ULTIMATE STRENGTH DESIGN PROCEDURE OF MASONRY SHEAR WALLS FOR LATERAL FORCES

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SUMMARY

This paper describes the development of an ultimate strength design procedure for clay brick masonry shear wall buildings. This is presented as the basis for an alternative to currently used working stress methods. The procedure incorporates modern probabilistic methods and utilizes recent research data on the shear and flexural modes of failure to develop the procedure for seismic design.

INTRODUCTION

In the past decade masonry research activity has increased substantially. This paper presents the development of an ultimate strength seismic design procedure for clay brick masonry shear walls which incorporates recent advances into the design process. Modern probabilistic methods and structural reliability concepts have been utilized in order to account for the uncertainties in the design and construction of a masonry shear wall and the available test results.

The current design procedures for masonry shear walls, included in codes such as the Uniform Building Code (UBC) are based upon working stress theory. In these codes the design load is not intended to be representative of the maximum earthquake loads that the walls may experience in their useful lifetime and the allowable stresses are working stresses and not ultimate strengths. The advances that have been made in the research area must be incorporated into the design process. One way to achieve this goal is to develop a design procedure which can provide acceptable safety today based upon our current state of knowledge and which can be improved when uncertainties in both earthquake loading and shear wall strength are reduced by future research. This paper presents the development of an ultimate strength design procedure for clay brick masonry shear walls for seismic loads which achieves this goal. The performance criterion considered here is that the walls may crack but they still can carry their vertical loads after the earthquake, thereby preventing collapse of the structure. This is done for the shear and flexural modes of failure. The detailed discussion of all the concepts and steps involved are given in Ref. 6 and partly in Ref. 7. The main points of the procedure are briefly discussed here.

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ULTIMATE STRENGTH DESIGN PROCEDURE

In utilizing an ultimate design procedure for masonry shear walls the design steps are essentially the same as the current procedures for working stress design. For example, for the shear mode of failure, a load factored design shear force is determined and this is divided by the code specified ultimate design shear strength to obtain the required area of shear walls. The difference is that the earthquake design load is multiplied by a load factor and the allowable stresses are more representative of the ultimate strength of the walls.

In developing appropriate load factors and code specified ultimate strengths, the objective is to ensure that the design of a building using the procedure results in a seismically safe structure. In order to achieve this objective the interrelationship of design load and design strength should account for the uncertainties in both the maximum expected loads and the realistic ultimate strength of the as-built walls.

A schematic flow chart of the methodology used here to develop the load factors and design ultimate strengths is shown in Fig. 1 (the dashed line is followed for the flexural failure mode). The methodology involves the following steps: 1) Selection of the code design lateral force; 2) Selection of an appropriate design ultimate strength of the wall; 3) Prediction of actual lateral force; 4) Prediction of the actual ultimate strength of the wall; 5) Calculation of a proper reliability measure; and 6) Comparison of the calculated reliability measure versus society's acceptable measure of safety. This methodology is used for both the shear and flexural mode of failures.

Step 5 involves the definition of a proper reliability measure. Following the modern probabilistic methods and structural reliability concepts, the safety of the wall in each failure mode may be expressed in terms of a performance function:

$$Z = R - S \qquad (1)$$

where R is ultimate shear or flexural strength and S is the maximum expected earthquake load. In general, random variables R and S are functions of other basic random variables. Failure will occur when $\{Z<0\}$; i.e., when the applied load is greater than corresponding ultimate strength of the wall. The <u>reliability index</u> of Z, denoted β, is defined as (Refs. 2, 3, 4, 5):

$$\beta = \bar{Z} / \sigma_{\bar{Z}} \qquad (2)$$

where Z and σ_Z are the mean and standard deviation of random variable Z, respectively. The probability of failure is closely related to this reliability index (Ref. 3). Using mean-value first-order second-moment (MVFOSM) method of reliability analysis (Ref. 5), closed form solutions may be obtained for load factors with regard to S or strength coefficients with regard to R, for given values of β and known means and standard deviations of S and R. A more refined reliability measure is one based on Hasofer-Lind reliablity index (Ref. 4). The recommended value for reliability index, β, is 1.75 for seismic design (Ref. 3).

SEISMIC DESIGN FOR SHEAR

The 1982 UBC gives the seismic design load, V_{sd}, as:

$$V_{sd} = C_o \text{ ZICSK } W_d \tag{3}$$

where W_d is the design specified weight on the wall. In this development $Z = 1.0$, $I = 1.0$, $CS = 0.14$, and $K = 1.33$ have been used. The load factor C_o was selected as 2.0 for this development. However, as discussed later, other load factors may be selected provided the ultimate strengths are scaled accordingly. The design ultimate shear strength, f_{ud}, may be written as (Refs. 6,7):

$$f_{ud} = C_d \sqrt{f'_{md}} \tag{4}$$

where f'_{md} is the design specified prism compressive strength and C_d is the strength coefficient which is a function of wall reinforcement, amount of grouting, and height-to-width ratio of the wall. C_d is the coefficient to be determined.

It can be easily shown (Refs. 6,7) that the actual ultimate shear capacity of a constructed masonry shear wall, V_{ua}, designed using Eqs. 3 and 4 is:

$$V_{ua} = (0.372 / C_d)\, X_A X_W X_{MR} C_T \sqrt{X_f}\, W_d \tag{5}$$

where random variables X_A, X_W, W_{MR}, and X_f represent uncertainties due to the actual size of the wall, workmanship, equation form, and the actual prism strength versus its design specified value; i.e., f'_{md}, respectively. C_T is a random variable whose variation is obtained from the test results.

Using the Riddell and Newmark model (Ref. 9), the maximum earthquake shear force, V_{sa}, can be written as:

$$V_{sa} = X_{MS} X_\Psi \Psi A_g X_b X_D W_d \tag{6}$$

where X_{MS}, X_Ψ, and X_D represent uncertainties in the equation form, nonlinear force-deflection relationship and actual dead load, respectively. A_g is the expected peak ground acceleration at the site and Ψ is the damping and ductility factor. The mean and coefficient of variation (COV) of Ψ for several combinations of damping ratios, ductility factors, and force-deflection relationships are available (Ref. 9). X_b is a random variable used to account for predominantly first mode response of a shear wall building (Ref. 10).

Dividing Eqs. 5 and 6 by W_d and substituting the results for R and S, respectively, in Eq. 1, the performance function becomes:

$$Z = (0.372 / C_d)\, X_A X_W X_{MR} C_T \sqrt{X_f} - X_{MS} X_\Psi \Psi A_g X_b X_D \tag{7}$$

Results utilizing this formulation are discussed in a later section.

SEISMIC DESIGN FOR FLEXURE

Using the 1982 UBC for the flexural design of masonry walls requires designing the walls for the base moment, M_{sd}, due to the total seismic lateral load given in Eq. 3. The procedure for flexural design is different than shear. For shear design a load factor was selected and a corresponding shear strength determined. For flexural design the strength of wall is determined by the wall properties and the load factor (C_o) is determined by the probabilistic proedure. According to 1982 UBC:

$$M_{sd} = V_{sd}\ F(h'_{is},\ w'_{is}) \tag{8}$$

where h_i and w_i are height and weight of i-th story. The function $F(\cdot)$ usually has a simple form which is obtained based upon elastic analysis. Using the same values for Z, I, CS and K as before, then Eq. 8 becomes:

$$M_{sd} = 0.186\ C_o\ W_d\ F(h'_{is},\ w'_{is}) \tag{9}$$

For a lower bound (or optimum) design the wall will be designed to have a design ultimate flexural strength, M_{ud}, equal to design seismic base moment, M_{sd}, using the principles of reinforced concrete section analyses adapted for low strength materials (Refs. 1,6). Then the design ultimate flexural capacity of the wall will be:

$$M_{ud} = M_{sd} = 0.186\ C_o\ W_d\ F(h'_{is},\ w'_{is}) \tag{10}$$

However, the actual ultimate flexural strength of the wall, M_{ua}, will be greater than the design value but is related to its design value, M_{ud}, is as follows:

$$M_{ua} = X_T X_W X_D X_{N1} X_A M_{ud} \tag{11}$$

where X_T is a random variable to account for the test versus design overstrength with a mean COV obtained from tests (Ref. 8) with controlled workmanship and known axial load. The random variables X_W, X_D, X_{N1}, X_A represent uncertainties in workmanship, actual dead load, axial force, and size of the wall, respectively. Eqs. 11 and 10 together give the actual ultimate flexural strength of the constructed wall.

The actual maximum expected earthquake base moment, M_{sa}, can be written easily using Eq. 6 as follows:

$$M_{sa} = X_F X_{MS} X_\Psi \Psi\ A_g X_b X_D W_d F(h'_{is},\ w'_{is}) \tag{12}$$

where X_F is a random variable accounting for error in using the function $F(\cdot)$ to predict the actual maximum earthquake base moment.

Dividing Eqs. 11 and 12 by $W_d\ F(h'_{is},\ w'_{is})$ and substituting the results for R and S, respectively, in Eq. 1, the performance function becomes:

$$Z = 0.186\ C_o X_T X_W X_D X_{N1} X_A - X_F X_{MS} X_D X_b X_\Psi \Psi\ A_g \tag{13}$$

Results utilizing this formulation are discussed in the following section.

DISCUSSION OF RESULTS

The strength coefficient (C_d) for the shear mode of failure is obtained from Eqs. 2 and 7 using the Hasofer-Lind method of analysis. The recommended value of $\beta = 1.75$ (Ref. 3) was used for the analysis and the EERC test results (Ref. 10) were used for determining the mean values of the strength coefficient C_T as given in Table 1. The best available estimates on the mean and COV's of the other variables are given in Table 2. The variable not given in Table 2 is the mean and COV of Ψ since this is different for the shear and flexural modes of failure. These variables were obtained from Ref. 9 and for shear they were based upon an elasto-plastic force-deflection relationship with a damping ratio of 10% and a ductility factor of 2 with $\Psi = 1.09$ and a COV of 0.15. These values are similar to those that would be obtained using a damping ratio of 5% and a ductility factor of 2.75. The force-deflection relationship of masonry shear walls lies within this range.

Because of the large number of variables that contribute to the value of C_d, it is necessary to evaluate how C_d varies as a function of each of these variables separately. This has been presented in graphic form in Ref. 6. A brief discussion of the more important variables follows: The impact of changes in β on C_d is non-linear. A 10% change in β results in an 8% change in C_d. For changes in the mean of the test results, C_T, the impact is linear. A 10% change in C_T will change C_d by 10%. Thus, as more test results become available, a lower COV can be placed on C_T since a more reliable value will be available. The effect of workmanship, X_W, also has a linear impact on C_d. For inspected masonry a uniform PDF with lower and upper bounds of 0.9 and 1.0 was assumed. This corresponds to accounting for the possibility of up to 10% decrease in the strength of the wall due to workmanship. The uncertainty in f'_m is represented by X_f with a mean of 1.25 and a COV of 0.20. The impact of X_f on C_d is less pronounced than other variables due to X_f being in square root form in the performance function. C_d has a highly non-linear variation with the mean of A_g. However, care must be exercised since as A_g decreases for lower seismic areas, the zone factor, Z, of the UBC also decreases. For the analysis, a Z-factor of 1 and an A_g of 0.24g was selected to be representative of the Los Angeles area.

For the variables given in Table 2, the corresponding values of C_d are given in Table 3. The benefit of the procedure is that the impact of changes in any of the variables can easily be evaluated. One of the major decision areas in the utilization of the ultimate strength design procedure is the load factor, C_o. A value of C_o of 2.0 was used herein, however, the ANSI A58.1 1982 load provisions recommend a value of 1.5. If this were adopted, the appropriate values of C_d would be 0.75 times the values given in Table 3. If a Φ factor were incorporated in the procedure, the values given in Table 3 would be increased by $1/\Phi$ since C_d has been calculated such that all the uncertainties have been accounted for.

The load factor, C_o, for the flexural mode of failure is obtained from Eqs. 2 and 13 using the Hasofer-Lind method of analysis. As with shear, a β value of 1.75 was used and values of other variables are given in Table 2. A value of $X_{N1} = 1.0$ with a COV of 0 was used, implying that a cantilever wall was assumed (Ref. 8). A value of Ψ equal to 0.78 with a COV of 0.17 was used and these values correspond to a damping ratio of 10% and a ductility ratio of

4. The higher ductility ratio compared to shear reflects the superior inelastic performance of the wall responding in the flexural mode of response. As with shear, the impact of variations in all the variables was evaluated and presented in graphic form in Ref. 6. For the values given in Table 2, a load factor, C_o, of 1.45 was obtained. Thus, if a load factor of 1.5 as recommended by ANSI A58.1, 1982, is used, walls designed for flexure will perform adequately provided a shear failure is prevented.

From the results presented for the shear and flexural modes of failure, a consistent ultimate strength design procedure can be developed utilizing the ANSI A58.1, 1982 recommended load factor of 1.5 with the 1982 UBC seismic loads and the strength coefficients, C_d, given in Table 3 multiplied by 0.75. appropriate Φ factor.

CONCLUSIONS

A formulation to provide a basis for the ultimate strength design procedure for clay brick masonry shear wall buildings has been presented. This procedure has been developed using probabilistic concepts for seismic loads considering both the shear and flexural modes of failure. The benefit of the method developed is that changes in any of the variables incorporated in the formulation can easily be evaluated. This will permit a continuing evaluation of the procedure as uncertainties in the major variables are reduced with updated research data.

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TABLE 1

C_T (FROM TESTS)

Material	M/VD Ratio	Mean and COV of Ratio of Average Ultimate Shear Stress f_s to $\sqrt{f'_m}$: Light Reinforcement (<.002) Mean	Light Reinforcement (<.002) COV	Heavy Reinforcement (>.002) Mean	Heavy Reinforcement (>.002) COV
Hollow Clay Brick (Grouted)	1.0	4.15	0.10	4.68	0.10
	0.25	5.97	0.10	7.61	0.10

TABLE 2

BEST ESTIMATES ON STATISTICS OF VARIABLES

No.	Name	Notation	Best Estimate Mean	Best Estimate COV
1	Shear Strength	C_T	Table 3	
2	Workmanship	X_W	0.95	0.03
3	Area Correction	X_A	1.0	0.05
4	Model Error	X_{MR}	1.0	0.10
5	Test/Design f'_m	X_f	1.30	0.15
6	Dead Load	X_D	1.05	0.10
7	Model Error	X_{MS}	1.00	0.10
8	SDOF Error	X_h	0.78	0.06
9	Material Model	X_Ψ	1.0	0.06
10	Test/Design M_u	X_T	1.38	0.10
11	PGA	A_g	0.24	0.80

TABLE 3

COMPUTED C_d (SEISMIC LOADING)

Material	M/VD Ratio	Computed Coefficient $C_d \sqrt{f'_m}$: Light Reinforcement <.002	Heavy Reinforcement >.002
Hollow Clay Brick (Grouted)	1.0	2.71	3.05
	0.25	3.89	4.98

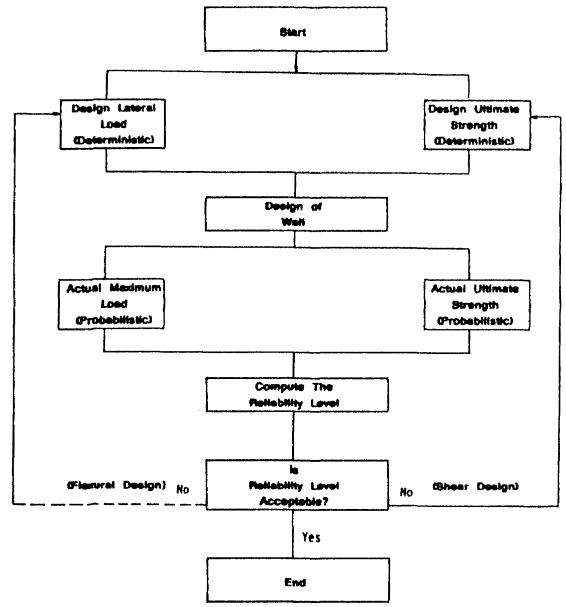

Figure 1
Schematic Flow Chart of Procedure Used to Develop Strength Coefficient or Load Factors