well as column moment occur at wall mid-height. The maximum beam moment occurs at mid-span.

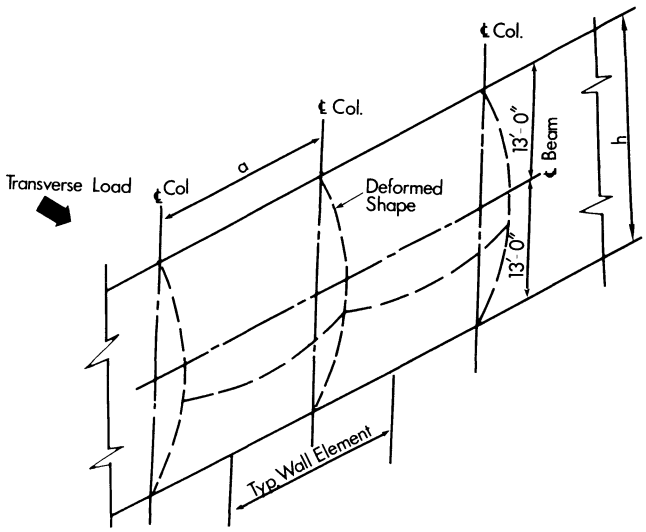

Fig. 1. Typical stiffened wall element

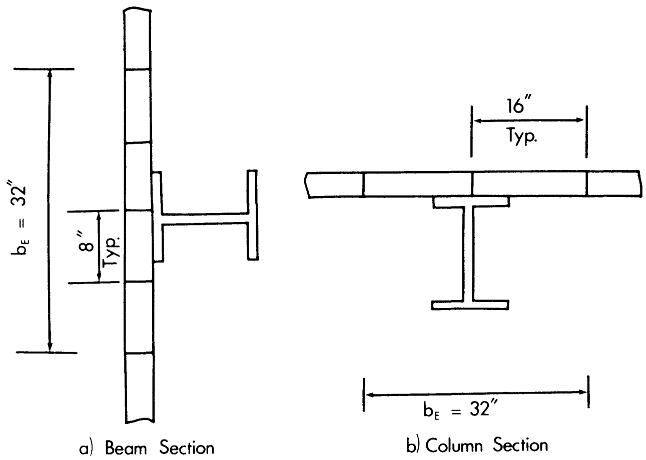

Fig. 3. Typical composite sections

RESULTS OF ANALYSIS

The maximum non-stiffened wall moment in the vertical span, M_w , is reduced by the addition of steel framing. The stiffened wall moment, M_{wt} , is the maximum moment in the vertical span of the wall after the addition of the steel framing. The moment reduction is dependent on the aspect ratio, defined as column spacing to wall height, a/h, and on the beam and column stiffness. These relationships are shown in Fig. 4. It is observed that increased beam stiffness does not reduce wall moment significantly; column spacing and stiffness are predominant.

Figure 4 gives the stiffness requirements for beam/column composite action. Figures 5 and 6 give the corresponding strength requirements. For various stiffnesses, the maximum column and beam moments are obtained in non-dimensional form related to a basis of the maximum non-stiffened wall moment.

The results of the analysis are demonstrated by an example of a 26-ft high non-bearing wall subjected to a 20 psf wind load, in which it is desired to reduce the maximum moment in the wall by 40% (i.e., $M_{wt}/M_w = 0.6$). From

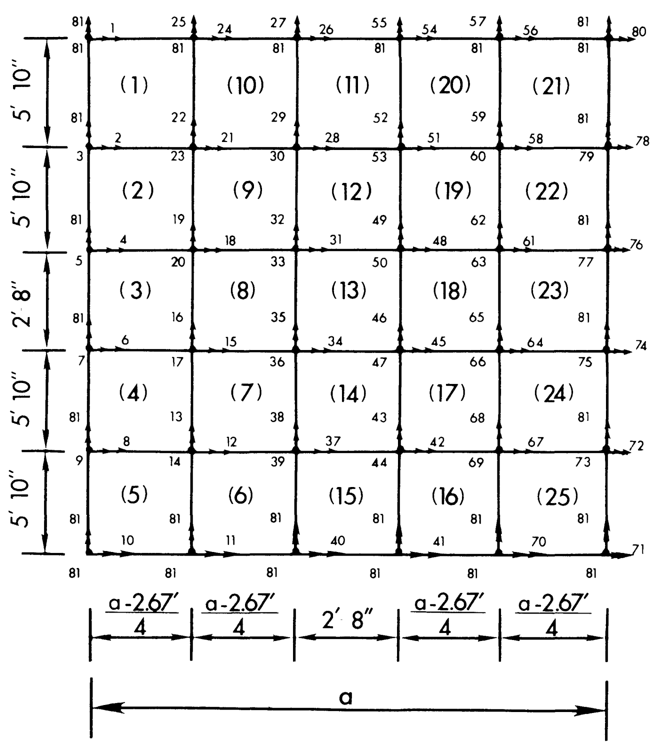

Fig. 2. Finite element model of wall

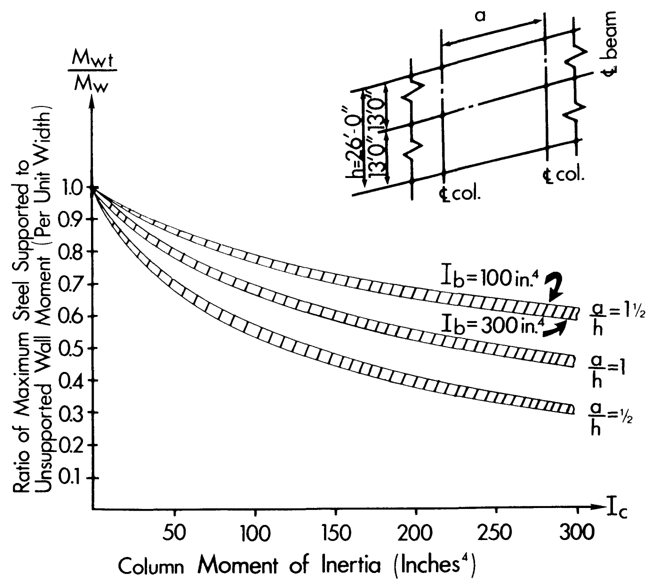

Fig. 4. Reduction in wall moment related to beam and column stiffness

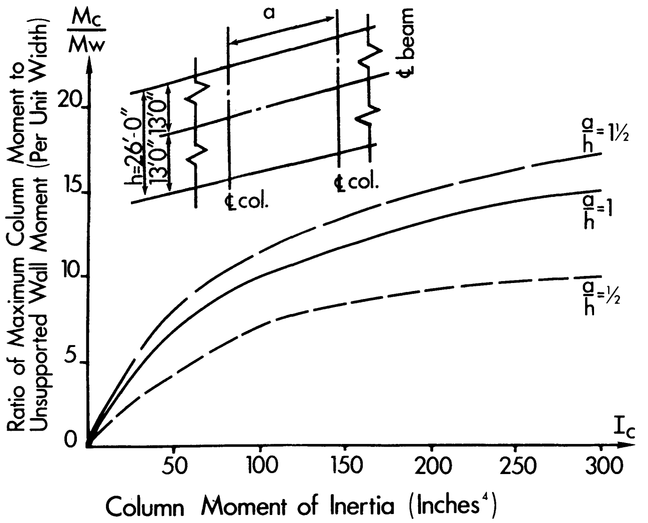

Fig. 5. Column strength-to-stiffness relationship

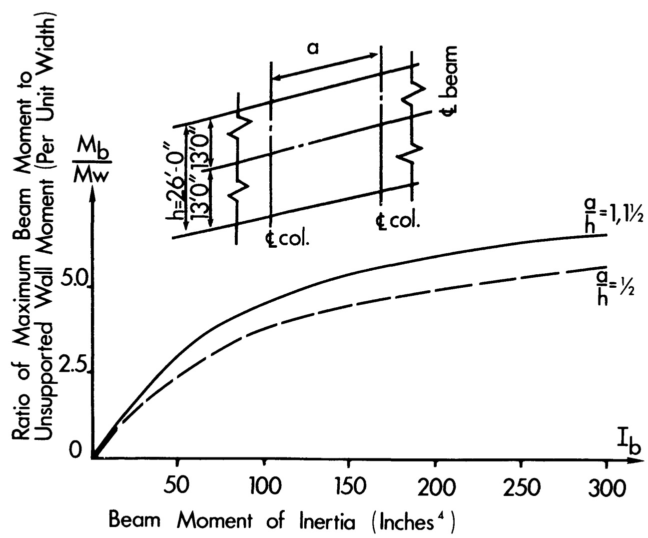

Fig. 6. Beam strength-to-stiffness relationship

Fig. 4, for M_{wt}/M_w = 0.6, any of the following options can be selected:

a) $a/h = 1/2$, $I_b = 300$ in.^{4}, $I_c = 75$ in.^{4}
b) $a/h = 1/2$, $I_b = 100$ in.^{4}, $I_c = 90$ in.^{4}
c) $a/h = 1$, $I_b = 100$ in.^{4}, $I_c = 165$ in.^{4}
d) $a/h = 1$, $I_b = 300$ in.^{4}, $I_c = 140$ in.^{4}
e) $a/h = 1\frac{1}{2}$, $I_b = 300$ in.^{4}, $I_c = 280$ in.^{4}

Additional options could be obtained by interpolation between the curves. For this example, select option (c): column spacing equal to wall height with column and beam stiffness requirements of 165 in.^{4} and 100 in.^{4}, respectively.

The strength requirements are obtained from Figs. 5 and 6. Entering Fig. 5 with $a/h = 1$, and $I_c = 165$, find $M_c/M_w = 12$. The maximum non-stiffened wall moment basis is the vertical simply supported moment of $1/8 \times 20 \times 26^2 = 1690$ ft-lbs. Thus the column strength requirement is $12 \times 1690 = 20{,}300$ ft-lbs. For A36 steel laterally supported, the required section modulus is 10.1 in.^{3} A similar analysis using Fig. 6 results in a required beam section modulus of 3.8 in.^{3}

A W10X33 column is selected to attain a moment of inertia of 165 in.^{4} and a section modulus of 10.1 in.^{3} A W10X21 beam is selected to attain the corresponding stiffness and strength requirements.

CONCLUSIONS

A method of solution to determine beam and column design to stiffen a non-bearing masonry wall is suggested with results given for a 26-ft high wall subjected to 20 psf transverse load. For this situation the column stiffness and spacing are predominant. Column and beam design are determined by stiffness requirements with strength requirements being secondary. Maximum column spacing not to exceed wall height is suggested.

REFERENCES

1. *Yokel, F. Y., R. G. Mathey, and R. D. Dikkers* Strength of Masonry Walls Under Compressive and Transverse Loads *Building Science Series 34, National Bureau of Standards, Washington, D.C., March 1971.*
2. *Wang, C. K.* Computer Methods in Advanced Structural Analysis *1st. Ed., Intext Educational Pub., New York, 1973.*