Proceedings of Ninth World Conference on Earthquake Engineering
August 2-9, 1988, Tokyo-Kyoto, JAPAN (Vol.VIII)

SL-12

SEISMIC DESIGN OF REINFORCED MASONRY

Gary C. HART

1 Professor, Department of Civil Engineering, University of California, Los Angeles and Englekirk and Hart Consulting Engineers, Inc. U.S.A.

SUMMARY

The seismic design of masonry requires ductility and ductility requires minimum nominal reinforcing. In order to rationally develop a reinforced masonry design criteria it is desirable to use a Limit State Design approach with design values selected through the use of structural reliability theory.

INTRODUCTION

Masonry must be reinforced in order to have ductility. The addition of small amounts of reinforcement, for example, #4 bars at 48 inches on center in each direction, can provide a shear wall with excellent ductility. In seismic design, ductility is important because further research may indicate that the intensity of ground shaking used in structural design may be underestimated. Ductility provides a "safety net" for such an occurrence.

Excellent summary papers have been presented which provide for individual countries the philosophical design approach and in many cases the prescriptive requirements of a reinforced masonry design criteria. The proceedings of the International, North American and Canadian Masonry conferences contain these papers. This paper will present an approach which I believe to be the most desirable for the seismic design of reinforced masonry. This approach incorporates the ideas and thoughts of many engineers and researchers from Japan, Italy, Great Britain, New Zealand and China. Note that the only current reinforced masonry seismic design criteria which incorporates this approach is the Masonry Shear Wall Design Criteria in Section 2412 of the 1988 Uniform Building Code.

LIMIT STATE DESIGN

Historically, masonry design, like concrete design, was working stress design. With the movement out of this design approach into one which uses ultimate loads and capacities it was logical to call the newer design approach by the name Ultimate Strength Design, or alternatively, Strength Design. Section 2412 of the 1988 UBC is entitled Strength Design. However, to be exact it is the Strength Limit State that is being addressed in Section 2412.

BEHAVIOR STATES AND LIMIT STATES are used by the engineer to help describe the probable states of response which shear walls or other structural members can be expected to experience. Limit states are defined in terms of parameters which can be assigned values which can be used to develop design equations. When the probabilistic methods of structural reliability are combined with limit state design it is possible to develop a physically meaningful and rational design approach. It is the intent of this paper to illustrate this approach through the use of a design example.

EXAMPLE OF A TYPICAL BUILDING

Concrete masonry buildings include hotels, apartments and condominiums. The layout is usually a central corridor with rooms on each side of the corridor. Figure 1 shows such a plan. The walls perpendicular to the corridor are typically load bearing walls and are spaced at 28 feet on center. They are denoted as transverse walls in Figure 1. Figure 2 shows an elevation view of these walls. The corridor walls are denoted as longitudinal walls in Figure 1 and are usually not load bearing walls.

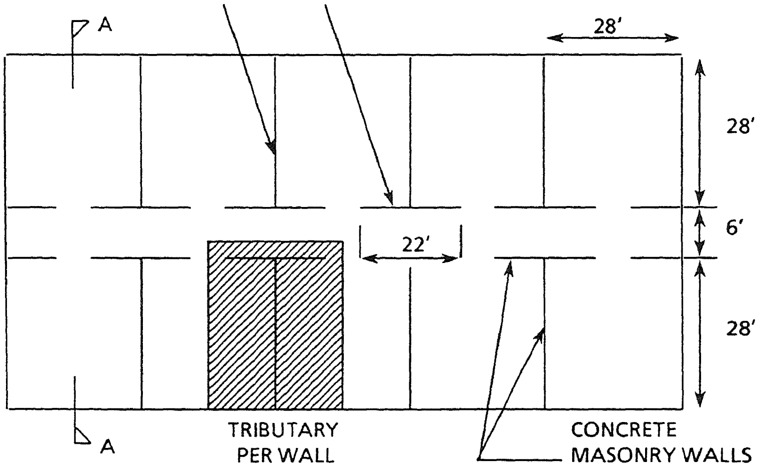

FIGURE 1
FLOOR PLAN OF A TYPICAL BUILDING

In this paper, rectangular load bearing walls 28 feet in length are considered. The walls have an 8-inch nominal thickness, a specified masonry prism compressive strength of 3,000 psi and Grade 60 steel reinforcing.

MAXIMUM NUMBER OF STORIES

Consider a design Limit State to exist when the axial load on the wall is equal to the balanced design axial load. If the axial load, construction process, and material properties were known exactly, and if complete confidence existed in an analytical equation's ability to model the real world, then the nominal design axial load limit for a ductile design region could be 100% of

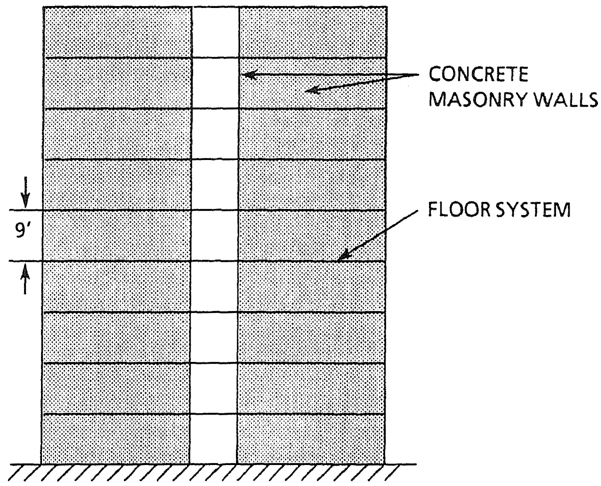

FIGURE 2
ELEVATION VIEW OF TYPICAL TRANSVERSE WALLS

the nominal balanced design load ($P_{b,n}$). However, this is not the case. Therefore, the nominal design axial load corresponding to the upper limit of the ductile design region is denoted as:

$$P_n = \theta_1 P_{b,n} \qquad (1)$$

where the value of θ_1 is selected to provide an acceptable level of safety against this limit state being violated.

A structural reliability analysis of this problem performed by the author and incorporating all parameter uncertainties indicates that it is reasonable to limit the axial load to 65% of the nominal balanced design load (i.e. θ_1=0.65). Therefore, for the building shown in Figures 1 and 2, the maximum height is 12 stories for an 8 inch load bearing wall, which will have a factored axial less than 65% of $P_{b,n}$ using the load factors in Section 2412 of the 1988 UBC.

MINIMUM VERTICAL STEEL

Minimum steel does two things. First, it controls shrinkage. Second, it provides a minimum moment capacity so that there is a smooth performance transition from loads below to loads above the cracking load capacity of the wall. The latter is considered herein.

The nominal cracking moment ($M_{cr,n}$) for our 12 story wall is 9268 kip-ft. In calculating this it was assumed that 20% of the live load exists at the time when the design lateral load is applied and that the value of the modulus of rupture is 4 times the square root of the specified compressive strength.

The structural reliability equation for the limit state where the cracking moment is equal to the cracked section moment capacity is

$$F = M_n - M_{cr} \qquad (2)$$

where M is the moment capacity and failure exists when F is equal to or less than zero. It is possible to develop a design recommendation for the minimum ratio of nominal moment capacity to nominal cracking moment. Similar to Equation (1), the design equation can be written as

$$M_n = \theta_2 M_{cr,n} \qquad (3)$$

where M_n is the nominal moment capacity of the wall. If it is desirable to have an acceptable line of safety, a structural reliability analysis indicates that the nominal moment capacity of the cracked wall section must be at least equal to approximately 1.5 times the calculated nominal cracking moment. It then follows that the minimum nominal reinforced moment is $M_{n,min}=1.5\ M_{cr,n}$ = (1.5)(9268) = 13,902 kip-ft. In order to provide a moment capacity equal to or greater than this moment, it is necessary to use vertical steel equal to #4 bars at 48 inches on center.

The #4 bars at 48 inches correspond to a nominal moment capacity of

$$M_n = 17{,}870 \text{ kip-ft} \qquad (4)$$

The lateral shear force that corresponds to this M_n is

$$V_{n,min} = M_n/(2h/3) = 248 \text{ kip or } 96.9 \text{ psi} \qquad (5)$$

MINIMUM HORIZONTAL STEEL

The limit state in this design situation exists when the earthquake induced shear force is equal to the shear strength capacity of the wall. The earthquake induced shear force is limited by the moment capacity of the vertical steel. Similar to Equation (1) the shear design force can be written as

$$V_{d,shear} = \theta_2 V_{n,min} \qquad (6)$$

where $V_{n,min}$ comes from Equation (5) with M_n calculated using the actual vertical steel. The design shear force $V_{d,shear}$ is called the DUCTILE DESIGN SHEAR FORCE. Note that the design shear force is a function of the vertical steel which is a function of the lateral seismic design force. The 1988 UBC shear design values in Section 2412 of the 1988 UBC are conservative, and thus, it is reasonable to assume that an acceptable level of safety is provided for this limit rate when $\theta_2=1$.

Using $\theta_2=1$ and the shear design values from Section 2412 of the 1988 UBC it follows that, for the case where minimum vertical steel is used in a shear wall, the minimum horizontal steel required is #4 bars at 32 inches on center.

MAXIMUM VERTICAL STEEL

The maximum vertical steel is a function of the maximum reliable shear capacity of a wall. This capacity can be estimated using Section 2412 of the 1988 UBC. This value is equal to 4 times the square root of the specified compression strength. In the design example under consideration that value is 219 psi. The horizontal steel that would correspond to this in the plastic hinge region of the shear wall is #5 bars at 16 inches on center. Therefore,

$$V_{max,shear} = (4.0 \sqrt{f'_m})A \tag{7}$$

and the corresponding moment is

$$M_{max,shear} = (V_{max,shear})(2h/3) = (8/3)hA\sqrt{f'_m} = 374h \text{ kip-ft} \tag{8}$$

The maximum vertical steel that can be placed in the wall is a quantity whose nominal moment capacity is equal to the moment capacity corresponding to the shear related moment capacity $M_{max,shear}$ as calculated using Equation (8).

Using our 12-story residential wall, it follows from Equation (7) that

$$V_{max,shear} = 561 \text{ kips} = 219 \text{ psi}$$

and from Equation (8) that

$$M_{max,shear} = 40,392 \text{ kip-ft}$$

The vertical steel must produce a nominal moment less than $M_{max,shear}$ and this is attained using #9 bars at 16 inches on center. This vertical steel represents a steel ratio of 0.0082.

SHEAR WALL DUCTILITY CAPACITY

The system displacement ductility of a wall can be calculated by dividing the lateral displacement at the top of the wall at compression crushing of the masonry by the lateral displacement at the top of the wall at first yielding of the tension steel. For axial loads less than approximately 15% of the balance design axial load the displacement system ductility is always equal to or greater than 3. However, for higher axial loads a ductility of at least 3 can still be attained but the quantity of vertical steel must be limited to below maximum values.

CONCLUSIONS

Reinforced masonry design based on a strength design limit state has been developed for shear wall design in Section 2412 of the 1988 UBC. The use of a limit state design philosophy enables the engineer to identify undesirable structural behavior and define limit states that will minimize the chance of having such behavior. If limit state design is combined with the analytical tools of structural reliability it is possible to develop design criteria that is easy to use and quantifies seismic risks.

ACKNOWLEDGMENT

The author wishes to acknowledge the funding of the National Science Foundation and the support of the Program Manager, Dr. A. J. Eggenberger.