MINIMUM STEEL REQUIREMENTS IN MASONRY WALLS FOR OUT-OF-PLANE FORCES

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SUMMARY

Dynamic amplification spectra for the response at different levels within masonry buildings subjected to earthquake ground motions are presented. Preliminary results of a test program to determine the minimum seismic reinforcement requirements of masonry walls are discussed.

I. INTRODUCTION

The minimum amount and maximum spacing of reinforcement in reinforced masonry walls, either infill or load bearing, is typically specified in codes without regard to the location in the building or the configuration of the walls, and in some cases, to the expected level of seismic activity. Questions have arisen as to the adequacy or inadequacy of the minimum requirements, and there has been a desire to base the code requirements on a more rational analysis.

A research project is presently underway to allow the inertia forces in masonry buildings to be computed on the basis of realistic dynamic amplification factors, and to determine experimentally the capacity and appropriate spacing of the main reinforcing steel, and the effectiveness and appropriate spacing of transverse reinforcement (referred to herein as distribution steel).

It is hoped that the work will lead to the formulation of more accurate detailing and rational design procedures.

The paper presents spectra of dynamic amplification factors for use at various floor levels in a building; an expression for estimating the fundamental period of masonry high rise buildings; the results of quasi-static lateral load tests on reinforced masonry panels.

II. ENVELOPES OF DYNAMIC AMPLIFICATION FACTORS

An elastic modal analysis has been performed on reinforced masonry shear and infill wall systems to determine the dynamic amplification factors at various height levels. (1)* Fourteen buildings in three plan forms, ranging in height from five to twenty five storeys were modelled. Shear

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* Numbers in round parentheses refer to references.

wall slab coupling was represented by a wide column analysis, and infill walls were modelled as diagonal struts within frames. The spectrum used was that prescribed by the National Building Code of Canada (2) with a slight modification made at the high frequency end. The root sum square method was used to combine modes. However, since the modal analysis technique is based on relative displacements of the structure and ground, it predicts zero response at the base of the structure. To circumvent this the peak ground acceleration was included in the root sum square procedure. The dynamic amplification factor at two different levels versus the fundamental period of the building is shown in Fig. 1.

The results were obtained by analyzing 5, 10, 15 and 20 (and one 25) storey buildings, for the three different layouts. The curves are upper bounds of the dynamic amplification, but it is seen that there is not a great deal of scatter. Fig. 2 shows the upper bound curves for five different levels.

For the seismic design of walls in the lateral direction, it is possible, with the use of the curves in Fig. 2, to compute the maximum inertia force once the peak ground acceleration and fundamental period of the building are known. The curves are conservative since they are based on an elastic response and any nonlinear behaviour would tend to decrease the acceleration levels, especially for buildings with periods in the range shown. Since the natural frequency of the walls themselves is much higher than the first few frequencies of the building, the forces caused by the accelerations can be applied as quasi-static loads to the walls and their connections.

III. ESTIMATION OF FUNDAMENTAL PERIOD

Examination of the fundamental period of the buildings studied showed that it could be approximated by (see Fig. 3)

$$T_m = (0.35 + 0.5\ T)\ \underline{T}$$
$$= (1750 + 125\ H/\sqrt{D})(H/\sqrt{D})\ 10^{-5}$$

where
T_m = fundamental period of masonry buildings (secs)
H = Building height (ft)
D = Plan dimension in direction of motion (ft)
T = $0.05\ H/\sqrt{D}$ = fundamental period of buildings given (typically) by National Building Code of Canada (2).

IV. LATERAL LOAD TESTS ON MASONRY PANELS

A. Objectives

An experimental program is in progress with the aim of determining the minimum reinforcement requirements of masonry walls in seismic regions. The first part of the program deals with non-load bearing walls subject to lateral or out of plane forces, and has as its objectives the following:

1. to establish the maximum spacing of main steel without any transverse distribution steel, for walls spanning either horizontally or vertically between lateral supports.
2. to determine the effect of distribution steel on the spacing of the main steel.

3. to determine the efficiency of joint reinforcement, either as distribution steel or as main steel for horizontally spanning walls.

To date the tests have involved only monotonic quasi-static lateral loads (except for two walls subjected to load reversals after "yielding" in the first direction). Quasi-static cyclic tests are planned for the near future, to be followed by dynamic tests on a shake table facility. It is hoped eventually to include vertical and in-plane shear loads.

B. Test Set-up

The test specimens, with minor exceptions, were all 8 x 8 ft. (2.4 x 2.4 m) panels built of 8 x 8 x 16 in. (200 x 200 x 400 mm) ungrouted hollow concrete blocks in running bond construction. Reinforcement was grade 60 (60,000 psi or 414 MPa nominal yield stress) reinforcing bars in grouted cores or bond beams, and 9 gauge (.14 in. (3.6 mm) diameter) ladder type joint reinforcement. The walls were tested in the vertical position and loaded by means of an air bag placed between the test panel and a reaction wall. Hooked flat steel bars spaced every 8 in. (200 mm), that bear on the unloaded face of the panel, provided a simply supported reaction condition (at the bottom the weight of the wall gave some rotational resistance). Horizontally spanning walls were generally supported vertically by sliding teflon base pads in an effort to prevent a lateral reaction from developing at the base.

Lateral displacements were measured at 9 points on a 3 x 3 grid by the use of rotary potentiometers, the lateral pressure was measured with a pressure transducer, and in most cases two strain gauges were placed on every main reinforcing bar.

Control tests on the wall components are reported below. While attempts were made to keep the materials the same from test to test there was considerable scatter; only the averages are presented.

Compressive strength:
Concrete block - based on net area = 4050 psi (27.9 MPa)
Concrete block prism test - net area = 2320 psi (16.0 MPa)
Mortar cubes = 1060 psi (7.3 MPa)
Concrete grout = 3210 psi (27.2 MPa)
Tensile strength:
Bond of mortar to block = 60 psi (20-110 psi range) (.42 MPa)
Concrete block = 210 psi (1.43 MPa)
Reinforcing bars - yield = 67,910 psi (468 MPa)
ultimate = 108,000 psi (744 MPa)
Joint reinforcement - yield = 59,000 psi (410 MPa) based on area
ultimate = 69,000 psi (480 MPa) of 0.017 in.^{2}

The joint reinforcement was galvanized deformed wire in a slightly corrugated pattern. The tensile test was performed on a single wire without any surrounding mortar, allowing the corrugations to straighten out.

C. Test Series

To date fourteen walls have been tested, 8 spanning vertically and 6 horizontally. The numbering of the walls follows the order in which they were tested. Table I shows the wall dimensions, reinforcing and boundary conditions along with the failure pressure, a short description of the mode of failure, and in some cases a load deflection plot. The walls with vertical main steel all have 2-#6 bars (19 mm dia.) for a steel ratio of 0.0011. Based on the measured strength of the reinforcement, this would imply a moment capacity able to resist a load of 250 psf (12 kPa) at yield and 390 psf (19.3 kPa) at ultimate for the 8 foot (2.4 m) spans.

The walls with horizontal main steel include #4 (13 mm dia.) and #6 (19 mm dia.) bars in bond beams, or joint reinforcement as described above.

D. Test Results

1. Vertical Spans:

The recorded pressures are not particularly accurate for the first 3 tests which were exploratory in nature, but the results from tests [1]* and [2] with main steel at 48 ins. (1.22 m) spacing showed that failure did not occur transversely in the blocks between the main reinforcement.

The spacing was then increased in stages to 72 ins. (1.83 m) [4,5,6]. Wall [4] showed the ability of the masonry to span 56 ins. (1.42 m) between main steel at high load. Wall [5] with a steel spacing of 72 ins. (1.83 m) and small edge distances finally failed by a bending mechanism in the masonry between the main reinforcement, although at a high load of 190 psf (9.10 kPa). Wall [6] was an attempt to model a wall similar to wall [5] which had been precracked by earlier or in-plane loading. To this end it was built with the mortar faces of the blocks dipped in Sternson bond release, a compound commonly used to prevent bond between lift slabs poured on top of one another. A transverse #4 bar was placed in the top course to provide some containment capacity. The failure mechanism and load were essentially identical to wall [5]. It is not yet known whether the bond breaker failed to perform, or whether the precracking did, indeed, have little effect.

Distribution steel in the form of joint reinforcement or bond beams was then added to determine the effect on the failure mechanism, with main reinforcement at 72 ins. (1.83 m) [7,8] and 88 ins. (2.24 m) [14]. In wall [7], joint reinforcement was placed in every course so as to give the same reinforcement ratio as #4 bars at 48 ins. (1.22 m) spacing which is essentially the reinforcing for wall [8]. A #4 bar was placed at the top to guard against shear or bond failure in the top course of blocks. Wall [7] failed by a bending mechanism in the blocks while wall [8] failed by bond on the horizontal #4 bar. In both cases the primary bending capacity was also essentially fully developed. Wall [7] performed better in the sense that there was less sign of distress before failure.

In view of the good performance of wall [7], wall [14] was constructed with the main vertical steel at an 88 ins. (2.24 m) spacing, and with

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* Numbers in square parentheses refer to wall numbers.

distribution steel consisting of joint reinforcing in every second course. This wall failed in transverse bending between the main bars.

2. Horizontal Spans:

The horizontally spanning walls have included main steel spacing of 48 ins. (1.22 m) [3,9] and 72 ins. (1.83) [10], and walls with only joint reinforcement as main steel [11,12,13]. No vertical distribution steel has yet been used in these walls. In wall [11], joint reinforcement was placed in every second course; in walls [12] and [13], it was placed in every course. In wall [13], the teflon pads were omitted at the base of the wall, so that there was some restraint and thus two-way bending.

In all these walls [9,10,11,12,13], the failure load was very nearly the same: 125-130 psf (5.98-6.22 kPa). In all cases, there was no cracking and little deformation up to this load, the load capacity dropped off with increased deflections. However, in walls [9,10], failure appeared to be in transverse bending while in [11,12,13] it was in primary bending.

Walls [11] and [12], after being tested to failure in one direction, were reversed and loaded from the other side. Fig. 4 shows that the post-cracking strength was only slightly reduced in the reverse cycle.

E. Discussion

At failure the walls are extensively cracked along the mortar joints which leads one to assume that the moment capacity about a horizontal axis must be very small. However, the running bond construction with essentially rigid blocks means that blocks in successive courses must undergo relative rotation to produce curvature about a vertical axis, and this action may provide resisting moment about the vertical axis. Yield line analysis of the panels between reinforcing bars based on very low moments about horizontal axes indicated that moment capacity of the blocks about vertical axes was in the order of 400-500 lb.in./in. (1.8-2.2 kNm/m) in walls [5,6,14]. The joint reinforcement appeared to be working at about 50,000 psi (345 MPa). Similar analyses for walls [7,8] indicated values about two thirds of these; wall [14] showed first cracks at a moment of 450 lbs.ft. (2.0 kNm/m), which corresponds to a tensile stress in the mortar of about 60 psi (.41 MPa), corresponding closely with the measured bond of mortar to block.

V. SUMMARY AND CONCLUSIONS

Dynamic amplification factors have been presented by means of which the inertia forces acting on block walls at different heights in a typical masonry building may be computed for the relevant level of ground motion.

Results of tests on masonry walls have been presented which, when further extended, will form a basis for designing the economical spacing of the main steel and the appropriate amount and spacing of distribution steel to carry out-of-plane inertia forces. The indications are that, for typical 8 foot (2.44 m) storey heights, the main steel may be spaced at more than 4 feet (1.22 m) and still develop the full primary bending moment.

Joint reinforcement appears to be effective as distribution steel for vertical spans or as main steel for horizontal spans. All these results should be treated with considerable caution at this stage, since they are based on only quasi-static monotonic loading and have not been replicated.

ACKNOWLEDGMENTS

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REFERENCES

1. Chung, R. W-K. Minimum Seismic Requirements for Masonry Walls. A thesis submitted in partial fulfillment of the requirements for the degree of Master of Applied Science, University of British Columbia, Canada, April 1978.

2. Commentaries on Part 4 of the National Building Code of Canada 1975. National Research Council of Canada. Association Committee on the National Building Code, Ottawa, NRCC No. 13989.

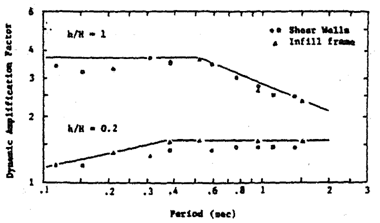


FIG. 1. Dynamic amplification factor spectrum for two levels (H = building height, h = height above base) in a building.

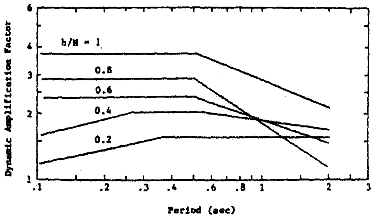


FIG. 2. Upper bound dynamic amplification spectrum curves.

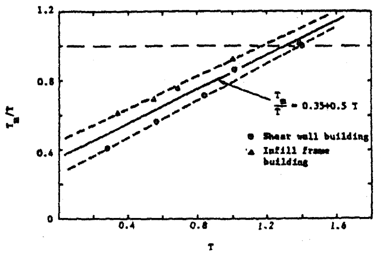


FIG. 3. Correlation of proposed period formula with computed values.

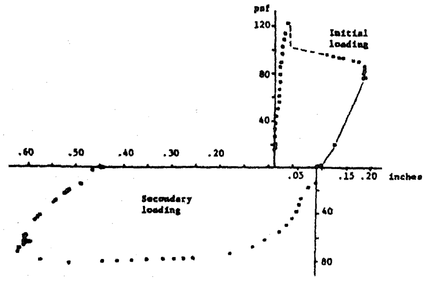


FIG. 4. Wall [11] Load vs central deflection for one complete cycle.

TABLE 1. DETAILS AND RESULTS OF TESTS ON MASONRY WALL PANELS

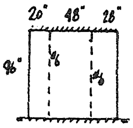
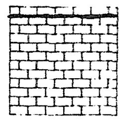
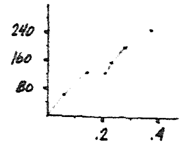
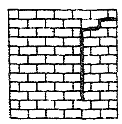
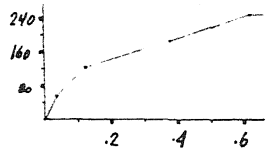
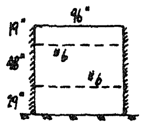
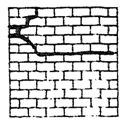
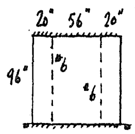
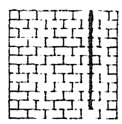
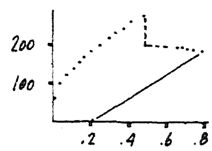
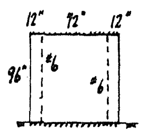
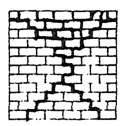
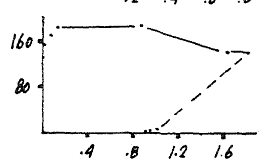
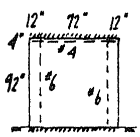
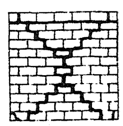
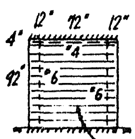
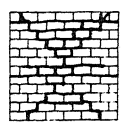
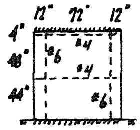
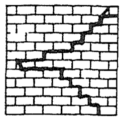
Wall Number	Dimensions, reinforcing, supports	Maximum Pressure, Reinforcement ratio	Mode of Failure	Major crack pattern at failure	Central deflection (inches) vs Pressure (psf)
1.		P_{max} =310 psf ρ_v =0.0011	Sudden shear and bond failure at top course.		
2.	same as 1.	P_{max} =250 psf	Bending failure of cantilever portion about vertical reinforcing.		
3.		P_{max} =160 psf ρ_H =0.0011	Bond failure of top bar allowing cantilever type failure of top portion.		
4.		P_{max} =280 psf ρ_v =0.0011	Bending failure at one side cantilever.		
5.		P_{max} =190 ρ_v =.0011	Bending failure of mansonry between reinforcement.		
6.		P_{max} =210 ρ_v =.0011	As in wall 5.		
7.	 Joint reinforcement	P_{max} =320 psf ρ_v =0.0011 ρ_H =0.0005	As in walls 5 & 6, vertical reinforcement reached yield.		

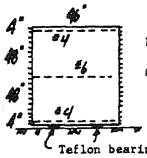
Table 1 continued

8. 

P_{max} =330 psf
ρ_V =0.0011
ρ_H =0.0005

Bond failure of #4 bar near midheight. Verical reinforcement reached yield.

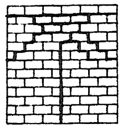


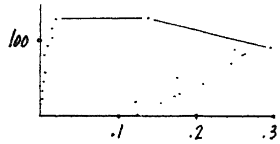
9. 

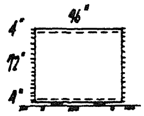
Teflon bearings

P_{max} =130 psf
ρ_H =0.0011

Shear failure along top course then bending mechanism in blocks.

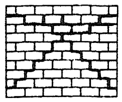


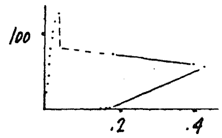


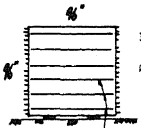
10. 

P_{max} =130 psf
ρ_H =0.0014

Bending mechanism in blocks.



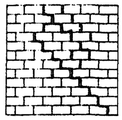


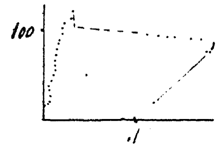
11. 

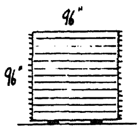
Joint reinforcement

P_{max} =125 psf
ρ_H =0.00025

One way bending at top changing to mechanism near base.

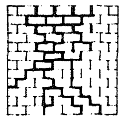


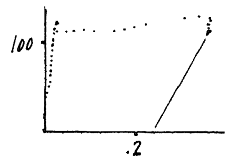


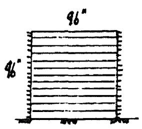
12. 

P_{max} =130 psf
ρ_H =0.0005

One way bending.

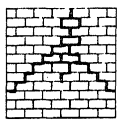


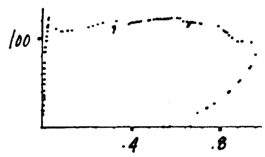


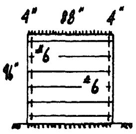
13. 

P_{max} =120 psf
ρ_H =0.0005

Mostly one way bending. Very extensive cracking.





14. 

P_{max} =180 psf
ρ_V =0.0011
ρ_H =0.00025

Bending mechanism. Very extensive cracking.