A METHODOLOGY FOR SEISMIC DESIGN AND CONSTRUCTION OF SINGLE-FAMILY DWELLINGS (SUPPLEMENTARY ENGINEERING ANALYSIS REPORT)

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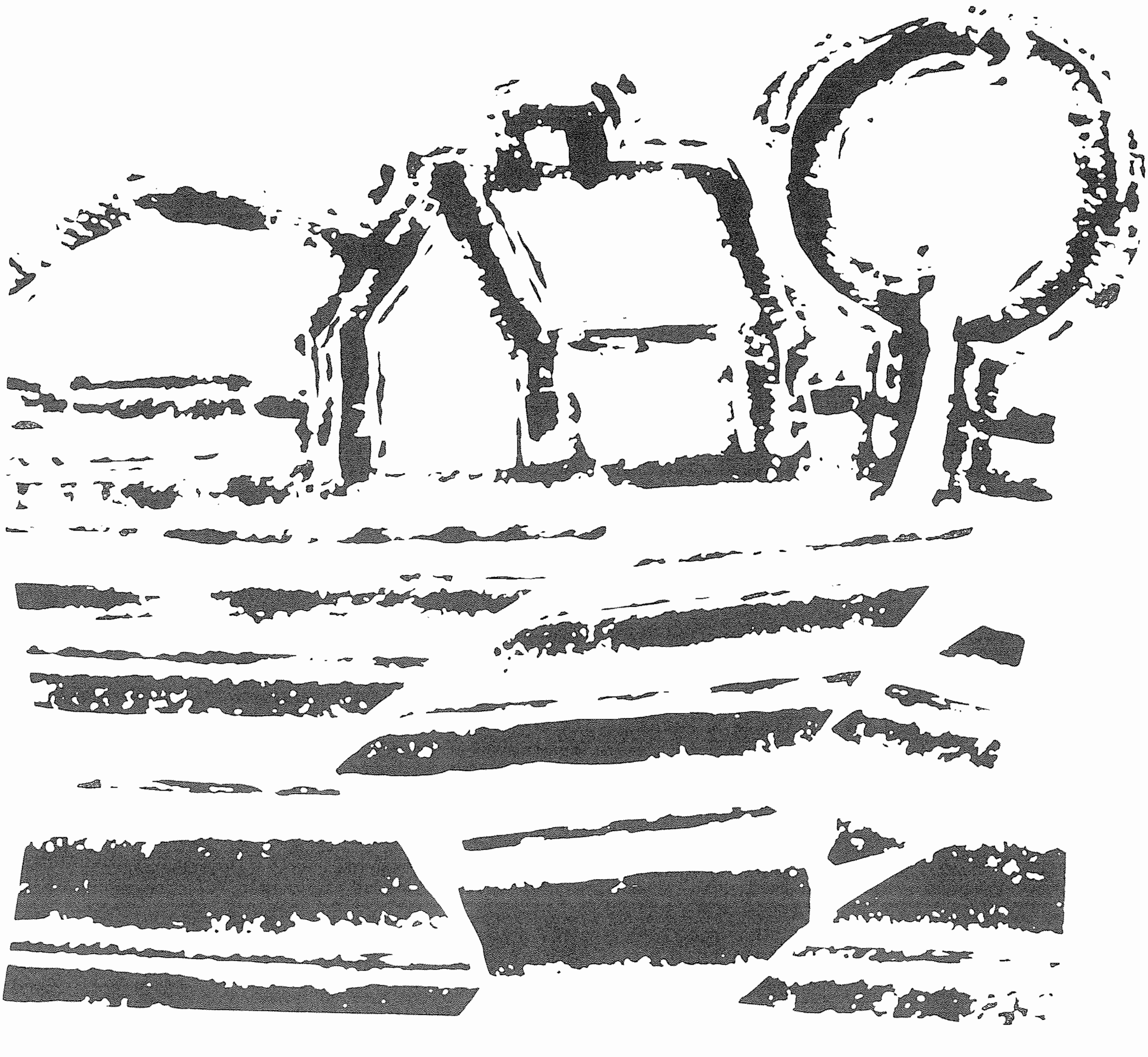
Office of Policy Development and Research

ORIGINAL

A Methodology for Seismic Design and Construction of Single-Family Dwellings

Supplementary Engineering Analysis Report



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SUPPLEMENTARY ENGINEERING ANALYSIS REPORT

For The Report Titled

"A METHODOLOGY FOR SEISMIC DESIGN AND CONSTRUCTION OF SINGLE-FAMILY DWELLINGS"

JANUARY 1977

by

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for

Applied Technology Council

U.S. DEPARTMENT OF HOUSING AND URBAN DEVELOPMENT

OFFICE OF POLICY DEVELOPMENT AND RESEARCH

Division of Energy, Building Technology and Standards

NOTICE

ABSTRACT

This report presents the engineering basis for the Report titled, "A Methodology for Seismic Design and Construction of Single-Family Dwellings".[1] The purpose of that Report was to develop seismic-resistive design and construction recommendations to reduce future probable earthquake caused damage and hazards for single-family residences.

Included in this report are the engineering calculations, reasoning and/or reports of field observations that form the basis for the design and construction procedures recommended in the Methodology. The theory and design calculations given in this report include considerations of the overall structure, as well as specific construction details. Discussions and calculations pertinent to the overall structure give the justification for the recommendations in the Methodology concerning shear wall and diaphragm layout for typical single-family residences. Discussions in this report relevent to construction details are limited to those details which have not been previously used and are introduced by the Methodology or those details that represent a significant departure from previous construction practice.

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INTRODUCTION

This report is prepared for the purpose of providing a description of the engineering basis for the seismic-resistive design and construction recommendations for single-family residences given in the report titled, "A Methodology for Seismic Design and Construction of Single-Family Dwellings".[1] Throughout the remainder of this report, that document is referred to as the "Report" or "Methodology".

Although requirements for seismic design have been included in building codes and other literature for over 40 years, it is believed the subject Report marks the first time that an attempt has been made to set forth the engineering techniques used to achieve these designs for a particular type of structure. Building codes currently allow extremely simplified procedures to be used by non-engineers in meeting seismic requirements for "lesser" structures such as single-family residences. Because home design has become more sophisticated, and frequently incorporates features far removed from conventional one-story, box-like construction, these simple code requirements often proved to be inappropriate for use with the more sophisticated, "unconventional" home designs. The available code requirements also appeared to be insufficient for conventional two-story construction. Finally, certain aspects of construction not previously considered important enough to warrant inclusion in the building codes, such as split level ties and the connection of wood studs to sill plates, have been determined to be important.

The home design and construction techniques used in Southern California were tested on February 9, 1971 in the San Fernando earthquake. Although not considered to be a major seismic activity, this disaster generated accelerations higher than any previously recorded. These accelerations in the major damage areas have not been used in the Report as a criteria for future designs. Of primary importance concerning the conduct of this study was the indication of the mechanisms of failure — the points of weakness and, conversely, those items of the structural system which performed well.

The goal of the Report is to reduce damage to the one type of construction not requiring the attention of an engineer; namely, the single-family dwelling. In order to accomplish this, the user of the Methodology is taken through a step-by-step approach more nearly parallel to the engineering requirements of the code as applied in the structural engineer's office. For the most part, the engineering design requirements of the 1973 Uniform Building Code[2] have been used as the basis for design by the methodology presented. In some instances, field observations or the complexity of the task to be performed indicated a departure from standard engineering procedure was necessary.

This document provides the engineering calculations, reasoning and/or reports of field observations forming the basis for the procedures given in the Report. Where details not previously used are introduced by the Report, calculations and discussions regarding these details are also provided. Since this data is intended for the use of other engineers, calculations normally made on scratch or directly on the calculator, justifying simple connections, have not been included. It is assumed that such designs will be apparent or readily checkable by the reader of this report. Calculations pertinent to each of the subjects discussed are placed at the ends of the various sections. The model homes referred to in the text and calculations in this document are those given in the Report[1].

SEISMIC CRITERIA

While many requirements for earthquake design have been modified in recent years and other major modifications are in process, the design requirement specified by UBC[2] for one- and two-story wood frame and masonry residences has been 0.133 times gravity load, in Zone 3, for over 20 years. One-half of this amount is required in Zone 2. In developing the methodology in the Report[1], the above cited factors (seismic coefficients) were used for design of the overall building. For parts or portions of buildings, the C_p factor (0.2g) in Table 23J of the UBC[2] was used. It is, of course, unknown what future changes might be made with regard to loads, geometry of the structure, site considerations, etc., and therefore, it is impossible to predict what future changes might be required in the Report. No such changes affecting residential structures were known at the time of writing. As long as seismic loads are determined as a percentage of gravity in any given zone, and as long as such percentage is a constant in that zone for residential structures, the only effect changes in seismic coefficients would have on the design section of the Report would be to change the load tables shown as Tables 3.2 through 3.8.

A more serious consequence of the changing of seismic coefficients would be reflected in the details given in the Report, particularly if the seismic coefficient were increased. Such an increase would probably imply the changing of the Nailing Schedule and would certainly affect some of the connections. An attempt has been made to keep as many of the details impervious to this type of change as possible. Although the Nailing Schedule itself might be changed, Details 1 through 6 and 15 through 18 would not be affected. In addition, such a change would not affect Details 19 through 21, Details 23 through 44, 50 through 52, and 58 through 60.

In view of present conventional code specifications with regard to residential construction, it is questionable whether the Report should be revised each time model code changes are introduced.

RIGIDITY ANALYSIS

A large number of split-level homes are constructed with the garage at either the front or the rear of the two-story portion. In such instances, sufficient shear wall on either side of the garage door is rarely provided. A description of the action of these homes in the San Fernando earthquake is contained in Chapter 1-2 of the Report, with particular emphasis placed on the homes shown in Figures 1.3 and 1.7. The floor plans and elevations of Model C (Figures 2.26 and 2.27) are similar to this residence except that Model C has shear wall adjacent to the garage door opening and dimensions vary somewhat.

In addition to the damage shown in Figures 1.7A through 1.7E of the Report, the following observations were made:

1. When the structure eventually came to rest, the front walls adjacent to the garage doors were essentially vertical when viewed head-on.
2. The second floor had 1-5/8" of lightweight concrete fill over the wood sheathing.
3. The nails remaining in the sill plate of the rear wall were only slightly bent.
4. The rear wall of the house shown in Figure 1.3 did not land vertically and had buckled. It therefore did not provide the same vertical support in the first aftershock as did the wall pictured in Figure 1.7B.

The lightweight concrete fill probably added to the rigidity of the second-floor diaphragm, as well as to the inertia force developed. The condition of the nails in the sill plate suggested that a high vertical acceleration had accompanied the horizontal force which removed the studs and finish from the plate. This was lent additional credence by the owner of the house — an engineer who noted the wall had left no drag marks on the ground. A few blocks away, a heavily loaded free-standing bookcase was observed to have scratched the adjacent wall to a height of 13" above its top.

The description in the Report of the actions of the house are based upon supposition. Obviously, all of the actions described must have occurred very nearly simultaneously. While observing the damage, it seemed apparent that the walls adjacent to the garage door were quite flexible and that the diaphragm had to act more nearly like a horizontal-plane cantilever beam with a backspan equal to the cantilever. If the walls were considered to be unyielding supports, this would imply that very nearly 100 percent of the load would have to be resisted by the single interior wall and therefore failed to account for any horizontal load in the rear wall. It further implies that diaphragm deflection to either side of the centrally located wall would be equal, thereby removing diaphragm deformation from consideration.

The shear wall supports are not unyielding and it was also obvious that the interior wall would attempt to deflect more than the rear wall — not only because of greater load but also because of the door opening creating two shorter walls at the interior. Rotation of the diaphragm would therefore be induced. If diaphragm deflection were neglected, the difference in translation from front to rear would be equal to twice the difference in the deflections of the interior and rear walls. It therefore appeared that the diaphragm had been unable to accomodate this much deflection and did indeed attempt to rotate almost from the time of the inception of motion. Wood diaphragms have heretofore been considered as completely flexible. In small diaphragms such as occur in houses, this assumption seemed fallacious.

It should be noted that if the rear wall were strong enough in all its parts to resist the forces induced in it by the rotation and if the interior wall were capable of resisting the high shears induced, the torsional effect described would not necessarily result in severe damage. As it was, the weakness in the connection of the base of the rear wall studs to the sill plate allowed the diaphragm to literally tear the wall apart at this location — probably as the direction of motion reversed. At this point, there can be no doubt that all of the load had to be resisted by the interior wall alone. Whether or not it would have failed (or was failing) as a result of the somewhat lesser load it received prior to the rear wall's failure became moot. All of this is easier to visualize if it is viewed in terms of the ground translating evenly and parallel to the walls discussed. Motion can be considered at the moment when the second floor diaphragm at the interior wall is about to begin translating and again at maximum translation with movement of the diaphragm superimposed over ground motion.

Finally, consideration was given to the fact that the walls adjacent to the garage door were not infinitely flexible. It can be seen that due to the vast difference in rigidity between these piers and the rear wall, the only effect would be to slightly reduce the effective length of the cantilever distance considered with a similar reduction in the rotational effect. Furthermore, it was realized that the same conditions applied to any small, relatively stiff diaphragm supported by shear walls of differing lengths or rigidities at its extremities. Thus, rotation is induced not only by ground motion but also by the character of the structure itself, i.e., the structure acts more nearly like structures with rigid diaphragms. The principle premise for this reasoning to be supported is that the rigidity of small wood diaphragms must be considered when assigning load to walls.

Based on the above observation, it was decided that interrelationship of shear wall and diaphragm rigidities warranted investigation to determine any constraints that might be desirable. In many two-story homes, the interior wall below the second floor is frequently longer than the effective length of either the front or rear walls. The interior wall is also more apt to be a single length such that somewhat more rigidity is developed. It was assumed that interior walls receive greater loads than present engineering techniques suggest, and that calculations might shed further light on this theory.

Limitations

In attempting to analyze wood frame diaphragms and shear walls for their relative rigidities, caution must be exercised in weighing the results. Limitations of the methods available and the many variables involved can influence results greatly. This analysis should therefore be considered for what it is--an extremely simplified first-approach to a very complicated subject. Some of the factors that should be considered in assessing the validity of the conclusions contained herein are as follows:

Formulae Available - The only mathematical formulations found applicable to the deflection of wood frame shear walls and diaphragms are those promulgated by the American Plywood Association for use with plywood-sheathed shear walls and diaphragms. The first two formulae presented in the calculations at the end of this section have been taken from APA material. The method used to derive each formula is shown and then extended to two other conditions of loading. APA does not contend that these formulae have the same degree of accuracy as deflection formulations for more homogenous materials. For this reason, greater accuracy, such as is presented in the subsequent calculations, is not actually warranted. Accuracy to five decimal places has been included only to account for the very small deflections occasionally encountered. The APA formulae also include a term to be used to account for slip in chord splices. Since no data was available for figuring the amount of slip in nailed splices, this term has been omitted in figuring diaphragm deflections. It has, however, been applied to bolts used in hold-down anchors for shear walls and estimated for the more concentrated nailing of strap hold-downs.

Type of Shear Resisting Materials - Obviously, each of the various shear resisting materials specified in the Report has its own deflection characteristics. In most instances, all shear walls in the analysis have been assumed to be sheathed with plywood. The use of other materials would affect deflections obtained for any given wall.

Combinations of Shear Materials - Most residential walls have a shear resisting material (finish) applied to both sides of the wall. Whether or not credit is taken for this combination of materials in determining shear resistance, the combining of such materials will definitely affect the deflection of the wall. Very little data is available indicating the degree to which deflections are influenced by combining materials with different deflection characteristics. The results of a test made combining plywood with gypsum board indicated that the gypsum board shear resistance could be added to that of the plywood. Thus, the stiffness for this seemingly incompatible combination of materials was greater than that for the plywood alone.

Length of Walls - In this short analysis it was necessary to assume specific wall lengths along each line of resistance. While it is felt that these wall lengths reflect the extremes of actual designs to be expected, it is hoped that this is amply demonstrated in the calculations.

Diaphragm Stiffness - In the past, wood diaphragms have been assumed to be completely flexible. In small structures, such as the second floor of wood frame dwellings, this assumption is not entirely correct. In the most extreme example, as shown on the last page of the calculations at the end of this section, the diaphragm is still several times stiffer than the longest shear wall. In the case of Model B, the diaphragm was found to be so stiff that its deflection was considered negligible for most cases. Overall diaphragm ratio appears to play a large part in determining the distribution of loads to shear walls.

Hold-down Anchors - Bolt slip for hold-down anchors has been assumed to be 1/16 inch. In 8-foot-high, 4-foot-long shear walls, this term alone can add 1/8 inch to the deflection of the wall. Obviously, deflections for other wall lengths are inversely proportional to the length of the wall. The amount of tension applied to the hold-down during installation will determine whether the slip is zero or the 1/16 inch assumed.

Assumptions

In addition to the assumptions stated above, it was necessary to assume characteristics of the diaphragm materials and shear wall construction.

Structural I plywood was assumed throughout. The modulus of rigidity (G) was therefore taken as 90,000 psi. Most other grades of plywood normally used would result in a G of 75,000 psi and would increase shear deflection by 20 percent. G can be as low as 45,000 psi with resultant shear deflection of double that calculated.

The chords at each end of each shear wall were assumed to be double 2 x 4's having an area of 10.5 square inches and a modulus of elasticity of 1.50×10^6 psi increased by 10 percent to 1.65×10^6 psi to restore the reduction usually used to account for shear deflection. Diaphragm chords were assumed to be the top plates of the exterior walls perpendicular to the load. Since these plates have staggered splices, a single 2 x 4 with an area of 5.25 square inches and the same modulus of elasticity was considered.

Discussion

Model B - With the normal distribution of loads usually assumed, a diaphragm deflection of +0.001 inch was determined (see calculations for Model B at the end of this section). When the interior wall reaction of 5/8 of total load is assumed, diaphragm deflection was found to be -0.007 inches. In other words, the interior wall would not only restrain the diaphragm from deflecting positively at this support, but would induce a small negative deflection. This is only slightly different from the normal beam where deflection is due primarily to bending. The deflections were considered to be minimal and the diaphragm was considered infinitely rigid for most of the cases checked.

With the walls as shown on the first page of the calculations, rotation of the diaphragm is obviously induced. Because rigidities are variable, a trial-and-error method must be employed. After several trials, the first condition is balanced with 47 percent of the load to the center wall.

In an actual design, the short walls would either be "solid" (as defined by the Report under overturning) and have somewhat less deflection than calculated or would require strap hold-downs. Strap hold-downs at the rear wall are next considered, but no reduction in deflection is assumed for the "solid" front walls. In this instance, 49 percent of the load is resisted by the interior wall.

When angle hold-downs are considered, an interesting phenomenon occurs. If the actual vertical load of 394 pounds per foot on the rear wall is considered together with a small concentrated load at each end, it is found that they alone will resist the overturning induced in the wall because of the small load allowed by the reduced rigidity. In other words, the use of these hold-downs can lead to a negation of the need for any hold-down in a short wall. Obviously, the hold-down will not slip if not subjected to a load and it must be assumed the wall is stiffer than indicated.

When two 4 foot 6 inch walls without hold-downs are assumed front and rear (see Model B calcs), 53 percent of the load is taken by a 19-foot long interior wall and 48 percent by a 16-foot long wall. If the interior wall were designed by the 25-50-25 method of apportioning load, shear to the 16 foot interior wall would be 257.9 lb/ft and 1/2 inch gypsum board blocked and nailed with 5d at 4" o.c. could be used each side. With 5/8 of the load applied, interior shear would be 322.4 lb/ft and plywood would be required. With the gypsum board applied as indicated, load to the interior wall drops from 48 percent to 44 percent with an attendant increase in exterior wall shear per foot of about 10 percent. If the interior wall length were 12 or 13 feet, it is anticipated that calculated shear using the methods shown would be about

300 lb/ft with a deflection of 0.21 inches and would increase exterior wall shears about 20 percent. Approaches such as those used in present building codes place great reliance upon the existance of interior walls to assist the required exterior walls. These walls, at best, would have unblocked 1/2-inch gypsum board each side nailed with 5d at 7" o.c. Unfortunately, no curves are available for deflection of such a wall, but under 300 lb/ft it is estimated that the deflection would be 0.28".

If hold-downs are required at the exterior walls, the condition is considerably better (49 percent to center wall). When the interior wall actually represents 5/8 of the total wall length available, and walls are "balanced", the load to the interior wall is somewhat greater than 5/8 of total load. For unbalanced walls, the eccentricity is obviously reduced (with so much wall at or near the center) with a corresponding increase in load seemingly obvious.

Finally, equal lengths of walls are assumed at the exterior and interior but with the interior wall one piece. In this case, diaphragm deflection must be considered. As might be expected, shears to exterior walls are significantly higher (32 percent of load) than usually considered (25 percent). Although not represented in this example, this condition would more normally occur because exterior walls were long rather than interior walls short. It does indicate, however, that exterior walls can also be overloaded in certain instances. Where the exterior walls are longer, such as at the front wall of the three rotational examples shown on the fourth and fifth pages of the example calculations, the overload will rarely overstress the shear resisting material supplied.

Model C - The analysis of this model is more difficult because the diaphragm is less rigid and most definitely enters into the apportionment of load to the walls. The condition analyzed (see calculations at the end of this section) is that which might be expected in a full design. Hold-down anchors are considered at the interior and front wall with shear resisting material supplied for a 25-50-25 distribution. Note that with 5/8 of the load at the center, closer nail spacing or thicker plywood would be required. The first would reduce nail slip deflection while the second would reduce shear deflection and would reduce nail slip slightly. The provision of either of these would reduce the center wall deflection by at least 0.026" and would result in more load being taken by the interior wall.

Provision of the special garage front wall detail instead of the 4-foot shear wall will increase deflection at this portion since the detail does not provide a way of avoiding bolt slip and increases bending deflection as well. The effect of this detail is considered further in the section titled "Special Garage Front Wall Details".

Conclusions

The brief analysis presented can only give an overview of the interaction of wood stud shear walls and small wood diaphragms. It quite definitely verifies that these diaphragms do enter into the distribution of load to the walls and that they cannot be considered as completely flexible. It would appear, however, that this becomes less true as diaphragm ratio increases and cannot necessarily be extended to diaphragms having larger ratios of length to width.

The omission of the consideration of walls perpendicular to the load has the effect of creating greater reactions due to eccentricity than actually exist. Placing the interior wall at the exact center in all cases considered will either increase or decrease the eccentricity depending on the direction in which it might be located in practice. Considering the trial-and-error nature of the solutions, it was not considered practical to consider a larger number of possibilities. It was concluded, however, that the distribution of load is greatly dependent upon the effective length of the shortest of the two exterior walls. It also appears that load to the interior wall is directly proportional to the degree of balance between the effective length of the exterior and interior walls (the more balanced, the more load to the interior) but inversely proportional to the length of the shortest exterior wall.

The determination of the relative rigidity of walls based upon length alone appears to be reasonably accurate for similar shears when identical materials are used. Because of the nail-slip factor, rigidity of a given wall over a range of shears is not a constant, however, and can vary considerably. Since each wall in a structure can (and most probably will) have a shear per foot considerably at variance with other walls, there is no simple method for determining relative rigidities but the trial-and-error process--for the most part not shown in the calculations--did indicate that linear relationships did work reasonably well once the range of load in each wall was approached. Relative length of walls appears to be as accurate a base as is reasonable in making judgmental decisions regarding tributary areas. Any more accurate method would be quite complex.

Although arbitrary eccentricities caused by the ground motion itself were not considered, the calculations seem to imply that damage to exterior walls is not so much a function of rotation (for many cases) as it is due to the exterior walls receiving more load than standard analysis techniques indicate. One solution for this problem would be to divide total load by total effective shear wall length with the added provision that the longest exterior wall be considered to have an effective length equal to the effective length on the opposing side with making such an apportionment. This would be adequate in some cases and would be more accurate than present methods. Both the reasoning applied in the field and this proposed method, however, fail to consider two items which became evident as the calculations were being prepared. They are:

A. Major damaging earthquakes exert forces well in excess of the equivalent static forces used in design. This is compensated for by the minimum factor of safety of 3 applied to all shear resisting materials but does result in larger deflections than those calculated.

B. Only designed materials have been represented in determining the deflection of each wall. Since combinations of materials are not presently acceptable due to the lack of test data, exterior walls will actually be stiffer than is represented in the calculations. This undoubtedly allows them to accept more load, provided sill bolting, hold-down anchors and wall construction will permit, but also implies they will receive more load. In contrast, interior walls frequently do not have additional shear resisting material and therefore deflect more nearly as predicted. This "softening" of the interior wall will lead to even less load being taken by the interior wall than is shown. As deflections are increased by the higher actual loads, the distribution of more load to the exterior walls would probably increase still further when the diaphragm is stiff.

In view of the above considerations, it was concluded that small overstresses in the exterior walls would be accommodated by the additional, uncredited shear resisting material, but that larger overstress could best be prevented, and overall factor of safety improved, by assuring a greater relative rigidity for the interior walls next adjacent to exterior walls. This can be accomplished by a multiplying factor applied to the tributary area for these walls. Moreover, it appears that the load determined through the use of this factor will be approached or even slightly exceeded when either the interior wall approaches 5/8 of total wall available or when the diaphragm ratio between walls to either side exceeds about 1.5:1, and the interior wall is stiffer than either the shorter of the two exterior walls or both exterior walls.

In order to assure adequate stiffness for the interior wall, 5/8 of the tributary area was selected as an arbitrary minimum to be applied. This results in a multiplying factor of 1.25 for the tributary area developed using standard procedures. Where very short exterior walls are used, resulting in a disproportionate ratio of interior wall length to shortest exterior wall length, the formula was developed for apportioning load to interior walls.

$$P = \frac{L_i \times 100}{L_i + 2L_e}$$

where: P = percentage of total load to interior wall

L_i = length of interior wall

L_e = length of shortest exterior wall

This percentage was then multiplied by 2 to obtain the multiplying factor shown on the curve of Figure 3.2 in the Report. The reasoning used for the application of this technique for conditions where more than one exterior wall is present should be self evident from the methodology as stated in Section 3.3B3 of the Report.

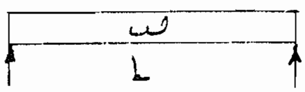
The many combinations of shear wall lengths, diaphragm ratios and materials available would imply that a great deal more work should be done in this area. Until more accurate test data is available, the resulting findings would still be questionable, however. When such data is available, more specific findings may enable more accurate methods to be developed which are at the same time simple enough to warrant use with this type of construction.

RIGIDITY ANALYSIS

SINCE PLYWOOD IS THE ONLY MATERIAL FOR WHICH DEFLECTION FORMULAE ARE AVAILABLE, IT WILL BE NECESSARY TO ASSUME ALL WALLS SO SHEATHED.

DEFLECTION FORMULAE

PER APA - 'PLYWOOD DIAPHRAGM CONSTRUCTION'



$$\Delta = \frac{5vL^3}{8EAb} + \frac{vL}{4Gt} + 0.094Le_n + \frac{\Delta_c X}{2b}$$

BENDING, SHEAR, NAIL SLIP, CHORD SLIP

b = DIAPHRAGM WIDTH

BENDING PORTION IS DERIVED AS FOLLOWS:

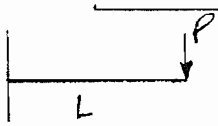
$$I = 2A\left(\frac{b}{2}\right)^2(12^2) = Ab^2\left(\frac{12^2}{2}\right) \quad \text{WHERE } A = \text{CHORD AREA}$$

$$\Delta = \frac{5wL^4(12^3)}{384EI} \qquad v = \frac{wL}{2b} \qquad w = \frac{2bv}{L}$$

$$\Delta_B = \frac{5(2bv)L^4(12^3)2}{384EAb^2(12^2)L} = \frac{5vL^3}{8EAb}$$

FOR SHEAR Δ - AREA OF SHEAR DIAGRAM TO ONE SUPPORT $= \frac{1}{2}v\left(\frac{L}{2}\right) = \frac{vL}{4}$ AND $\Delta_S = \frac{vL}{4Gt}$

FOR NAIL SLIP - .094 IS A COEFFICIENT RELATED TO THE AREA OF THE SHEAR DIAGRAM



$$\Delta = \frac{8vL^3}{EAb} + \frac{vL}{Gt} + .376Le_n$$

BENDING DERIVATION:

$$\Delta = \frac{PL^3(12^3)}{3EI} \qquad v = \frac{P}{b} \qquad P = vb$$

$$\Delta_B = \frac{vbL^3(12^3)2}{3EAb^2(12^2)} = \frac{8vL^3}{EAb}$$

SHEAR AREA $= vL$ $\qquad \Delta_S = \frac{vL}{Gt}$

NAIL SLIP - $\frac{vL}{\frac{vL}{4}} \times 0.094 = 4 \times .094 = 0.376$

ralph w. goers and associates *structural engineers*	JOB: HUD MANUAL	JOB NO. 2220
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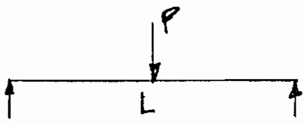
RIGIDITY ANALYSIS

APPLYING THE SAME PRINCIPLES, THE FOLLOWING FORMULAE CAN ALSO BE DERIVED:



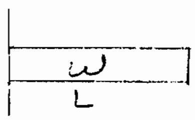
$$\Delta = \frac{PL^3(12^3)}{48EI} \qquad v = \frac{P}{2b} \qquad P = 2bv$$

$$\Delta_b = \frac{2bvL^3(12^3)2}{48EAb^2(12^2)} = \frac{vL^3}{EAb}$$

SHEAR AREA $= v\left(\frac{L}{2}\right) = \frac{vL}{2} \qquad \Delta_s = \frac{vL}{2Gt}$

NAIL SLIP $- \frac{4}{2} \times .094 = 0.188$

$$\Delta = \frac{vL^3}{EAb} + \frac{vL}{2Gt} + 0.188\,Le_n$$



$$\Delta = \frac{wL^4(12^3)}{8EI} \qquad v = \frac{wL}{b} \qquad w = \frac{vb}{L}$$

$$\Delta_b = \frac{vbL^4(12^3)2}{8EAb^2(12^2)L} = \frac{3vL^3}{EAb}$$

SHEAR AREA $= \frac{1}{2} vL = \frac{vL}{2} \qquad \Delta_s = \frac{vL}{2Gt}$

NAIL SLIP $- \frac{4}{2} \times .094 = 0.188$

$$\Delta = \frac{3vL^3}{EAb} + \frac{vL}{2Gt} + 0.188\,Le_n$$

ralph w. goers and associates
structural engineers

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RIGIDITY ANALYSIS

MODEL 'B'

SO THAT WIND LOAD DOES NOT GOVERN, ASSUME:

INTERIOR FINISH - GYP. BD.
EXTERIOR FINISH - STUCCO

ROOF:		CLG:		2^{ND}	
RFG -	3.0	FRMG -	2.0	FLRNG -	1.0
SHTG -	2.0	FIN, -	3.0	SHTG -	2.0
FRMG -	2.0		5.0 PSF	FRMG -	4.0
	7.0 PSF			CLG -	3.0
					10.0 PSF

SEISMIC LD - ROOF - 40x32x7x.133 = 1195
CLG - 36x28x5x.133 = 672
2^{ND} - 36x28x10x.133 = 1344
WALLS - 122x8x10x.133 = 1301
- 87x4x10x.133 = 464
- 128x12x16x.133 = 3277
8253#

WIND LD = 17.67x28x15 = 7420#

SEISMIC LD PER MANUAL

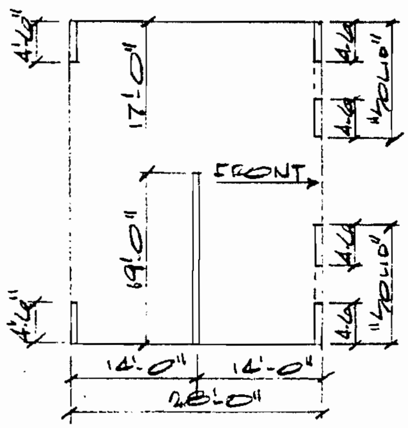
ROOF A = 40x32 = 1280$^{\square'}$ FLR A = 36x28 = 1008$^{\square'}$

W_{RF} = 0.933 W_2 = CLG - 0.667
FLR - 1.333
WALLS - 15+.25 - 1.750
3x1.125 - 3.375
7.125 PSF

1280x0.933 = 1194
1008x7.125 = 7182
8376 VARIANCE = 1.5%

RIGIDITY ANALYSIS

MODEL 'B'



ASSUME REVISED FLOOR PLAN AS SHOWN FOR EASE OF CALCULATIONS. IGNORE END WALLS WHEN CONSIDERING ROTATION OF DIAPHRAGM.

IF ALL LOAD WERE CONSIDERED UNIFORM AND SUPPORTS WERE UNYIELDING, THEN 5/8 OF TOTAL LOAD WOULD GO TO INTERIOR SUPPORT AND 3/16 TO EACH EXTERIOR WALL. BUT SUPPORTS DO YIELD. DUE TO TYPE OF SHEAR RESISTING MATERIAL, LENGTH OF WALLS, PRESENCE OF HOLD-DOWNS, DIAPHRAGM RATIO, ETC., AMOUNT OF YIELD WILL VARY.

CHECK DIAPHRAGM RIGIDITY—

USING FULL LOAD = 8253#, ASSUME EQUAL DISTRIBUTION. THIS IS CONSERVATIVE SINCE ROOF LD, FRONT & REAR EXT. WALLS, ETC., WILL NOT LOAD 2ND FLOOR DIAPHRAGM

$w = \frac{8253}{28} = 294.75\#/'$

IF NO INT. WALL EXISTED: $v = \frac{8253}{2\times 36} = 114.6\#/'$

$$\Delta = \frac{5\times 114.6\times 28^3}{8\times 1.65\times 10^6\times 5.25\times 36} + \frac{114.6\times 28}{4\times 90{,}000\times .5} + 0.094\times 28\times 0.008$$

$$= 0.00504 + 0.01783 + 0.02106 = 0.04393''$$

WITH NO EXT. WALLS: $v = 114.6\#/'$

$$\Delta = \frac{3\times 114.6\times 14^3}{1.65\times 10^6\times 5.25\times 36} + \frac{114.6\times 14}{2\times 90{,}000\times 0.5} + 0.188\times 14\times 0.008$$

$$= 0.00303 + 0.01783 + 0.02106 = 0.04192''$$

WITH INT. REACTION $= \frac{8253}{2} = 4127\#$ $\quad v = \frac{114.6}{2} = 57.3\#/'$

$\Delta_V \;\&\; \Delta_N = 0$

$$\Delta = 0.00504 - \frac{57.3\times 28^3}{1.65\times 10^6\times 5.25\times 36} = 0.00504 - 0.00403$$

$$= 0.00101''$$

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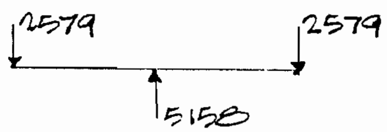
RIGIDITY ANALYSIS

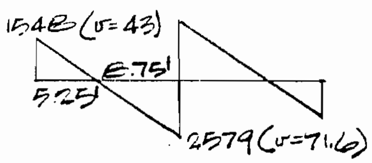
MODEL 'B'

CHECK DIAPHRAGM RIGIDITY—

WITH INT. REACTION = 5/8 x 8253 = 5158#



$\Delta = 0.00504 - \frac{71.6}{57.3} \times 0.00403$
$+ 0.01783 - \frac{71.6 \times 28}{2 \times 90000 \times 0.5} + 0.188 \times 5.25 \times .001$
$- 0.188 \times 8.75 \times 0.002$
$= 0.00504 - 0.00504 + 0.01783$
$- 0.02228 + 0.00099 - 0.00329$
$= -0.00675''$



WITH LOAD AND WALLS AS SHOWN—

IF ALL WALLS RECEIVED EQUAL LOAD PER FOOT:

$v = \frac{8253}{46} = 179.4\#/'$ $F_9 = 1615\#$ $F_{19} = 3409\#$ $F_{18} = 3229\#$

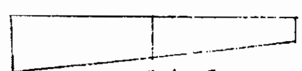
$\Delta_9 = \frac{8 \times 179.4 \times 8^3}{1.65 \times 10^6 \times 10.5 \times 4.5} + \frac{179.4 \times 8}{90000 \times .375} + 0.376 \times 8 \times 0.013$

$= 0.00943 + 0.04253 + 0.03910 = 0.09106''$

$\Delta_{19} = 0.00223 +$ " + " $= 0.08386''$

$\Delta_{18} =$ $= 0.09106''$

CENTER OF RIGIDITY IS OBVIOUSLY FORWARD OF INTERIOR WALL, ECCENTRICITY WILL ∴ ADD LOAD TO REAR AND INT. WALLS AND SUBTRACT FROM FRONT WALL.



3229
−900
2329

1615
+625
2250

3409
+265
3674

$v_9 = 250.0\#/'$ $v_{19} = 193.27\#/'$ $v_{18} = 129.39\#/'$

$\Delta_9 = 0.01313 + 0.05926 + 0.07520 (.025)$
$= 0.14759''$ (.0155) 0.05272
$\Delta_{19} = 0.00241 + 0.04584 + 0.04662 = 0.09487'$
(.075) 0.05484
$\Delta_{18} = 0.00680 + 0.03067 + 0.02256 = 0.06003'$

RIGIDITY ANALYSIS

MODEL 'B'

WITH LOAD AND WALLS AS SHOWN-

IT APPEARS THAT EXTERIOR WALL LOADS ARE APPROACHING EQUALITY. AFTER SEVERAL TRIALS:

$v_4 = 243.56 \quad v_{14} = 203.63 \quad v_{1B} = 121.78$

$\Delta_4 = 0.01280 + 0.05773 + \overset{(.024)}{0.07219} = 0.14272$

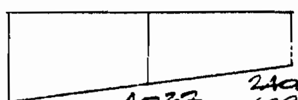
$\Delta_{14} = 0.00253 + 0.04827 + \overset{(.016)}{0.04813} = 0.09893 \quad .04379$

$\Delta_{1B} = 0.00640 + 0.02887 + \overset{(.007)}{0.02106} = 0.05633 \quad .04266$

$\frac{3869}{8253} = 0.469 \qquad \frac{2192}{8253} = 0.266$

ADD STRAP HOLD-DOWN TO REAR WALLS-

ASSUME NAIL SLIP = 0.025" $\Delta_{4-6} = \frac{8}{4.5} \times .025 = 0.04444''$



$v_4 = 202.22 \quad v_{14} = 212.26 \quad v_{1B} = 133.33$

$\Delta_4 = 0.04444 + 0.01062 + 0.04793 + \overset{(.0165)}{0.04963} = 0.15262''$

$\Delta_{14} = 0.00264 + 0.05031 + \overset{(.018)}{0.05410} = 0.10709 \quad .04553$

$\Delta_{1B} = 0.00700 + 0.03166 + \overset{(.008)}{0.02406} = 0.06266 \quad .04443$

$\frac{910 \times 8 - 394 \times 4.5 \times 2.25}{4.5} = 731\#$ STRAP H.D. OK

ADD ANGLE HOLD-DOWN TO REAR WALLS-

ASSUME 1/16" BOLT SLIP- $\Delta_{4-6} = \frac{8}{4.5} \times 0.0625 = 0.11111''$

$v_4 = 116.67 \quad v_{14} = 228.84 \quad v_{1B} = 156.61$

$\Delta_4 = 0.11111 + 0.00613 + 0.02766 + \overset{(.0055)}{0.01654} = 0.16144''$

$\Delta_{14} = 0.00285 + 0.05424 + \overset{(.0205)}{0.06166} = 0.11875 \quad .04264$

$\Delta_{1B} = 0.00833 + 0.03760 + \overset{(.0105)}{0.03158} = 0.07751 \quad .04124$

RIGIDITY ANALYSIS

MODEL 'B'

: WITH TWO 4'-6" WALLS FRONT AND REAR —

WHEN THIS ASSUMPTION IS MADE, CENTER OF RIGIDITY AND CENTER OF MASS COINCIDE

IF SHEARS WERE EQUAL, CENTER WALL WOULD TAKE $\frac{19}{19+2\times 9} = 0.5135 \times$ LOAD

TRY: $V_{19} = 4347\#$ (52.7%) $\quad V_9 = 1953\#$ (23.7%)

$v_{19} = 228.8\#/'$ $\quad v_9 = 217\#/'$

$\Delta_{19} = 0.00285 + 0.05423 + 0.06166 = 0.11874$ (.0205)

$\Delta_9 = 0.01140 + 0.05144 + 0.05565 = 0.11849$ (.0185)

16' WALL AT CENTER - 2 - 4'-6" WALLS FRONT & REAR

$\frac{16}{16+18} = 0.4706$

TRY $V_{16} = 3949$ (47.8%) $\quad V_9 = 2152\#$ (26.1%)

$v_{16} = 246.8\#/'$ $\quad v_9 = 239.1\#/'$

$\Delta_{16} = 0.00365 + 0.05850 + 0.07370 = 0.13585$ (.0245)

$\Delta_9 = 0.01256 + 0.05668 + 0.06768 = 0.13692$ (.0225)

SAME WITH 1/2" GYP. BD. EA SIDE OF INTERIOR WALL

IF DESIGNED FOR 50% LOAD; $v = \frac{8253}{2\times 16} = 257.9\#/'$

TRY 1/2" GYP BD, BLOCKED - FIG. 4 (SEE ATTACHED CURVE)

ASSUME $\Delta_{BENDING}$ SAME AS FOR PLYWOOD

FOR $257.9 \times 8 = 2063\#$; $\Delta = 0.1825"$

BUT BENDING $\Delta_B' = 0.00733"$

16' PANEL WOULD BE 0.00366"

AND $\Delta = 0.18250 - 0.00366 = 0.17884$

TRY $V_{16} = 3601$ (43.6%) $\quad V_9 = 2326$ (28.2%)

$v_{16} = 225.1\#/'$ $\quad v_9 = 258.4\#/'$

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RIGIDITY ANALYSIS

MODEL 'B'

16' WALL AT CENTER - 2-4'-6" WALLS FRONT AND REAR

WITH GYP BD AT CTR (CON'T.)

$\Delta_{16} = 0.16250 - 0.00333 = 0.15917''$

(.028)

$\Delta_9 = 0.01358 + 0.06125 + 0.08422 = 0.15905$

SAME WITH STRAP HOLD-DOWNS ON 4'-6" WALLS

TRY $V_{16} = 4001 (48.5\%)$ $V_9 = 2126 (25.8\%)$

$v_{16} = 250.1 \#/'$ $v_9 = 236.2 \#/'$

$\Delta_{16} = 0.18250 - 0.00370 = 0.17880$

(.022)

$\Delta_9 = 0.04444 + 0.01241 + 0.05599 + 0.06618 = 0.17902$

WALL AT CENTER = 5/8 OF TOTAL WALL - 2-4'-6" FRONT & REAR

$\frac{x}{2 \times 9 + x} = 0.625'$ $x = 30'$ TRY TWO 15' LONG WALLS

FOR 5/8 LD. - $V_{30} = 5/8 \times 8253 = 5158\#$ $V_9 = 1547\#$

$v_{30} = 171.9 \#/'$ $v_9 = 171.9 \#/'$

(.013)

$\Delta_{30} = 0.00271 + 0.04075 + 0.03910 = 0.08256$

$\Delta_9 = 0.00902 +$ " $+$ " $= 0.08887$

INT. WALL WILL TAKE MORE THAN 5/8 OF LOAD.

9'-0" LONG INT. WALL - 2-4'-6" WALLS FRONT & REAR

IF DIAPHRAGM DEFLECTION IS NEGLECTED:

$V_9 = 2837\# (34.4\%)$ $V_{2\text{-}4.6} = 2708 (32.8\%)$

$v_9 = 315.2 \#/'$ $v_{2\text{-}4.6} = 300.9 \#/'$

(.0175)

$\Delta_9 = 0.00828 + 0.07471 + 0.05264 = 0.13563$

(.0166)

$\Delta_{2\text{-}4.6} = 0.01561 + 0.07152 + 0.04813 = 0.13526$

RIGIDITY ANALYSIS

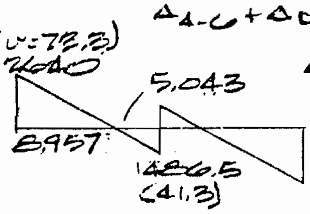
MODEL 'R'

9'-0" LONG WT WALL - 2-4'6" WALLS FRONT & REAR

IF DIAPHRAGM Δ IS CONSIDERED

TRY $V_9 = 2973\#$ (36.0%) $V_{4\text{-}6} = 2640\#$ (32.0%)

$\Delta_{4\text{-}6} + \Delta_{DIA} = \Delta_9 \quad v_9 = 330.3\#/' \quad v_{4\text{-}6} = 293.3\#/'$



$$\Delta_{DIA} = 0.00504 - \frac{41.3\times 28^3}{1.65\times10^6\times5.25\times36} + \frac{73.3\times8.957}{2\times90{,}000\times.375}$$

$$- \frac{41.3\times5.043}{2\times90{,}000\times.375} + 0.188(8.957\times.003 - 5.043\times.001)$$

$$= 0.00504 - 0.00291 + 0.00973 - 0.00309$$

$$+ 0.00410 = 0.01287$$

$$\Delta_{4\text{-}6} = 0.01541 + 0.06952 + \overset{(.0155)}{0.04662} = 0.13155$$

$$\underline{.01287}$$

$$0.14442$$

$$\Delta_9 = 0.00868 + 0.07829 + \overset{(.0195)}{0.05866} = 0.14563$$

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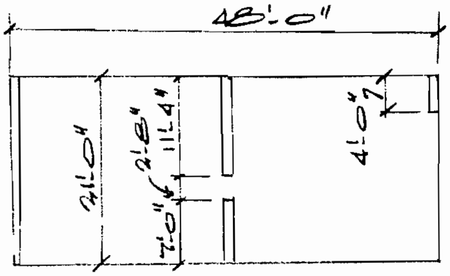
RIGIDITY ANALYSIS

MODEL 'C'



ASSUMED SAME AS MODEL 'C' EXCEPT SOLID WALL IN REAR FOR WORST CASE.

ASSUME 15 PSF WIND FOR EASE OF CALCULATING.

$W = 18 \times 15 = 270\#/'$

TOTAL LD $= 48 \times 270 = 12{,}960\#$

FOR 25-50-25 LOAD DISTRIBUTION:

$v_{SIMPLE} = \frac{6480}{21} = 308.6\#/' \qquad v_P = \frac{6480}{2 \times 21} = 154.3\#/'$

$\Delta_{DIA} = \frac{5 \times 308.6 \times 48^3}{8 \times 1.65 \times 10^4 \times 5.25 \times 21} - \frac{154.3 \times 48^3}{1.65 \times 10^4 \times 5.25 \times 21} = 0.11726 - 0.09381$

$= 0.02345''$

FOR 5/8 @ CENTER — $V_{ENDS} = 2430 \quad V_{CTR} = 8100$

$v = 115.7 \qquad v = 192.9 (\times 2)$

$\Delta_{DIA} = 0.11726 - \frac{192.9 \times 48^3}{1.65 \times 10^4 \times 5.25 \times 21} + \frac{115.7 \times 9 - 192.9 \times 15}{2 \times 90{,}000 \times 5}$

$+ 0.188(9 \times .005 - 15 \times .014) = 0.11726 - 0.11727 - 0.02058$

$- 0.03102 = -0.05161''$

CHANGE IN $\Delta \cong \frac{(0.02345 + 0.05161)100}{8100 - 6480} = 0.00463''/100\#$

WITH 50% AT CENTER —

$V_{ENDS} = 3240\# \qquad V_{INT} = 6480\#$

$v_{21} = 154.3\#/' \qquad v_{8\text{-}4} = 353.5\#/' \qquad v_4 = 810\#/'$

3/8 - 8d @ 6   3/8 - 8d @ 4   1/2" 2 SIDES - 10d @ 4

L H.D.'s   L H.D.'s

$1/16 \times \frac{8}{11.33} = 0.04412 \qquad 1/16 \times \frac{8}{4} = 0.12500$

NOT SHOWN: $\Delta_{11\text{-}4} + \Delta_{7\text{-}0} \cong \Delta_{8\text{-}8}$

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Rigidity Analysis

Model 'C'

With 50% at center –

$\Delta_{21} = 0.00174 + 0.03657 + 0.06016 \;(.020) = 0.09847''$

$\Delta_{18\text{-}4} = 0.00964 + 0.08279 + 0.06166 \;(.0205) + 0.04412 = 0.19921''$

$\Delta_4 = 0.04788 + 0.07200 + 0.07520 \;(.025) + 0.12500 = 0.32008''$

With 5/8 at center –

$V_{ends} = 2430\#$ $V_{int} = 8100\#$ – Use same v resist.

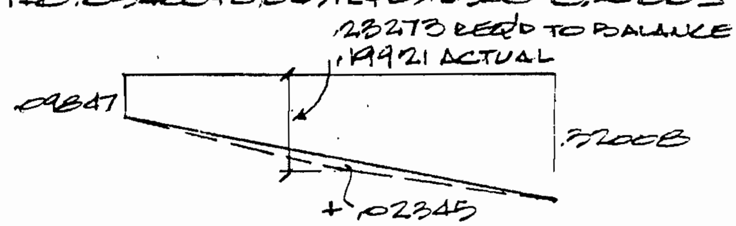
$v_{21} = 115.7\#/1$ $v_{18\text{-}4} = 441.8\#/1$ $v_4 = 607.5\#/1$

$\Delta_{21} = 0.00130 + 0.02743 + 0.03910 \;(.013) = 0.06783''$

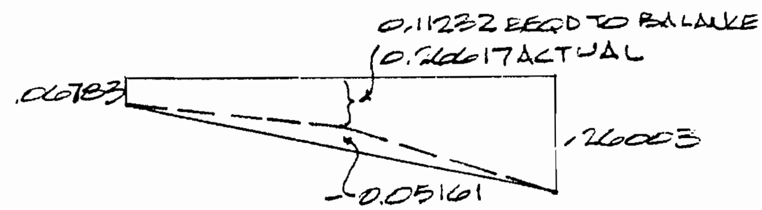
$\Delta_{18\text{-}4} = 0.01205 + 0.10472 + 0.10528 \;(.035) + 0.04412 = 0.26617$

$\Delta_4 = 0.03591 + 0.05400 + 0.04512 \;(.015) + 0.12500 = 0.26003$

1/2 –



5/8 –



Load to ctr wall is greater than 50%
but less than 5/8 – Probably about 52.5%

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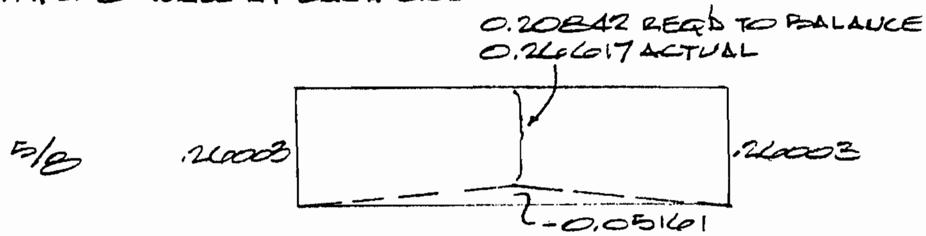
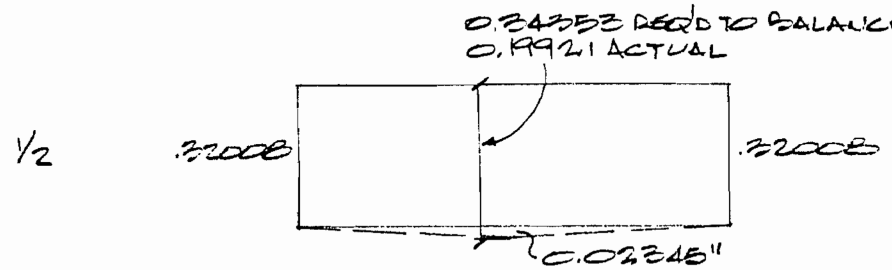
RIGIDITY ANALYSIS

MODEL 'C'

WITH 4-0 WALL AT EACH END—





LOAD TO CTR WALL PROBABLY ABOUT 59%

SPECIAL GARAGE FRONT WALL DETAILS

In designing small wood frame structures for lateral forces, engineers have frequently utilized the principle of rotation in the horizontal plane for calculating the resistance of structures with large openings at one end. Using the single-story wood frame two-car garage as an example, this principle states that all loads developed parallel to the walls containing the garage door opening will be taken by the rear wall. Since the center of gravity of the load is at the center of the depth of the garage, while resistance is at the very rear, the resulting rotational moment is to be resisted by the two exterior walls perpendicular to the load. This theory has failed to consider the fact that free-standing wood stud walls do not offer much resistance to loads perpendicular to them, when such load is placed at the top of the wall. Observations of damage caused by the San Fernando earthquake showed that while the diaphragms did rotate about the rear wall of the garage, the side walls were simply twisted once whatever wall adjacent to the garage doors had failed. While this type of damage was not general in the case of one-story garages, similar action played a large part in contributing to the rather extensive damage to split-level homes. In this latter case, the seismic loads developed are considerably greater than those encountered in one-story structures.

Reference 3 recommends that bracing for one-story garages be provided on either side of the garage door and that plywood sheathing be used at the rear wall of garages on the first floor of split-level homes. A question was raised as to whether detached garages should be considered in the Report. It was the expression of the Advisory Committee that they should be included. Calculations made for the Rigidity Analysis led to the conclusion that such bracing should also be supplied at the front of garages in split-level houses. Special bracing details are therefore presented for the wall adjacent to garage door openings for both one- and two-story conditions. The details themselves are designed in accordance with standard engineering principles and require little explanation.

Because of the difficulty many contractors have in placing hold-down anchors, a rigid connection is made between the garage door header and the 4 x posts rather than at the base of the wall. The seismic loads used for the development of the one-story detail are the maximum considered conceivable and are unlikely to be encountered in the field. A garage has been assumed to be 20' x 25' with masonry veneered walls and a heavy roof. The two-story design is not as conservative and utilizes Model C as a typical example. Wall loads are figured for exterior stucco and interior lath and plaster. Roofing has been assumed as 6 psf.

The design of rigid frames in wood is at best questionable. In addition, the deflections of these frames is high, thereby inferring that the diaphragm will interact with the frame and transfer much of the load to other walls. In the case of one-story structures, it was felt that the roof diaphragm would not be capable of a great deal of interaction (particularly in the case of detached garages) and the frame has therefore been treated as a shear wall for this condition. As inferred by the calculations, the deflection of the two-story frame under full design load is approximately 1/2-inch, while a diaphragm over a 24-foot deep garage will deflect approximately 1/3-inch when considered as a full cantilever. In order to not place too much reliance on the frame, Section 3.3B4 of the Report requires that when the Special Garage Front Wall Detail is used in conjunction with two-story construction, twice the seismic load developed by the two-story portion over the garage shall be taken by the rear wall of the garage. This requirement gives no credit for shear resistance to the Detail. The essential function of the Special Garage Front Wall Detail is to inhibit excessive rotation of the second-floor diaphragm, as well as to reduce damage adjacent to garage doors.

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SPECIAL GARAGE FRONT WALL DETAILS

ONE STORY GARAGE LOADS

USE 20'-0" WIDE x 25'-0" DEEP WITH TILE ROOF & 4" MASONRY VENEER WALLS AS WORST CASE.
ROOF OVERHANG = 3'-0"

REG - 14.0
SHTG - 2.0
FRMG - 2.0
18.0 PSF

P_{SEIS} = ROOF - 15.5 x 26 x 18 x .133 = 967
WALLS - 2 x 12.5 x 4 x 44 x .133 = 587
1554#

W/O VENEER - P_S = 967 + 25 x 4 x 12 x .133 = 1127#

WIND - USE 6:12 ROOF SLOPE W/ 15 PSF WIND

P_W = 9 x 12.5 x 15 = 1688# USE SEIS FOR DESIGN

TWO STORY GARAGE LOADS

USE MODEL 'C' AS TYPICAL CONDITION

USE AS LOADS - ROOF - 6 PSF
EXT. FIN. - STUCCO
INT. FIN. - PLASTER
CLG - "

ROOF - 10 PSF CLG - 10 PSF 2ND FLR - 17.5 PSF

SEIS - ROOF - 1.33 PSF CLG - 1.33 2ND 2.33
EXT - 1.50 3.00
INT. - 0.50 3.00
3.33 PSF 8.33 PSF
3.33
11.67 PSF

P_{SEIS} = 650 x 1.33 = 865 P_W = 3080#
254 x 11.67 = 2964
3829#

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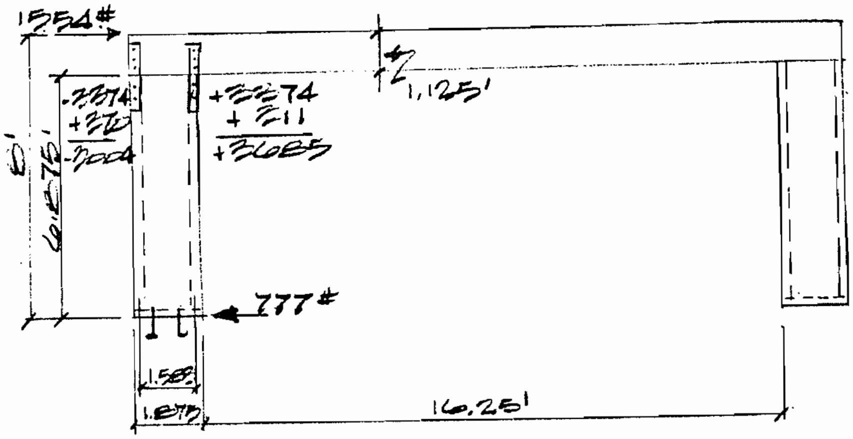
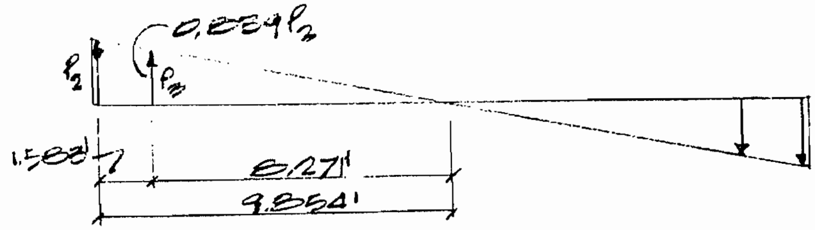
SPECIAL GARAGE FRONT WALL DETAILS

ONE STORY





$M_{PIER} = 777 \times 6.875 = 5342'\#$ $\quad f_1 = \frac{5341}{1.583} = 3374\#$

$v_{PIER} = \frac{777}{1.875} = 414.4\ \#/'$ $\quad$ 5/8" ST I - 8d @ 4,12

OR 1/2" ST II - 10d @ 4,12

$OTM = 1554 \times 8 = 12{,}432'\#$

$2[6.27(.839P_2) + 9.854P_2] = 12432$

$(6.941 + 9.854)P_2 = 6216 \quad P_2 = 370\#$

$P_3 = 311\#$

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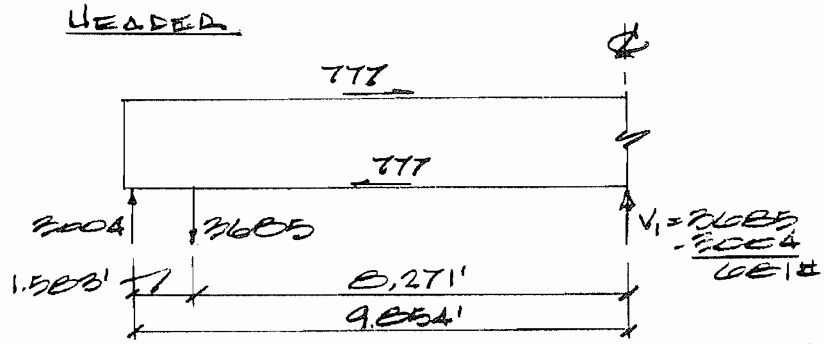
SPECIAL GARAGE FRONT WALL DETAILS

ONE STORY

HEADER



$M = 681 \times 8.271 \times 12 = 67{,}591''\#$ $S = \frac{67591}{1500 \times 1.33} = 33.80$

$A = \frac{1.5 \times 681}{95 \times 1.33} = 8.06\,\square''$ 4x14 OK

STRAP

$\frac{3685}{930 \times 1.33} = 2.97$ 3-3/4"φ BOLTS TO BM

$\frac{3685}{1470 \times 1.33} = 1.88$ 2-3/4"φ BOLTS TO POST

NET AREA $= 0.1875(1.75 - .8125) = 0.1758\,\square''$

$f_{STRAP} = \frac{3685}{0.1758 \times 1.33} = 15{,}762$ PSI 1 3/4 x 3/16 STRAP

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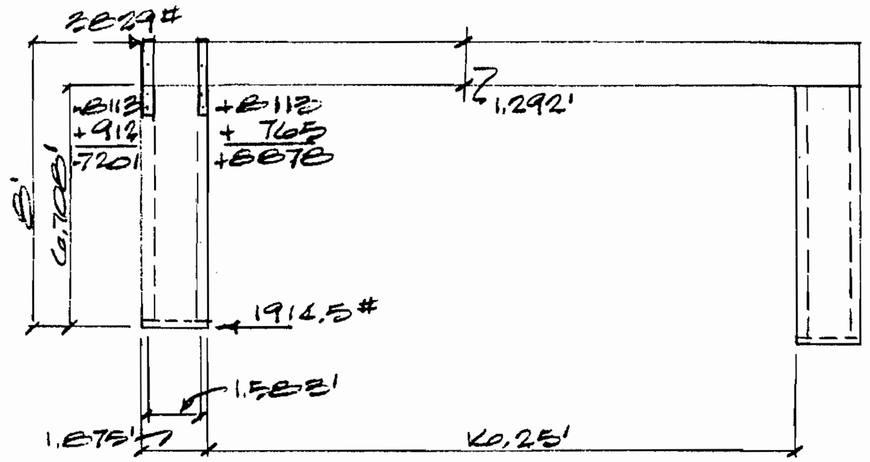
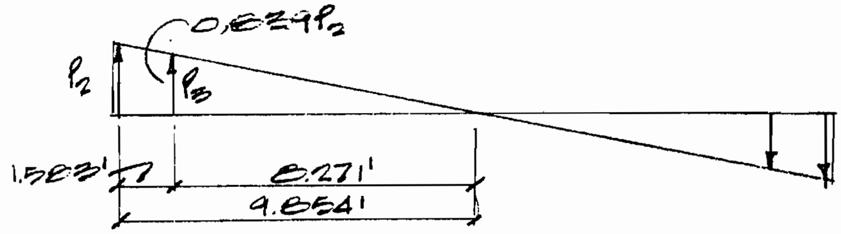
SPECIAL GARAGE FRONT WALL DETAILS

TWO STORY





$M_{PIER} = 1914.5 \times 6.708 = 12{,}843'\#$ $\quad P_1 = \frac{12843}{1.583} = 8113\#$

$v_{PIER} = \frac{1914.5}{1.875} = 1021\#/'$ $\quad$ ½" ST I EA SIDE - 10d@4,12

$OTM = 3829 \times 8 = 30{,}632'\#$

$2[8.27(.829P_2) + 9.854P_2 = 30{,}632'\#$

$(6.941 + 9.854)P_2 = 15{,}316 \qquad P_2 = 912\#$

$P_3 = 765\#$

ZONE 2 -

USE WIND LD — $v = \frac{3080}{2 \times 1.875} = 821.3\#/'$

½" ST II - 10d @4,12

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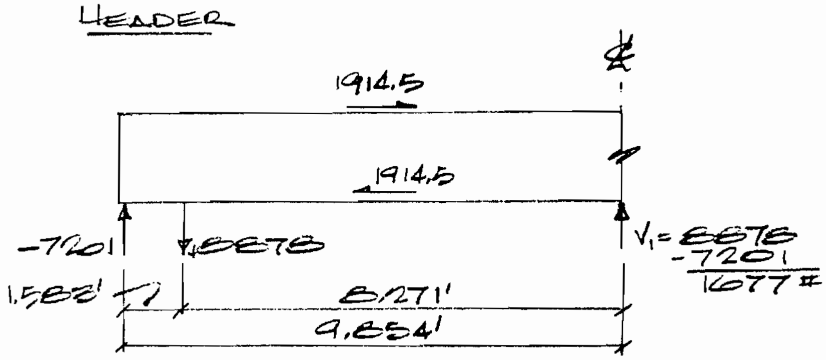
SPECIAL GARAGE FRONT WALL DETAILS

TWO STORY

HEADER



VERT LD/FT — USE SAME AS REPORT, WALL 'E', MODEL 'C' BUT WITH LOADS SHOWN ON PREV. PAGE.

LU: ROOF - 3x10 = 30
CLG - .67x10 = 7
2ND - .67x17.5 = 12
WALL - 10x20 = 200
249#/'

$M_{END} = 249 \times 16.25^2 \times 12/2 + 1677 \times 8.271 \times 12 = 232{,}197''\#$

$S = \frac{232{,}197}{1500 \times 1.33} = 116.10$

$A = \frac{1.5 \times 1677}{95 \times 1.33} = 19.91\square''$ 4x16 OK

STRAP

$\frac{8878}{2870 \times 1.25 \times 1.33} = 1.86$ 2-3/4"φ BOLTS TO POST

$\frac{8878}{385 \times 1.33} = 17.30''$ $\frac{17.3}{3.5} = 4.94''$ USE 3½"x3/8"x6'-5" PL
USE 4x6 POSTS

$f_{STRAP} = \frac{8878}{2 \times .1758 \times 1.33} = 18942\ psi$ 1¾x3/16 STRAP

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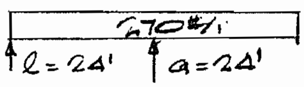
SPECIAL GARAGE FRONT WALL DETAILS

CONTINUE WITH MODEL 'C' WITH SAME LOAD OF 270#/1 (SEE RIGIDITY ANALYSIS), ASSUMING THE DETAIL APPLIES IN PLACE OF THE 40' SHEAR WALL, WITH 25-50-25 LOAD DISTRIBUTION, LOAD TO FRONT IS 3240# WHICH IS LESS THAN THE 3830# THE DETAIL IS DESIGNED FOR.

DIAPHRAGM RIGIDITY

ASSUME CANTILEVER OVER TWO SUPPORTS

$$\Delta_c = \frac{wa(12)^3}{24EI}(4a^2l - l^3 + 3a^3)$$



$$I = l = a \quad - \quad \Delta = \frac{6wa^4(12)^3}{24EI} = \frac{wa^4(12)^3}{4EI}$$

$$v = \frac{wa}{b} \qquad w = \frac{bv}{a} \; ; \quad I = A b^2 \left(\frac{12^2}{2}\right)$$

$$\Delta = \frac{bva^4(12)^3 2}{4aEAb^2(12)^2} = \frac{6va^3}{EAb} + \frac{va}{2Gt} + 0.188ae_n$$

$$v = \frac{270 \times 24}{21} = 308.6\#/1$$

$$\Delta = \frac{6 \times 308.6 \times 24^3}{1.65 \times 10^6 \times 5.25 \times 21} + \frac{308.6 \times 24}{2 \times 90{,}000 \times 0.5} + 0.188 \times 24 \times 0.030$$

$$= 0.14071 + 0.08229 + 0.13536 = 0.35836''$$

IF REAR SUPPORT IS ALSO CONSIDERED TO BE ZERO (TEETER-TOTTER EFFECT) THEN:

$$\Delta = \frac{0.14071}{2} + 0.08229 + 0.13536 = 0.28801''$$

INTERIOR AND REAR WALLS FOR RIGIDITY ANALYSIS

CHECK FRONT WALL DEFLECTION, SINCE DETAIL ACTS AS A RIGID FRAME, Δ WILL BE GREATER THAN THAT CALCULATED (ROTATION OF JOINT NOT CONSIDERED).

25-50-25 – $V_{FRONT} = 3240\#$ $\quad v = \frac{3240}{2 \times 1.875} = 864\#/1$

BOLT SLIP $\Delta = \frac{6.7083}{1.6458} \times 0.0625 = 0.25475''$

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SPECIAL GARAGE FRONT WALL DETAILS

INTERIOR AND REAR WALLS PER RIGIDITY ANALYSIS

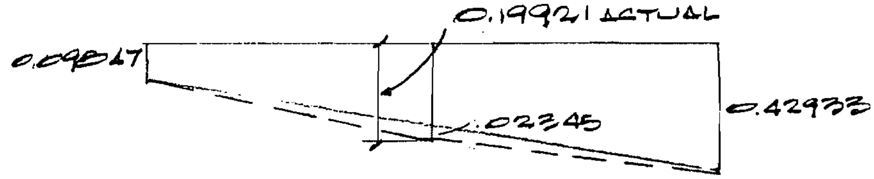
25-50-25

$$\Delta = \frac{8 \times 864 \times 6.7083^3}{1.8 \times 10^6 \times 1.1 \times 19.25 \times 1.875} + \frac{432 \times 6.7083}{75000 \times 0.5} + 0.376 \times 6.7083 \times 0.027$$

$+0.25475 = 0.02920 + 0.07728 + 0.06810 + 0.25475$

$= 0.42933''$

0.28735 REQ'D TO BALANCE
0.19921 ACTUAL



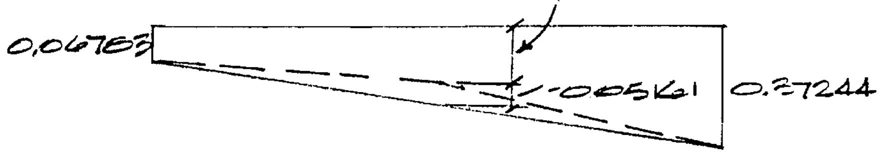
18.75-62.5-18.75

$V_{EQ} = 12960 \times 3/20 = 2430\#$ $v = 648\#/1$

$\Delta = 0.02190 + 0.05796 + 0.03783 + 0.25475$ (.015)

$= 0.37244''$

0.16852 REQ'D TO BALANCE
0.26617 ACTUAL



ESTIMATED LOAD TO CENTER WALL = 56%

CHECK WITH SOLID 21' WALL AT REAR OF GARAGE

25-50-25 — $v = \frac{6480}{21} = 308.6\#/1$ - 3/8" PLY, 8d@4

WITH STRAP H.D. -

$\Delta = \frac{8}{21} \times 0.025 + 0.00347 + 0.07315 + 0.05114 = 0.13728''$ (.017)

18.75-62.5-18.75 — $v = \frac{8100}{21} = 385.7\#/1$ - 3/8" PLY, 8d@4

WITH L H.D. -

$\Delta = \frac{8}{21} \times 0.0625 + 0.00434 + 0.09143 + 0.07821 = 0.19778$ (.031)

ESTIMATED LOAD TO CENTER WALL = 60.5%

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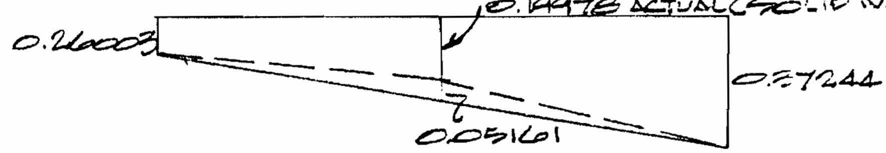
SPECIAL GARAGE FRONT WALL DETAILS

4'-0" LONG SHEAR WALL AT REAR - 5/8 LD TO CTR WALL

SEE RIGIDITY ANALYSIS FOR Δ CALCS.

0.22462 REQ'D TO BALANCE
0.22617 ACTUAL (AS SHOWN)
0.19978 ACTUAL (SOLID WALL)



WITH 21' SOLID INTERIOR WALL, ESTIMATED LOAD REQUIRED TO BALANCE APPROACHES 70%

WITH 70% TO CENTER - $V_{ENDS} = 12960 \times 0.15 = 1944\#$

$v_{4'} = \frac{1944}{4} = 486\#/'$

DEFLECTION OF INTERIOR WALLS (Development of Table 3.1 in the Report)

Whether or not an interior wall should be a designed shear wall depends on many factors. The Methodology gives various "rules" for making that determination. For some situations it is essential that interior walls be designed as shear walls to ensure simple structure integrity. Examples of where interior shear walls are required are: (1) to preclude diaphragm ratios greater than 1-1/2:1, (2) at diaphragm openings such as at stair-wells, and (3) for walls extending to the ceiling. The Methodology simply states that interior shear walls are required at these locations. For the vast majority of cases, however, the question of whether or not an interior wall should be a designed shear wall is resolved through consideration of the relative rigidities (force and deformation characteristics) of the structure system--including the designed shear walls, nondesigned walls, and diaphragm.

Interior walls not designed as shear walls will be carried along by the diaphragm to which they are attached when the diaphragm moves. Their movement will therefore be a function of the deflection of the shear wall to either side plus whatever amount the diaphragm deflects at the point where the interior wall is located. Short nondesigned walls offering little in the way of resistance will deflect more than the designed shear walls. When the wall is short and nondesigned, this deflection can be tolerated. It would seem desirable, in fact, that such walls have their shear resisting material attached with minimal nailing so that the deflection can be attained with the least amount of load.

Obviously, every wall attached to a diaphragm above will attempt to take load. Short nondesigned walls will therefore absorb a small amount of the load intended for designed shear walls on either side (thereby slightly increasing the factor of safety in the designed walls) and will affect diaphragm characteristics as well. The dilemma regarding the rigidity required for the proper distribution of load versus the limberness required to prevent damage resolves itself down to the question of the amount of deflection that can be tolerated by an interior wall and the amount of load the wall will attempt to absorb in achieving this deflection. When this load becomes a substantial percentage of the total load, it seems reasonable to require that the wall be designed.

The Rigidity Analysis has indicated the variable nature of the load supported by interior walls and the importance of providing adequate rigidity when such a wall is used. Observation of earthquake caused damage, particularly to two-story homes, led to the conclusion that performance would have been substantially better if more interior walls were designed. Based upon this consideration alone, it would seem justifiable to arbitrarily choose a ratio and simply state that an interior wall must be 1.5 times as long as the shortest effective length of exterior wall, for instance, before it must be considered as acting and therefore be required to be a designed shear wall. The other side of the coin, however, is the consideration that the purpose of the Methodology is to mimimize damage. One of the functions of successful shear wall design is to restrict total deflection at all points in the structure to a degree that can be tolerated by the finish material.

Once the determination based on observations of damage was made that significant interior walls should be designed, the question of minimum length required next needed to be answered. It was recognized that this would be dependent upon the deflection of the exterior designed shear walls and upon diaphragm deflection which in turn, for the average two-story house, is a function of the diaphragm ratio. It was decided that diaphragms of 800 to 1000 square feet represented average two-story construction and diaphragms of various ratios were determined within this range. The calculations at the end of this section are provided for the purpose of illustrating the relative deformation and load carrying characteristics for various assumptions of designed and nondesigned interior walls, wall lengths, and diaphragm ratios. Eight-foot shear walls were assumed at each edge of the diaphragm and loads to these walls obtained by assuming wind load to the second floor only and a seismic load using "heavyweight" materials.

To obtain an indication of how interior wall length might vary (as a function of exterior wall length) as diaphragm ratio varies, the deflection of the 8-foot interior wall under 50 percent of load, was divided by the deflection of an 8-foot long interior wall under the same load plus the diaphragm deflection. The assumption was that this would yield a percentage which, when multiplied by the length of exterior wall, would yield the length of the interior wall which would have, under 50 percent of load, the same deflection as the exterior wall plus the diaphragm. Results of these analyses are summarized on the bottom of page 4 of the calculations at the end of this section.

The analyses given in pages 46 through 48 of the calculations at the end of this section show the load distributions for various assumed cases of wall material and interior wall lengths. Case I shows the equal distribution of forces between the three walls when the walls are assumed to have equal rigidity. Case II indicates that little change in load distribution takes place when the interior wall is 90 percent as long as the exterior wall (for a 3/4:1 diaphragm ratio). Case III considers a 1-1/2:1 diaphragm ratio and shows that a nearly equal distribution of forces occurs when the interior wall is 79 percent as long as the exterior wall. Case IV shows that when the rigidity of the interior wall is reduced, the load carried by that wall is substantially reduced. Conversely, Case V shows that when the rigidity of the interior wall is substantially increased, as it would be if designed, the load to the interior wall is substantially increased.

The percentages stated in Table 3.1 of the Report were established arbitrarily, but are based on the field observation that significant interior walls should be designed and the calculations given in this section.

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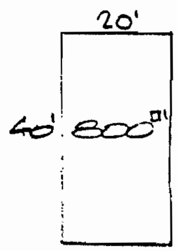
DEFLECTION OF INTERIOR WALLS

1. ASSUME 2^{ND} FLOOR DIAPHRAGMS OF 800 TO 1000 SQUARE FEET AS REPRESENTING TYPICAL 2^{ND} FLOOR
2. SINCE FORMULAE ARE AVAILABLE FOR PLYWOOD DEFLECTION ONLY, ASSUME ALL SHEAR WALLS WITH PLYWOOD
3. ASSUME STIFFNESS PROPORTIONAL TO LENGTH OF WALL.

1/2 TO 1



ASSUME 15 PSF WIND LOAD AND 8' LONG WALL AT 40' SIDE

$$w = 9 \times 15 = 135^{\#}/'$$

$$\Delta_{DIA} = \frac{5vL^3}{8EAb} + \frac{vL}{4Gt} + 0.094\,L\,e_n$$

$$w_{DIA} = \frac{10 \times 135}{40} = 33.75^{\#}/'$$

$$\Delta_D = \frac{5 \times 33.75 \times 20^3}{8 \times 1.65 \times 10^6 \times 5.25 \times 40} + \frac{33.75 \times 20}{4 \times 90{,}000 \times .5} + .094 \times 20 \times .001$$

$$= 0.00049 + 0.00375 + 0.00188 = 0.00612'$$

$$\Delta_{WALL} = \frac{8vh^3}{EAb} + \frac{vh}{Gt} + 0.376\,h\,e_n$$

$$V = 1350^{\#} \quad v = \frac{1350}{8} = 168.75^{\#}/'$$

$$\Delta_W = \frac{8 \times 168.75 \times 8^3}{1.65 \times 10^6 \times 10.5 \times 8} + \frac{168.75 \times 8}{90{,}000 \times .375} + 0.376 \times 8 \times 0.014$$

$$= 0.00499 + 0.04000 + 0.04211 = 0.08710$$

TOTAL Δ @ CENTER $= 0.08710 + 0.00612 = 0.09322$

$$\frac{0.08710}{0.09322} = 93.4\%$$

IF Δ WERE COMPLETELY PROPORTIONAL TO LENGTH, A WALL AT CENTER 93.4% AS LONG AS WALL AT END WOULD DEFLECT THE SAME. SHORTER WALLS WOULD DEFLECT MORE AND THEREFORE "RELIEVE" THEMSELVES OF SOME LOAD WHICH WOULD HAVE TO THEN BE TAKEN BY THE EXTERIOR WALLS

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DEFLECTION OF INTERIOR WALLS

<u>1/2 TO 1</u>

CHECK FOR SEISMIC LOAD VS WIND LOAD
USE 'HEAVYWEIGHT' LOAD W/O VENEER (SEE NLG SCHED)

$w = 40 \times 11.33 = 453.3^{\#/'}$ $V = 453.3 \times 10 = 4533^{\#}$

$v_D = \frac{4533}{40} = 113.3^{\#/'}$ $v_W = \frac{4533}{8} = 566.6^{\#/'}$

$\Delta_D = 0.00163 + 0.01259 + 0.01316(.007) = 0.02738''$

ASSUME 3/8" PLYWD - 8d @ 2 1/2 FOR WALL

$\Delta_W = 0.01674 + 0.13431 + 0.06918(0.023) = 0.22023$

TOTAL $\Delta = 0.22023 + 0.02738 = 0.24761''$

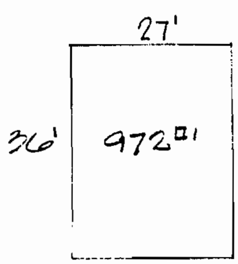
$\frac{.22023}{.24761} = 88.9\%$

CHECK Δ OF WALL 90% × 8' LONG = 7.2'; $v = \frac{4533}{7.2} = 629.6$

$\Delta_W = 0.02067 + 0.14924 + 0.08422(.02) = 0.25412$

$.25412 \cong 0.24761$ (2.6% VARIANCE - SAME NLG)

<u>3/4 TO 1</u>



MAKE SAME WALL LENGTH & LOAD ASSUMPTIONS THROUGHOUT

$V = 135 \times 13.5 = 1822.5$

$v_D = \frac{1822.5}{36} = 50.6^{\#/'}$ $v_W = \frac{1822.5}{8} = 227.8^{\#/'}$

$\Delta_D = 0.00200 + 0.00759 + 0.00431(.0017)$
$= 0.01390''$

$\Delta_W = 0.00673 + 0.05400 + 0.06467(.0215)$
$= 0.12540''$

$.12540 + .01390 = 0.13930$ $\frac{.12540}{.13930} = 90.0\%$

DEFLECTION OF INTERIOR WALLS

3/4 TO 1

$w_{SEIS} = 36 \times 11.33 = 408 \#/1 \quad V = 408 \times 13.5 = 5508 \#$

$v_D = \frac{5508}{36} = 153 \#/1 \quad v_W = \frac{5508}{8} = 688.5 \#/1$

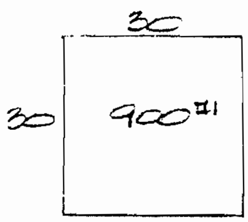
$\Delta_D = 0.00604 + 0.02295 + 0.02665 = 0.05564$

$\Delta_W = 0.02035 + 0.12240 + 0.09024(.030) = 0.23299$

$.23299 + .05564 = 0.28863 \quad \frac{.23299}{.28863} = 80.7\%$

THIS IS AT A VERY HIGH SHEAR IN WALL WITH 1/2" PLYWD.

1 TO 1



$V = 135 \times 15 = 2025 \#$

$v_D = \frac{2025}{30} = 67.5 \#/1 \quad v_W = \frac{2025}{8} = 253.1 \#/1$

$\Delta_D = 0.00438 + 0.01125 + 0.00705 = 0.02268''$

$\Delta_W = 0.00748 + 0.05999 + 0.07670(0.55) = 0.14417''$

$.14417 + .02268 = 0.16685 \quad \frac{.14417}{.16685} = 86.4\%$

$w_{SEIS} = 30 \times 11.33 = 340 \#/1 \quad V = 340 \times 15 = 5100 \#$

$v_D = \frac{5100}{30} = 170 \#/1 \quad v_W = \frac{5100}{8} = 637.5 \#/1$

$\Delta_D = 0.01104 + 0.02833 + 0.03666(.013) = 0.07603$

$\Delta_W = 0.01884 + 0.15111 + 0.05715(.019) = 0.22710$

$.22710 + .07603 = 0.30313 \quad \frac{.22710}{.30313} = 74.9\%$

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Deflection of Interior Walls

1½ to 1

37.5

25   937.5□'

$V = 135 \times 18.75 = 2531^{\#}$

$v_D = \frac{2531}{25} = 101.3^{\#}/1 \quad v_W = \frac{2531}{8} = 316.4^{\#}/1$

$\Delta_D = 0.01542 + 0.02110 + 0.02115(.006)$
$= 0.05767$

$\Delta_W = 0.00935 + 0.07500 + 0.12032(.04)$
$= 0.20467$

With 8d @ 4 -- + -- + 0.05715(.019) = 0.14150

$.20467 + .05767 = 0.26234 \quad \frac{.20467}{.26234} = 78.0\%$

$.14150 + .05767 = 0.19917 \quad \frac{.14150}{.19917} = 71.0\%$

$w_{SEIS} = 25 \times 11.33 = 283.3^{\#}/1 \quad V = 283.3 \times 18.75 = 5312.5^{\#}$

$v_D = \frac{5312.5}{25} = 212.5^{\#}/1 \quad v_W = \frac{5312.5}{8} = 664.1^{\#}/1$

$\Delta_D = 0.03234 + 0.04427 + 0.06698(.019) = 0.14359''$

$\Delta_W = 0.01963 + 0.15742 + 0.06166 = 0.23871''$

$.23871 + .14359 = 0.38230 \quad \frac{.23871}{.38230} = 62.4\%$

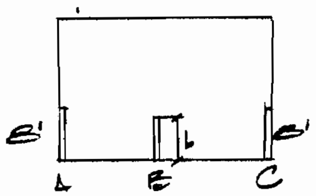
Design Criteria

Diaphragm Ratio	Wind			Seismic		
	Δ_D	Δ_W	%	Δ_D	Δ_W	%
½ to 1	.00612	.08710	93.4	.02738	.22023	88.9
¾ to 1	.01390	.12540	90.0	.05564	.23299	80.7
1 to 1	.02268	.14417	86.4	.07603	.22710	74.9
1½ to 1	.05767	.20467	78.0	.14359	.23871	62.4

DEFLECTION OF INTERIOR WALLS

AS AN IDEALIZED CONDITION, ASSUME THE FOLLOWING:

1. DEFLECTION IS PROPORTIONAL TO LOAD WHEN OTHER FACTORS (HEIGHT, LENGTH, MATERIAL, ETC.) ARE CONSTANT
2. DEFLECTION IS INVERSELY PROPORTIONAL TO LENGTH WHEN LOAD AND OTHER FACTORS ARE CONSTANT.
3. DEFLECTION OF ALL MATERIALS AT THEIR RATED SHEAR PER FOOT IS EQUAL.
4. FOR 25-50-25 DISTRIBUTION, $\Delta_{DIAPHRAGM} = 0$



START WITH 3/4:1 ANALYSIS SINCE LOAD CAUSES $\Delta_{WALL} = 0.125''$, LOAD IS 3645# TOTAL.

WITH NO INTERIOR WALL:

$\Delta_A = \Delta_C = 0.125''$ $\Delta_{DIA} = 0.014''$

WITH AN INTERIOR WALL PRESENT:

IF INTERIOR WALL TOOK 50% OF LOAD THEN:

$V_A = V_C = 911.25\#$ $V_B = 1822.5\#$ $\Delta_{DIA} = 0$

FOR A LESSER OR GREATER LOAD TO THE INTERIOR WALL, THE LOAD TO THE WALL CAN BE CONSIDERED A NEGATIVE LOAD ON THE DIAPHRAGM AND:

$$\Delta_{DIA} = 0.014'' - \frac{V_B}{1822.5} \times 0.014''$$

CASE I: L = 8' – ALL WALLS 3/8" PLYWD – 8d@6,6,12

$$\Delta_A = \frac{V_A}{1822.5}(0.125) \; ; \quad \Delta_B = \Delta_A + \Delta_{DIA} = \frac{V_B}{1822.5}(.125)$$

SUBSTITUTING:

$$\Delta_B = \frac{V_A}{1822.5}(0.125) + 0.014 - \frac{V_B}{1822.5}(.014) = \frac{V_B}{1822.5}(0.125)$$

BUT $2V_A + V_B = 3645\#$; $V_A = \frac{3645 - V_B}{2}$

$$\frac{3645 - V_B}{2}(0.125) + 25.515 - 0.014V_B = 0.125V_B$$

$$227.8125 + 25.515 = (.125 + .0625 + .014)V_B$$

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DEFLECTION OF INTERIOR WALLS

CASE I (CONT)

$V_B = \frac{253.3275}{0.2015} = 1257.2\#$ $\quad v_B = 157.2\#/'$ $\quad$ % TOT. LD = 34.5%

$V_A = 1193.9\#$ $\quad v_B = 149.2\#/'$ $\quad$ 32.75% OF TOTAL LD.

$\Delta_{DIA} = .014 - \frac{1257.2}{1822.5} \times .014 = 0.0043''$

CASE II

IF $L = \frac{0.125}{.125+.014} \times 8 = 7.2'$ $\quad$ THEN $\Delta_B = \frac{8}{7.2}\Delta_B = 1.111\Delta_B$

$253.3275 = (1.111 \times .125 + .0625 + .014)V_B = 0.2154 V_B$

$V_B = 1176.1\#$ $\quad v_B = 163.3\#/'$ $\quad$ 32.3% OF TOTAL LD.

$V_A = 1234.5\#$ $\quad v_A = 154.3\#/'$ $\quad$ 33.9% " " "

$\Delta_{DIA} = .014 - \frac{1176.1}{1822.5} \times .014 = 0.0050''$

CASE III

IF DIAPHRAGM IS CONSIDERED TO BE THE SAME AREA BUT WITH A RATIO OF 1½:1, THEN HORIZONTAL LOAD WOULD BE THE SAME BUT Δ_{DIA} WOULD BE LARGER.

DIAPHRAGM WOULD THEN BE $\left(\frac{972}{1.5}\right)^{1/2} = 25.46' (\times 38.18')$

AND w WOULD BE $\frac{3645}{38.18} = 95.5\#/'$; $v_{DIA} = 71.6\#/'$

$\Delta_{DIA} = 0.01129 + 0.01519 + 0.00718 = \overset{(COMP)}{0.03366''}$

$\frac{.125}{.125+.034} = 0.786\ (\times 8 = 6.29')$

IF WALL IS 6.29' LONG − $\Delta_B = 1.272\,\Delta_B = 1.272 \times .125 = 0.159$

$\frac{3645 - V_B}{2}(.125) + 1822.5 \times .034 - .034 V_B = 0.159 V_B$

$227.8125 + 61.965 = (.159 + .0625 + .034)V_B$

$V_B = \frac{289.7775}{0.2555} = 1134.2\#$ (3.7% VARIANCE FROM CASE II)

$\Delta_{DIA} = .034 - \frac{1134.2}{1822.5} \times .034 = 0.013''$

THUS, AS DIAPHRAGM RATIO INCREASES, INTERIOR WALL LENGTH REQ'D TO MAINTAIN CONSTANT LOAD TO THIS WALL DECREASES.

DEFLECTION OF INTERIOR WALLS

CASES I THRU III ASSUME ALL 3 WALLS WITH PLYWOOD SHEATHING. IF INTERIOR WALL WERE GYP. BD., IN ACCORDANCE WITH ASSUMPTION 3, THE GYP. BD. CAN BE ASSUMED TO DEFLECT $\frac{260}{100} = 2.60$ TIMES AS MUCH AS PLYWOOD.

CASE IV

AGAIN ASSUME $L = 7.2'$ BUT WITH GYP. BD. ONE SIDE

THEN $\Delta_D = 1.111 \times 2.60 = 2.889\,\Delta_B\ (\times .125 = 0.361)$

FROM CASE II:

$255.3275 = (0.361 + .0625 + .014)V_D = 0.4375\,V_D$

$V_D = 579.0\#$ $\quad v_D = 80.4\#/1$ $\quad$ 15.9% OF TOTAL LD.

$V_A = 1533\#$ $\quad v_A = 191.6\#/1$ $\quad$ 42.1% " " "

$\Delta_{DIA} = 0.00096''$

BUT IF MATERIALS ARE REVERSED (INTERIOR WALL IS DESIGNED), THEN INT. WALL WOULD BE DESIGNED FOR $3645 \times 5/8 = 2278\#$ OR $316.4\#/1$ FOR 7.2' LONG WALL. THIS WOULD REQUIRE 8d@4,6,12 AND GYP. BD. WOULD DEFLECT $\frac{380}{100} = 3.8$ TIMES AS MUCH. THIS IS THE SAME AS SAYING!

CASE V

FOR $L = 7.2'$ $\quad \Delta_D = \frac{1.111}{3.8} = 0.292\,\Delta_B\ (\times .125 = 0.037)$.

$255.3275 = (.037 + .0625 + .014)V_D = 0.1135\,V_D$

$V_D = 2252\#$ $\quad v_D = 310\#/1$ $\quad$ 61.2% OF TOTAL LOAD

$V_A = 706.5\#/1$ $\quad v_A = 88.3\#/1$ $\quad$ 19.4% " " "

$\Delta_{DIA} = 0.00031''$

ALTHOUGH THE CALCULATIONS ARE APPROXIMATIONS, IT SEEMS OBVIOUS THAT DESIGNED INTERIOR WALL LENGTH SHOULD BE A FUNCTION OF EXTERIOR WALL LENGTH AND DIAPHRAGM RATIO. LOAD WILL FLOW TO THE MORE RIGID WALL(S) AND IT APPEARS THAT THE LENGTHS CHOSEN WILL NOT CAUSE SEVERE OVER-STRESS OF NON-DESIGNED WALLS.

WALL LOADS (Determining Loads for Wall Load Tables)

The methods proposed in the Report for the determination of seismic loads are, for the most part, consistent with standard structural design procedures. In certain instances, some judgment is required by the user of the Methodology to determine tributary areas, but the design requirements are adequately specific in this regard to leave little room for major judgmental error. In determining seismic loads contributed by exterior and interior walls, the structural designer frequently employs shortcuts which were considered as requiring more judgment than it was felt wise to leave to the discretion of the user of the Methodology.

It is general practice for structural engineers to ignore door and window openings and consider the walls of small structures as solid. Some engineers consider only walls perpendicular to the load in determining the seismic load, using the reasoning that the parallel walls support themselves and that the shear wall being designed contributes a very small percentage of the total load and therefore may be neglected. In such cases, the engineer often simply views the plan and determines that an "average" of 2, 2-1/2, or 3 interior walls run perpendicular to the load across the structure and then calculates seismic load for the "average" number of walls. Other engineers use a uniform partition load assuming it accounts for all load from both interior and exterior walls. The partition load used in this instance is frequently lighter than that which would be required by some building review agencies for the design of offices or other similar commercial structures. Again, the reasoning is probably that walls parallel to the load carry themselves and may therefore be ignored in "averaging" wall load. Obviously there are innumerable variations on the techniques described. Such reasoning has its justification in that it reflects the way in which houses respond to earthquake generated forces.

As mentioned above, it was felt that it would be too difficult to require persons untrained in engineering to make the judgments necessary to determine seismic loads for interior and exterior walls. An attempt was therefore made to determine if houses damaged in the San Fernando earthquake bore any similarities with respect to the amount of wall present, expressed as a percentage of floor area. For this purpose, Models A, B and C in the Report are used to represent the three broad categories of housing studied. Although not shown in the calculations at the end of this section, the findings have been cross-checked against several other houses and found to be fairly reliable. The variation between larger homes with larger rooms and smaller homes with smaller rooms is not as great as might be expected. The method proposed is more conservative for large houses with large rooms than it is liberal for houses with many small rooms.

In determining interior wall lengths, all walls were scaled ignoring normal openings, but wardrobe doors and other wide openings were totally omitted. Since dimensions were determined by scaling, the lengths indicated in the calculations are approximate. Certain rules had to accompany the method of determining wall loads. Thus, for example, the designer is told that interior wall loads need not be considered for the area of attached garages in one-story construction but garages at the first floor of two-story construction must be considered as a room, i.e., as though interior walls were present. This latter requirement is conservative but it would have required still another table for this special case had this assumption not been made. The first replacement stated is in keeping with the decision to consider Model A without the attached garage in determining the ratio of wall to floor area. This was done because this model could conceivably be constructed with a detached garage.

As shown by the tabulation on the bottom of the second page of the calculations, the ratio of interior wall to floor area is remarkably constant with the exception of the mid-level of Model C. Although most such mid-levels contain the kind of rooms shown in the example, it is possible to conceive of floor plans requiring much more wall at this level. The assumption that one square foot of interior wall is present for each square foot of floor area was therefore felt to be a reasonable approximation, although it was recognized that homes with smaller rooms will probably exceed this value somewhat.

In the case of exterior walls, only the mid-level of Model C has less than one square foot of wall per square foot of floor area. In this case, the wall common to the two portions of the structure has not been included in the mid-level exterior wall length. The ratio of exterior wall to floor area is largely dependent upon the size of the house. It was determined to use 1-1/8 sq. ft. of wall area per square foot of floor area on the basis that houses as small as 825 sq. ft. would fall within this ratio, that wings on larger homes would increase exterior wall length faster than floor area increased and, finally, to somewhat compensate for the smaller or many-roomed average-size house having more interior wall than would be shown by the ratio assumed for those walls.

At the first floor of two story houses, a lesser interior wall ratio seems justified in that two-story houses tend to be larger than one-story, and most of the smaller rooms are congregrated on the second floor. The ratio of interior walls on the first floor of Model C is especially low since the garage area is included and also because, in this particular model, a large room was located behind the garage. When these houses were actually built, some of the models were similar to the floor plan shown while others had the large room behind the garage sub-divided into a bedroom or study and a smaller family room. It was therefore felt that the amount of wall in this model could be as much as double that shown on the example in the Report. The ratios found were decreased slightly since two-story plans tend to be more rectangular and the more conservative exterior wall load considered is three times the amount required to be used in one-story construction (upper 4-feet of wall to roof and 8-feet to second floor). Exterior wall loads for two-story construction will therefore tend to be somewhat larger than would be determined from engineering calculations.

In considering wall loads to be used, it was felt desirable to approximate the assumptions made in a structural engineering office such that undue penalty was not placed upon designs made using the Report. Although the assumptions could not be based upon a single floor plan, and therefore needed to be somewhat conservative, they are nevertheless founded on a rational basis. In one-story houses or the top floor of two-story houses, only the top half of interior and exterior walls load the roof or ceiling diaphragms. The ratios of wall to floor area can therefore be reduced by one-half for any design. It was also assumed that the wall lengths in each direction were equal (not necessarily true) and that the walls parallel to the load would be self-supporting and therefore transfer no load to the shear walls, and that, in addition, they would help support perpendicular walls through the action of the ceiling "diaphragms." In order to be conservative it was assumed that walls perpendicular to the load falling within the center half of the house were supported by these interior, nondesigned parallel walls.

The assumption of walls parallel with the load supporting themselves allows another 50 percent reduction in the load to the shear walls. One-half the length times one-half the width of the main body of the house (or the portion between parallel shear walls) results in one-fourth the area assumed to have walls perpendicular to the load supported by the non-designed walls. This allows a further reduction to three-fourths of the previously reduced interior wall area contributing load to shear walls. The total reduction is therefore 1 x 1/2 x 1/2 x 3/4 = 3/16 of interior wall load actually affecting shear walls. In other words, 0.1875 sq. ft. of interior wall is assumed as contributing seismic load to shear walls per square foot of floor area. It should be noted that the 3/4 reduction is not applied to ceiling load. It was reasoned that this consideration would be unduly confusing and that the additional load generated would tend to compensate for the fact that total interior wall length is not 50 percent in each direction.

When second-floor diaphragms were considered, it was necessary to compensate for the load not considered acting at the roof level since walls parallel with the load would still contribute their reaction to the second-floor diaphragm together with all perpendicular walls. This load is not considered at the roof level and therefore has to be added to the wall load directly acting on the second-floor diaphragm. While the reasoning seems somewhat complex, the results are that one-story houses or the top level of two-story houses are not unduly overloaded when using the methodology, while total load for the first floor of two-story dwellings is not undercompensated.

When interior walls extend to the underside of the roof framing, such as occurs in houses with flat roofs, with sloping ceilings applied directly under the framing or with houses with exposed framing, many of the interior walls must be considered as shear walls but the load to each such wall is usually quite low. In this instance, the reductions in interior wall load affecting the roof diaphragm are not applicable with the exception that only the top half of each wall loads the diaphragm. It was therefore necessary to provide additional tables in the Report for houses of this configuration utilizing 1/2 sq. ft. per square foot of floor area as the wall load affecting the roof. When second-floor diaphragms are present, obviously the other 1/2 sq. ft. is taken by the floor and therefore the same table used for the roof may be used for that portion of the load emanating from the walls above the second floor. In this instance, the second-floor wall load table in the Report lists loads from below the second floor only, and refers to the roof table for the remainder of the load.

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WALL LOADS (DETERMINE LOADS FOR WALL LOAD TABLES)

USE MODELS A, B & C AS 'AVERAGE'

MODEL 'A'

MAIN WING –

INT WALL LENGTH = 114'

INT WALL AREA = 114×8 = 912□'

AREA W/O GARAGE = 37×27.5 = 1017.5
GARAGE = 20×27.5 = 550
TOTAL – 1567.5□'

INT WALL A/FLR A W/O GAR $= \frac{912}{1017.5} = 0.8963$

INT WALL A/TOTAL FLR A $= \frac{912}{1567.5} = 0.5818$

EXT WALL L W/O GARAGE = 129' A = 1032□'

EXT WALL L W GARAGE = 169' A = 1352□'

W/O GARAGE – $\frac{\text{WALL A}}{\text{FLR A}} = \frac{1032}{1017.5} = 1.0143$

W GARAGE – $= \frac{1352}{1567.5} = 0.8625$

MODEL 'B'

2ND FLR –

INT WALL L = 122' A = 122×8 = 976□'

FLR A = 28×36 = 1008□' $\frac{976}{1008} = 0.9683$

EXT WALL L = 128' A = 128×8 = 1024□'
$\frac{1024}{1008} = 1.0159$

1ST FLR –

INT WALL L = 87' A = 87×8 = 696
$\frac{696}{1008} = 0.6905$

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WALL LOADS (CONT)

MODEL C

2ND FLR –

INT WALL L = 122.5' A = 122.5×8 = 980$^{\square'}$

FLR A = 21×48 = 1008$^{\square'}$ $\frac{980}{1008} = 0.9722$

EXT WALL L = 138' A = 138×8 = 1104$^{\square'}$

$\frac{1104}{1008} = 1.0952$

1ST FLR –

(THIS AREA COULD HAVE MORE INT. WALLS IF ROOM BEHIND GARAGE WERE DIVIDED)

INT WALL L = 43.5' A = 43.5×8 = 348$^{\square'}$

$\frac{348}{1008} = 0.3452$

MID-LEVEL

INT WALL L = 41' A = 41×8 = 328$^{\square'}$

FLR A = 31×26 – 8×6 = 806 – 48 = 758$^{\square'}$

$\frac{328}{758} = 0.4327$

EXT. WALL L = 94' A = 94×8 = 752$^{\square'}$

$\frac{752}{758} = 0.9921$

WALL TO FLOOR AREA RATIOS FOR LOADS TO ROOF

MODEL	INT RATIO	EXT RATIO
A	0.8963	1.0143
B (2ND)	0.9683	1.0159
C (2ND)	0.9722	1.0952
C (MID)	0.4327	0.9921
	USE 1.0	USE 1.125

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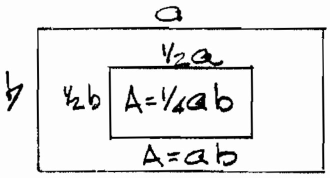
WALL LOADS (CON'T)

LOADS TO 2ND FLR FROM WALLS BELOW

MODEL	NUT RATIO
B	0.6905
C	0.3452 (x2 = 0.6904)

$\boxed{\text{USE } 0.625}$ (SLIGHTLY LIBERAL)

LOADS TO ROOF



ASSUME:
1. 1/2 OF WALL HEIGHT GOES TO ROOF
2. WALL LENGTH IN EACH DIRECTION IS EQUAL
3. WALLS PARALLEL TO LOAD CARRY THEMSELVES
4. WALLS PERPENDICULAR TO LOAD IN INTERIOR HALF OF STRUCTURE ARE SUPPORTED BY NON-DESIGNED WALLS PARALLEL TO THE LOAD

THEN INTERIOR WALL AREA TO BE CONSIDERED IS

1 SF/SF x 1/2 OF WALL HEIGHT x 1/2 OF WALL LENGTH
x 3/4 OF FLR. AREA = $1 \times 1/2 \times 1/2 \times 3/4 = 0.1875$ SF/SF

FOR WALL WT = 10 PSF –

$10 \times .133 \times .1875 = \underline{0.25 \text{ PSF}}$

BUT IF ALL WALLS EXTEND TO ROOF (FLAT ROOF OR SLOPING CLG) THEN ONLY ASSUMPTION #1 IS VALID SINCE MOST INTERIOR WALLS PARALLEL TO LOAD WILL THEN BE SHEAR WALLS

THEN FOR 10 PSF –

$10 \times .133 \times 1/2 = \underline{0.667 \text{ PSF}}$

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WALL LOADS

LOADS TO ROOF (CON'T)

EXTERIOR WALLS (W_T = 10 PSF)

1/2 OF WALL WEIGHT GOES TO ROOF

10×.133×1.125×1/2 = 0.75 PSF

LOADS TO 2ND FLOOR DIAPHRAGMS

DESIGNER IS TOLD TO ADD 2ND FLR. 'CEILING' LOAD TO 2ND FLOOR LOAD TO GET TOTAL EQUIV. SEIS. LD. BUT ALL WALL LOAD FROM ABOVE MUST BE CARRIED BY 2ND FLR. DIAPHRAGM AND ONLY A PORTION IS IN 'CLG' LOAD.

FOR 10 PSF WALL WEIGHT –

LD. TO 2ND – UPPER 1/2 OF 2ND FLR WALLS – 0.667 – 0.25 = 0.417

LOWER " " " " " – = 0.667

UPPER 1/2 OF 1ST " " – .625×1/2×10×.133 = 0.417

1.501

USE 1.500 FOR STD CASE AND 0.420 (.417) FOR FLAT ROOFS OR SLOPING CLGS.

USE 1/2 OF ALL LOADS SHOWN IN ZONE 2

SNOW LOADS

Requirements for handling snow loads in seismic design considerations are not as clear as most other similar concerns. Where snow is likely to be present on a roof it is clear that its inertial force will add to the load which must be resisted by the structure. Snow, however, is often not attached directly to the structure and for all but flat roofs it might not only be presumed that the snow would have a tendency to begin sliding during severe shaking, but also it is logical to assume that only the weight transferred to the structure by frictional forces and adhesion (of ice or hardpack) should be considered. In Section 2314 of the Uniform Building Code under the definition of W, the exception states "where snow loads are considered the snow load shall also be included; however, when approved by the Building Official the snow load may be reduced up to 75 percent maximum." Even the statement in the Code leaves more latitude than is allowed for most other design requirements. In addition, the Seismology Subcommittee of the Structural Engineers Association of California currently has under consideration a modification which would allow snow loads of 30 psf or less to be ignored in seismic design.

The HUD Manual of Acceptable Practices (MAP) presents snow load charts indicating ground snow weight in multiples of 10 psf and states that for most conditions 80 percent of this weight shall be used to determine roof snow load. The Report has taken these maps into consideration and has allowed snow load to be disregarded for roof loads of 32 psf or less. It is felt that this position is principally justified by practical considerations with respect to seismic design for dwellings. In addition, the current considerations of the Structural Engineers Association of California add some weight to this decision.

Where local building officials require snow loads to be considered in seismic design, the Report's methodology is capable of allowing the designer to incorporate such loads without other special considerations or methods being required.

HEIGHT TO LENGTH RATIOS FOR WALLS WITH VARIOUS SHEAR RESISTING MATERIALS

The Uniform Building Code, Table 25-I, specifies maximum height to width ratios as follows:

Diagonal Sheathing conventional	2:1
Diagonal sheathing special	3-1/2:1
Plywood nailed all edges	3-1/2:1
Plywood blocking omitted at intermediate joints	2:1

A 1-1/2:1 height to width ratio is specified for fiberboard sheathing by Section 2515 of the UBC, and for gypsum lath and plaster, gypsum sheathing board and gypsum wall board by Section 4713 (d). Although it is not specifically stated, woven or welded wire lath and Portland Cement plaster (stucco) is also covered by the latter section and it is assumed that the 1-1/2:1 ratio is intended to apply to this material as well.

Section 2518 (f) 5 of the section of the UBC titled "Conventional Construction Provisions" allows the use of diagonal sheathing, plywood, fiberboard sheathing, gypsum sheathing, particle-board and gypsum wall board as shear resisting materials and specifies that each panel so covered shall be at least 48 inches in width. The "Conventional Construction Provisions" are intended primarily for home construction. The typical wall height in residences is 8'-0" and in designs incorporating cathedral ceilings, higher ceilings in living rooms, etc., these heights will exceed 8'-0" and may conceivably extend to 16'-0" or greater. The implication that a 2:1 ratio is acceptable in home construction for the "lesser" shear resisting materials is clear.

The Report specifies an allowable height to width ratio for plywood sheathing of 3-1/2:1 and an allowable 2:1 ratio for all other materials. Special diagonal sheathing is rarely used and is not specified in the Report, so therefore the 3-1/2:1 ratio for that material is also not mentioned. The 2:1 ratio has been adopted for the other materials for the following reasons:

1. The HUD Minimum Property Standards presently refers to the Uniform Building Code for Seismic Design and the Code implies this ratio as acceptable as described above.

2. By specifying a 2:1 ratio, even higher ratios (and therefore higher deflections) are eliminated from use in dwellings containing shear walls of greater than average height. These types of dwellings tend to incur greater damage in seismic disturbances, and it would be undesirable to continue to encourage the use of such shear walls.

3. The Report apportions shear resistance in the same manner as is required for engineered structures in that the allowable shears for the various materials have been specified, thereby requiring that wall lengths along a given line of resistance either be proportional to the load if ordinary finish materials are used for shear resistance, or requiring that plywood be utilized when lesser total lengths are provided.

4. Using the front elevation of Model A, as presented in the Report, as an example, it appears that requiring a height to width ratio of less than 2:1 could severely penalize residential structures in terms of providing required fenestration. When considering economy, practicality, and esthetic requirements in combination with structural considerations, it does not seem unreasonable to allow this higher height to width ratio to be used for dwelling construction.

SHEAR WALL RESISTANCE

Because the determination of shear wall is such an important part of earthquake resistive design, the value of the Methodology in offering earthquake protection is directly related to the assignment of allowable shear values for various materials. The shear values assigned in the Report have been taken directly from the Uniform Building Code[2] for all finish materials other than hardboard. It was felt necessary to use the shear values assigned in the Uniform Building Code in order to make the Report viable and workable. A review of wall panel racking test data has revealed that the UBC values may not be warranted in all cases but the assignment of other values would have meant that designers using the Report might have met HUD requirements but could not use the designs they prepared to obtain local building permits.

No shear resistance value for hardboard is given in the 1973 edition of the UBC.[2] An ICBO approval has been obtained by The Masonite Corporation of Chicago, Illinois for many of their hardboard siding materials in accordance with ICBO Report No. 1487. Representative samples of the test data and the ICBO report itself are included at the end of this section. The extension of this information to include all hardboard siding is based first on the fact that hardboard is one of the few shear resisting materials for which allowable shear is based on ultimate load rather than 1/8 inch deflection criteria. Secondly, Masonite states that their competitors' products, manufactured in accordance with Voluntary Product Standard PS60-73, will display essentially the same properties with respect to racking as is exhibited by the Masonite materials. Since there are many materials similar to hardboard, this Product Standard has been referenced in the Report to identify those products for which the shear values indicated are applicable.

The allowable shear values assigned for hardboard (see Racking Test Summary at the end of this section) have been developed using ICBO criteria rather than the criteria set forth in Appendix D of the MPS. There is a conflict in the MPS requirements stating that lateral earthquake load designs should be in accordance with the Uniform Building Code and the test procedure and performance criteria stated for wall assemblies and sheathing materials in Appendix D of the MPS. It was partially necessary to use the ICBO requirements since these requirements are the only ones which state a method for determining allowable shear, i.e., one-third of ultimate load (U ÷ 24 for an 8' x 8' panel) or the shear developed at 1/8 inch deflection in an 8' x 8' panel, whichever is less. Hardboard easily meets these requirements and, in fact, on the basis of the test results, appears to be second only to plywood in performance. The hardboard shear values used in the Report were determined from the racking test data based on the above stated criteria and are given in the Racking Test Summary at the end of this section.

HARDBOARD

PACKING TEST SUMMARY

SOURCE	DESCRIPTION	ULTIMATE LOAD (lb)	$\frac{U}{24}$ (lb/ft)	1/8" Δ LOAD (lb)	$\frac{1/8"}{8}$ (lb/ft)	Δ AT ALLOWABLE SHEAR (in)
1970 - 1 Test	7/16" Barkridge - Shiplap 6d @ 4 & 8	5600	233.3	2827	353.4	0.084
1969 - 1 Test	7/16" X-90 Moonspot - Shiplap 6d @ 4 & 8	7600	316.7	3328	416	0.040
1969 - 1 Test	7/16" X-90 V-Grove 6d @ 4 & 8	5200	216.7	3274	409.3	0.037
1964 - 3 Tests	1/4" Weatherall - Butt 6d @ 4 & 8	7600	316.7	3440	430	0.076
1961 - 3 Tests	3/8" X Siding - Butt 6d @ 6 & 12	5867	244.5 (x1.5=366)	2571	321.4	0.110

International Conference of Building Officials

RESEARCH COMMITTEE RECOMMENDATION

Report No. 1487
June, 1974

MASONITE X-90 SIDING PRODUCTS
MASONITE CORPORATION
29 NORTH WACKER DRIVE
CHICAGO, ILLINOIS 60606

I. Subject: Masonite X-90 Siding Products as an exterior wall covering.

II. Description: A. General: All of the Masonite X-90 Siding products utilize a uniform base hardboard. The products are manufactured from selected woods that are mechanically reduced to chips: then further reduced through steam, pressure and mechanical refining to individual fibers which retain the natural binding agent lignin. Certain additives are blended in to enhance specific properties. The Colorlok lap and panel sidings have a flame-spread rating of 200 and a smoke density rating of 400.

The fibers are felted into a blanket which is pressed between heated platens to the desired thickness and density. The resultant panel has essentially equal strength in all direction.

Panels are humidified to an optimum moisture content, then cut to size and fabricated into the final product.

B. Surface Textures: The X-90 siding products have a variety of surface textures with and without grooves.

1. X-90 designates the smooth conventional surface. This surface is suitable for paint type finishes only.

2. Ruf-X and Resawn designate a simulated rough-sawn textured surface suitable for paint and some stain type finishes.

3. Barkridge designates a deep embossed simulated wood bark-like texture suitable for paint and some stain type finishes.

4. Woodsman designates an embossed surface simulating weathered barn board suitable for paint and some stain type finishes.

5. Moonspot designates a surface covered with many small craters simulating certain types of stucco. This surface is suitable for paint type finishes.

6. Bayside is a multilevel embossed lap siding which simulates wood shingles. The surface is suitable for paint and some stain type finishes.

7. Stuccato designates an embossed stucco texture simulating a skip-trowel surface. The surface is primed or prepainted

8. Colorlok designates a completely prefinished smooth face siding with a long term finish.

C. Groove Profiles: The panel type sidings are available in a plain ungrooved design with square edge butt joints for use with field applied battens or lengthwise grooves of varying widths, profiles or spacings. The grooves are all a maximum depth of ⅛-inch and the panels have shiplapped long edges for a weathertite joint without the use of a batten.

D. Surface Finishes: The X-90 siding products are available in a variety of surface finishes.

1. Unprimed: This finish is uncoated exterior hardboard which is prepared for field priming and painting (all products) or staining with semi-transparent or heavy-bodied stains (products with woodlike surface textures only).

2. Primed: This finish is prepared for field painting (all products) or staining with heavy-bodied stain (products with woodlike surface textures only). Finishing is to take place within 120 days of application.

3. Prestained: A uniform factory application of a stain type finish is applied thus eliminating the field staining operation. (Not available on smooth or stucco embossed products.)

4. Prepainted: A short term factory applied finish, intended to replace the first field paint application is applied. It can be re-painted. (Selected products only.)

5. Prefinished: A long term finish is factory applied and is available on Colorlok series of products only.

E. Field Finishing: Primer: Primecoated panels must be painted within 120 days after installation. If exposed for a longer period of time, the siding must be reprimed with an exterior grade linseed oil base primer. Unprimed panels must be field primed before finish painting.

Painting: Alkyd, linseed oil or latex type paints are to be used. For all paints, follow the manufacturer's recommendations concerning the use of special primers or undercoats, the rate of spread and application procedures.

Coats of Paint Required: Total dry film thickness including primer and topcoat are to be a minimum of 4 mils.

Staining: Only textured sidings are to be stained. Use heavy-bodied stains on primed textured sidings. Unprimed textured sidings and trim may be finished with semi-transparent or heavy-bodied stains. All stains must be applied following the manufacturer's recommendations for mixing, method of application, rate of spread and number of coats required. All exposed surfaces and edges are to be stained.

Prestained Products: Prestained panel sidings do not require field finishing.

Prepainted Products: This finish is formulated to replace one normal paint application and does not require field finishing.

F. Application: 1. Lap Siding Products: Lap siding products are furnished in varying widths, in lengths to 16 feet and have a nominal [illegible]-inch thickness. Corners and wall intersections are covered with metal or wood cornering with all joints caulked.

All lap sidings must be overlapped a minimum of 1 inch. The Bayside and Colorlok have fixed exposure and overlap, all others can be varied. These products must be nailed into every stud and every 16 inches on center above and below doors and windows.

Lap siding may be used on sheathed or unsheathed walls with studs spaced a maximum of 16 inches on center provided the wall is braced. Except for applications of Colorlok over solid sheathing, all butt joints must fall at stud locations. Unless applied over an approved weatherproof sheathing, a building paper barrier must be used.

The Colorlok Lap siding has concealed nailing and an interlocking spline mounting system.

The lap siding is nailed to each stud of unsheathed walls with 8-penny by 2½-inch galvanized nails. When applying the lap siding over sheathing up to [illegible] inch thick, 10-penny by 3-inch galvanized nails are used.

2. Panel Siding Products: Panel siding products are furnished in nominal 4-foot widths, lengths to 16 feet and have nominal thicknesses of ⅜ inch and [illegible] inch.

Ungrooved panels with a square butt joint may be applied to sheathed or unsheathed walls with studs spaced a maximum of 24 inches on center grooved panels, or panels with vertical shiplap joints may be applied to sheathed or unsheathed walls with studs spaced a maximum of 16 inches on center without corner bracing. All panel joints must occur at stud locations. When the vertical edges have lap joints or square edge joints are covered with a batten, building paper may be omitted. Colorlok panel siding has concealed nailing and snap-on color matched battens.

The siding is nailed directly to the studs with 6-penny by 2-inch galvanized box nails spaced 4 inches on center at all edges at intermediate framing members. For nominal ⅜-inch thick panels with shiplap edges and [illegible]-inch deep square cut grooves, additional nails spaced 8 inches on center are installed in the groove formed at the shiplap joint. All nails are to be located ⅜ inch from the panel edges. Where racking strength requirements are not needed, either a box head or siding nail of the same size may be used, and the nail spacing may be increased to 6 inches on center and 12 inches on center at vertical joints and intermediate supports, respectively.

G. Masonite Trade Names:

X-90 Lap Siding
Ruf-X Lap Siding
Bayside Lap Siding
Colorlok Lap Siding

Barkridge Lap Siding
X-90 Panel Siding
X-90 V-Groove Panel Siding
X-90 Panelgroove Siding
Ruf-X Panel Siding
Ruf-X Panelgroove Siding
Ruf-X Reverse Batten Siding
Woodsman Panel Siding
Moonspot Panel Siding
Barkridge Panel Siding
Resawn Panel Siding
Stuccato Panel Siding
Colorlok Soffit System
Plain and Slotted Soffit

II. Identification: Each product is back branded to identify the material as X-90 siding, and that it is manufactured by the Masonite Corporation. An Underwriters Laboratories, Incorporated label is also provided on the Colorlok Lap and Panel Sidings.

III. Evidence Submitted: Modulus of rupture, racking strength, weatherability and fire hazard classification test reports have been submitted.

Recommendation

IV. Recommendation: That the Masonite X-90 Siding Products are alternates to the exterior siding materials specified in the Uniform Building Code where no fire-resistive rating is required, subject to the following provisions:

1. The product shall be designated as Masonite X-90 siding.
2. No lateral value is assigned to lap siding and therefore all of the walls to which it is applied must be braced in accordance with the requirements of the Code.
3. Unless applied directly over an approved weatherproof sheathing, building paper is installed under lap siding.
4. Grooved siding used as sheathing or in lieu of corner bracing shall not be installed with horizontal joints and supporting framing must be spaced not to exceed 16 inches on center.
5. Three-eighths-inch panel siding used as sheathing or in lieu of corner bracing shall not be installed with horizontal joints and supporting framing must be spaced not to exceed 24 inches on center.
6. All window, door and other openings shall be flashed in accordance with the requirements of the Code.
7. The panel shear values are as set forth in Table No. I.
8. All materials are installed and finished according to this report and the manufacturer's recommendations.
9. The Colorlok Lap and Panel Sidings are classified as Class III material under the provisions of Chapter 42 of the Code.

This recommendation is subject to annual re-examination.

PANEL	STUD SPACING (In Inches)	ALLOWABLE SHEAR LBS./FT.
⅜-inch Grooved Panel Sidings with Shiplap Joints	16	230
Smooth Panel Sidings with Butt Joints	16	300
	24	280

[1] All panel edges backed with 2-inch nominal or wider framing. Panels installed vertically. All nails shall be 6 penny common spaced at 4-inch centers on all edges and boundaries with 8-inch centers for field nailing. These values are for short time loads due to wind or seismic forces and must be reduced 25 per cent for normal loading.

Racking Test Summary No. 32

ROBERT W. HUNT COMPANY, ENGINEERS
CHICAGO 7, ILLINOIS

January 29, 1970

FILE NO. 1371-1 REPORT E-26
ORDER 12-F-784 PAGE 2

Masonite Corporation--Racking load test of Barkridge panels.

Details of Racking Load Test

Load pounds	Deflection - inches Single Panel	Set - inches Single Panel
400	0.005	
800	0.013	0.008
1,200	0.024	0.014
1,600	0.069	0.054
2,000	0.094	
2,400	0.111	0.069
2,800	0.124	
3,200	0.139	
3,600	0.159	
4,000	0.181	
4,400	0.210	
4,800	0.264	
5,200	0.292	
5,600	0.374 *	

* Failure occurred after this particular loading was made.

Respectfully submitted,
ROBERT W. HUNT COMPANY

A. C. Dorr

Physical Laboratory Division

ACO/ms-6

Racking Test Summary No. 33

ROBERT W. HUNT COMPANY, ENGINEERS
CHICAGO 7, ILLINOIS

July 11, 1969

FILE No. 1371-1 REPORT D-263
ORDER L-266 PAGE 2

Masonite Corporation--X-90 Moonspot Siding.

Wooden Test Panel Frame:

The 8' wide x 8' high test panel frame was constructed of construction grade Douglas Fir 2" x 4" studs and plates. The frame was constructed and nailed per Figure 9 of ASTM E72-68. There were no let-in braces on 2" x 4" frame.

Racking Load Test Method

Procedure mainly followed those specified in ASTM E72-68 and FHA Circular No. 12 except as to measurement of horizontal movement. Measurement involved the use of a large triangle whose base was fastened to the test panels floor plate. The test panels movement at top corner, where load was being applied, was measured with a dial gage which had 0.001" graduations.

Deflection and set readings were taken at the required loadings as listed in the two specifications.

Load - Deflection - Set Test Data

Load-pounds	Deflection - inches	Set -inches	FHA Minimums or Maximums Deflection	Set
400	0.002			
800	0.006	0.001		
1,200	0.017	0.006	0.200" -Max.	- 0.100"
1,600	0.029	0.008		
2,000	0.048			
2,400	0.057	0.017	0.600" -Max.	- 0.300"
2,800	0.090			
3,200	0.117			
3,600	0.142			
4,000	0.172			
4,400	0.209			
4,800	0.262			
5,200	0.287	(F.H.A. minimum load for dry panels)		
5,600	0.336			
6,000	0.404			
6,400	0.476			
6,800	0.636			
7,200	0.674			
7,600	0.809			

(Failure occurred on trying to reload to 8,000 lbs.)

Respectfully submitted,
ROBERT W. HUNT COMPANY

A. C. Owen

Physical Laboratory Division

ACO/ms-6

Racking Test Summary No. 34

ROBERT W. HUNT COMPANY, ENGINEERS
CHICAGO 7, ILLINOIS

July 11, 1959

FILE NO. 1371-1 REPORT D-262
ORDER L-266 PAGE 2

Masonite Corporation--X-90, V-Groove siding.

Wooden Test Panel Frame:

The 8' wide x 8' high test panel frame was constructed of construction grade Douglas Fir 2" x 4" studs and plates. The frame was constructed and nailed per Figure 9 of ASTM E72-68. There were no let-in braces on 2" x 4" frame.

Racking Load Test Method

Procedure mainly followed those specified in ASTM E72-68 and FHA Circular No. 12 except as to measurement of horizontal movement. Measurement involved the use of a large triangle whose base was fastened to the test panels floor plate. The test panels movement at top corner, where load was being applied, was measured with a dial gage which had 0.001" graduations.

Deflection and set readings were taken at the required loadings as listed in the two specifications.

Load - Deflection - Set Test Data

Load-pounds	Deflection - inches	Set -inches	F.H.A. Minimums or Maximums Deflection	Set
400	0.004			
800	0.010	0.000		
1,200	0.015	0.001	0.200" -Max.	- 0.100"
1,600	0.024	0.004		
2,000	0.045			
2,400	0.067	0.024	0.600 -Max.	- 0.300
2,800	0.095			
3,200	0.117			
3,600	0.160			
4,000	0.208			
4,400	0.271			
4,800	0.430			
5,200	0.630	(F.H.A. minimum load for dry panels) (Failure occurred on reloading to 5,600 lbs.)		

Respectfully submitted,
ROBERT W. HUNT COMPANY

A. C. Over

Physical Laboratory Division

ACO/ms-6

Racking Test Summary No. 35

ROBERT W. HUNT COMPANY, ENGINEERS
CHICAGO 7, ILLINOIS

FILE NO. 1371-1
ORDER L-4100

April 28, 1964

REPORT 21586
PAGE 3

DETAILS OF RACKING LOAD TESTS

DRY PANEL TESTS

Load - Pounds	Test No. 4 Deflection - inches	Test No. 4 SET inches	Test No. 4 SET % of Deflection	Test No. 5 Deflection - inches	Test No. 5 SET inches	Test No. 5 SET % of Deflection	Test No. 6 Deflection - inches	Test No. 6 SET inches	Test No. 6 SET % of Deflection
400	0.01			0.005			0.005		
800	0.015	0.0	-	0.02	0.0	-	0.01	0.0	-
1200	0.02	0.0	-	0.025	0.01	40	0.03	0.01	33
1600	0.03	0.01	33	0.04	0.015	37	0.05	0.015	30
2000	0.035			0.07			0.07		
2400	0.05	0.01	20	0.09	0.02	22	0.09	0.03	33
2800	0.06			0.11			0.11		
3200	0.07			0.14	0.03	21	0.14	0.05	36
3600	0.09			0.18			0.17		
4000	0.105	0.02	19	0.22	0.07	32	0.22	0.09	41
4400	0.13			0.27			0.25		
4800	0.16			0.32	0.12	37	0.30	0.13	43
5200	0.18			0.38			0.37		
5600	0.21	0.04	19	0.43	0.17	34	0.42	0.20	48
6000	0.26			0.50			0.60		
6400	0.29			0.58	0.26	45	0.69	0.37	62
6800	0.34			0.66	0.31	47	0.85		
7200	0.41	0.14	34	0.78			0.95	0.55	58
							Failed on Reloading after 7200 lb load		
7600	0.50	0.18	36	0.95					
8000	0.66			Failed on Reloading After 7600 lb load					
8400	Failed on Reloading after 8000 lb load								
8800									
9200									

Racking Test Summary No. 35

ROBERT W. HUNT COMPANY, ENGINEERS
CHICAGO 7, ILLINOIS

April 28, 1964

FILE NO. 1371-1
ORDER L-4100

REPORT 21586
PAGE 4

DETAILS OF RACKING LOAD TESTS

WET PANEL TESTS

	Test No. 1			Test No. 2			Test No. 3		
	Deflec-	SET		Deflec-	SET		Deflec-	SET	
Load - Pounds	tion - inches	inches	% of Deflection	tion - inches	inches	% of Deflection	tion - inches	inches	% of Deflection
400	0.015			0.005			0.005		
800	0.025	0.0	-	0.01	0.0	-	0.01	0.0	-
1200	0.035	0.005	14	0.025	0.0	-	0.02	0.0	-
1600	0.04	0.01	25	0.03	0.005	17	0.03	0.01	33
2000	0.045			0.05			0.05		
2400	0.06	0.02	33	0.07	0.015	21	0.065	0.01	15
2800	0.075			0.09			0.075		
3200	0.09	0.03	33	0.11	0.035	32	0.10	0.025	25
3600	0.11			0.14			0.12		
4000	0.13	0.04	39	0.17	0.08	47	0.15	0.055	37
4400	0.16			0.20			0.18		
4800	0.18	0.07	39	0.26	0.13	50	0.21	0.07	33
5200	0.21			0.32			0.24		
5600	0.24	0.10	42	0.37	0.19	51	0.29	0.12	41
6000	0.27			0.45			0.34		
6400	0.32	0.15	47	0.51	0.27	53	0.40	0.19	47
6800	0.36			0.57			0.45		
7200	0.43	0.23	53	0.68	0.38	56	0.50	0.25	50
7600	0.56	0.32	57	0.83			0.61		
8000	0.68			Failed on Reloading after 7600 lb load			0.68		
8400	0.78						0.76	0.41	
8800	1.02						Failed on Reloading after 8400 lb load		
9200	Failed on Reloading after 8800 lb load								

Respectfully submitted,
ROBERT W. HUNT COMPANY

Physical Laboratory Division

ACO:gp-5

Racking Test Summary No. 36

ROBERT W. HUNT COMPANY, ENGINEERS
CHICAGO 7, ILLINOIS

November 22, 1961

FILE NO. 22622-1
ORDER L-3803

REPORT 17368-A
PAGE 3, Revision as of December 4, 1961

DETAILS OF RACKING LOAD TESTS

DRY PANEL TESTS

Load-pounds	Test No. 1 Deflection - inches	Test No. 1 Set inches	Test No. 1 Set % of Deflection	Test No. 2 Deflection - inches	Test No. 2 Set inches	Test No. 2 Set % of Deflection	Test No. 3 Deflection - inches	Test No. 3 Set inches	Test No. 3 Set % of Deflection
200				0.01			0.00		
400				0.02			0.01		
600				0.03			0.015		
800	0.03	0.00		0.04	0.01	25	0.02	0.0	0
1000				0.05			0.025		
1200	0.04	0.01	25	0.06	0.02	33	0.03	0.005	17
1400				0.65			0.035		
1600	0.045	0.015	33	0.07	0.03	43	0.04	0.01	25
1800				0.08			0.06		
2000				0.09			0.075		
2200				0.10			0.09		
2400	0.07	0.025	35	0.11	0.04	36	0.11	0.04	36
2600				0.11			0.13		
2800				0.12			0.155		
3000				0.14			0.18		
3200				0.20	0.08	40	0.21	0.09	43
3400				0.23			0.245		
3600				0.26			0.275		
3800				0.31			0.315		
4000	0.28			0.36	0.17	47	0.34	0.16	47
4200				0.41			0.38		
4400				0.47			0.43		
4600				0.54			0.445		
4800				0.63	0.34	54	0.49	0.24	49
5000				0.69			0.54		
5200				0.74			0.58		
5400	0.74			0.81			0.64		
5600	0.78			0.85	0.51	60	0.70	0.38	54
5800				0.92			0.83		
6000				0.96			0.94		
6200				1.00					
6400				1.06	0.66	62			
6600				1.25					

Racking Test Summary No. 36

ROBERT W. HUNT COMPANY, ENGINEERS
CHICAGO 7, ILLINOIS

November 22, 1961

FILE NO. 22622-1
ORDER L-3803

REPORT 17368-A
PAGE 4, Revision as
December 4, 1961

DETAILS OF RACKING LOAD TESTS

WET PANEL TESTS

Load-pounds	Test No. 4 Deflection - inches	Test No. 4 Set inches	Test No. 4 Set % of Deflection	Test No. 5 Deflection - inches	Test No. 5 Set inches	Test No. 5 Set % of Deflection	Test No. 6 Deflection - inches	Test No. 6 Set inches	Test No. 6 Set % of Deflection
200	0.02			0.01			0.00		
400	0.02			0.015			0.00		
600	0.03			0.02			0.005		
800	0.035	0.01	28	0.025	0.000		0.015	0.00	
1000	0.045			0.03			0.03		
1200	0.05	0.03	60	0.035	0.01	29	0.04	0.005	12
1400	0.06			0.04			0.045		
1600	0.065	0.035	54	0.06	0.02	33	0.05	0.01	20
1800	0.08			0.07			0.06		
2000	0.09			0.08			0.07		
2200	0.11			0.09			0.09		
2400	0.125	0.08	64	0.11	0.045	41	0.095	0.035	37
2600	0.15			0.12			0.11		
2800	0.17			0.19			0.14		
3000	0.19			0.23			0.18		
3200	0.21	0.11	52	0.27	0.15	56	0.215	0.13	60
3400	0.24			0.33			0.255		
3600	0.28			0.36			0.29		
3800	0.31			0.42			0.36		
4000	0.35	0.21	60	0.47	0.29	62	0.43	0.26	60
4200	0.38			0.51			0.49		
4400	0.43			0.56			0.56		
4600	0.48			0.63			0.63		
4800	0.52	0.32	61	0.68	0.44	65	0.71	0.42	59
5000	0.55			0.76			0.76		
5200	0.61			0.81			0.82		
5400	0.66			0.86			0.89		
5600	0.77	0.49	64	0.90	0.58	65	0.95	0.60	63
5800	0.88			1.01			1.05		
6000	0.97			1.08			1.10		
6200	1.11			1.17			1.17		
6400				1.40	0.94	67.2	1.28	0.86	67
6600							1.43		
6800							1.67		

Respectfully submitted,
ROBERT W. HUNT COMPANY

A. C. Oren

Physical Laboratory Division

ACO:gr-15

STABILITY OF SHEAR WALLS (Overturning)

Because of the complexities involved in determining the stability of individual shear walls, it was felt necessary to "simplify" the determination of stability and the requirements for hold-downs in the Report. In view of the considerable discussion in the Report, the length of the portion of the Design Methodology dealing with this subject and the number of tables and graphs presented, it is obvious that even this "simplified" presentation is more complex than might be desired. The philosophy used in developing the approach presented was based upon observations indicating that overturning of shear walls in one-story and most two-story homes was not a principal consideration. On the other hand, higher-than-average one-story shear walls created problems which could not be ignored. The approach taken is therefore somewhat liberal, with the intent being to allow the designer to determine those shear walls which critically required hold-down anchors, as opposed to indications of exact uplift loads on each and every occasion. Within this framework of liberalized assumptions it was attempted to make the calculations as accurate as possible.

Several of the assumptions made in determining the Design Methodology are not fully supportable by conventional engineering theory. In addition to presenting samples of the calculations made to develop the hold-down graphs and other uplift data presented in the Report, this section presents the reasoning used in determining the various procedures and requirements. It is repeated that these requirements were based on field observations of mechanisms of failure as much as by engineering theory.

Vertical Loads

In determining the stability of a shear wall, the engineer considers the vertical load per foot produced by the weight of the wall itself and any floor or roof framing supported by it, as well as concentrated loads from perpendicular or parallel headers, beams, etc. The loading diagram is either visualized or drawn and moments are taken about the end which will produce the least moment of stability. In the Report, it was deemed necessary to require the designer to determine the uniform load per foot produced by the weight of the wall and any other uniform loads supported by it.

Dealing with concentrated loads became quite another matter. The concentrated loads themselves are naturally variable and do not necessarily occur at the end of the wall. In addition, resistance to uplift is frequently provided by perpendicular walls, especially at exterior corners of the structure. The shear resisting material around the corner is usually fastened to the same corner studs which represent the end of the shear wall and resistance to uplift can thus be transferred around the corner.

These considerations were thought to be much too complex to attempt to define explicitly in the Report. In place of these determinations the designer is arbitrarily told that 100 pounds can be added to the allowable horizontal load determined for each segment of shear wall from the appropriate graphs or calculations. This allowance is very conservative for all but the shortest nonbearing shear walls. A 4-foot long nonbearing wall, for instance, would require a 200 pound vertical dead-load at each end in order to justify the additional 100 pound horizontal load. If the wall is nonbearing it is logical to assume the headers on either side of the wall would be nonbearing also, and it would therefore be necessary to assume an opening of approximately 5 feet or greater in order to justify the allowance in this case. Obviously a 4-foot long wall adjacent to a 3-foot wide man-door would not so qualify. On the other hand, a shear wall 8 feet or longer along the same line of resistance, or even a reasonably short header in a bearing wall, would easily develop the required concentrated load at the end of the wall. At exterior corners and other similar locations the shear resisting material extending around the corner would be capable of resisting a minimum load of 100 pounds per foot (or 800 pounds total for an 8-foot high wall) in the average house.

Finally, it was reasoned that most houses tend to have the largest openings and shortest lengths of shear wall at the front and rear, and that these same walls are bearing walls in most houses. Despite the many examples of walls that could be shown for which this 100 pound horizontal load assumption is too liberal, it is postulated that, on the average, the assumption should not make a critical difference in the determination of the requirement for hold-down anchors.

"Solid" Walls

A large number of houses are built with relatively short shear walls having windows and other similar openings adjacent to them. Field observations after the San Fernando earthquake indicated that most conventionally designed and framed one- and two-story homes encountered little difficulty with overturning of shear walls. It was only very high walls in one-story construction and first-floor walls having very short lengths of wall along a particular line of resistance in two-story construction which showed signs of distress due to this type of failure. When checked by standard engineering methods it is found that most short shear walls in one-story residences require some type of hold-down. It therefore became apparent that some credit must be given to the wall above and below adjacent window openings when determining the length of wall to be considered for overturning resistance. The calculations at the end of this section investigate some of the more obvious ways in which this action might take place. They are, of course, limited since specific assumptions have been made with regard to a specific wall, and are intended only to indicate possible sources of overturning resistance rather than a definitive solution to all cases.

The conditions as set forth in the calculations would appear to be the worst possible case. The shortest (and therefore the least stable) shear wall allowed by the Methodology (4'-0") is considered in combination with the least height allowed below the window and the greatest length of opening for this height. In order to have only one shear wall considered, the length of wall opposite the shear wall is assumed to be 3'-9". The wall is considered to be loaded with a nominal vertical load of 100 pounds per foot.

Several other possible methods of action were investigated but are not shown in the calculations presented. Included were the possibility that some fixity is developed at the joints, truss action, and the use of anchor bolts alone as hold-downs. It seems obvious that the anchor bolts must play a significant role but the method shown seems to be the most logical for the transference of moment above and below the window. The method would explain damage at the corners of the windows but this damage would not significantly affect the ability of the wall to act as a unit in the manner shown. It is obvious, however, that the entire structure of the wall is required to act as a unit in order to justify the action considered. This is particularly true for walls having shear in excess of about 180 lb./ft. (with shear material one side only) but except for hardboard, plywood would have to be added for shear in excess of this amount. Although the maximum condition indicates that plywood will be overstressed, its high ultimate strength implies that less damage would be incurred for this case than for stucco, for instance.

The theory of the action of these walls as presented is a reasonably satisfactory explanation for many conditions but obviously configurations can be visualized which would not be explained by the mechanics of this system. It may be that response displacements for stiff low-rise structures is so small that walls do not fail in overturning. Another possibility is that the ceiling and roof, even when framed parallel to the wall, provide sufficient torsional resistance to, in effect, substantially increase the vertical load acting on the wall. It would be particularly difficult, for example, for a shear wall located near the center of the total length of the exterior wall to rack a roof sufficiently to allow the wall to overturn. If this is the case it would make little difference what type of opening occured adjacent to the wall. Lastly, it should be noted that shear walls interconnected by a continuous top plate would have difficulty in overturning if one of the walls (or a combination of walls) were capable of resisting the overturning. In this case, some redistribution of shears seems far more likely. This principle is used in the Methodology for the determination of stability along a given line of resistance.

In the interest of economy and the consideration of field observations, it was determined to allow the use of "solid" walls based upon opening heights which appeared to perform reasonably well, and widths of openings which allow the height to width ratio of the shear resisting material either above or below the opening (but not necessarily both) to be only slighter greater than that allowed for shear walls. It was on this basis that a height of opening of 4'-8" in an 8-foot nominal height wall was established with a height to width ratio of the material above or below the opening of 2.5, which is only slightly greater than that allowed for the shear wall itself. When solid wall occurs at the far end of the opening opposite the shear wall, this wall will tend to fix the wall sections above and below the opening, thereby substantially reducing the height to width ratio. For this reason, the length of the wall was allowed to be extended beyond a single opening when wall, at least as wide as the highest section of panel above or below the opening, was provided between openings. When a post or very narrow section of wall occurs at the far side of the opening, "fixity" is either non-existent or diminished, and in these instances the designer is allowed to consider the length of wall as being extended to the far side of such post or solid wall, but to not extend it further.

When doors or other large openings exist adjacent to a shear panel, the small section of wall above the door cannot provide sufficient "fixity" in one-story construction. Where shear resisting material is continuous from above the door to the sills of windows at the second floor, it has been assumed that "fixity" can be established. This allowance is reflected in the Methodology.

It should be expected that the allowances for "solid" walls will result in some damage to shear resisting finishes at the corners of window openings. The inclusion of the use of "solid" walls, however, should limit the use of hold-down anchors to those walls having little or no redeeming features which would limit the use of such devices were the wall engineered. To further reduce the need for costly and troublesome hold-down installations, allowances have been made for both the framing anchor and strap-type hold-down. While the framing anchor hold-down is considered to be an intermediate solution for residential shear walls having little uplift, it is not recommended for use in commercial structures.

Determination of Hold-Downs Required

Since concentrated loads were not considered as a factor in determining shear wall stability, it was possible to present the graphs entitled, "Hold-Down Graphs A Through G", the formula contained in Section 6.4D3 of the Methodology, and the graphs for hold-downs Numbers 1 through 4. When no hold-down anchor is required, the allowable horizontal force, P, is easily determined as being the moment of stability created by the vertical uniform load divided by the height of the wall. Reduction of the vertical load due to the vertical acceleration is rarely considered in wood frame and small building design, but, if it is used, the reduction should obviously be made before using the graphs. Graphs were prepared ranging from 60 to 600 pounds per foot on the theory that 60 pounds per foot represented the lightest weight imaginable for a nonbearing wall and 600 pounds per foot was the highest load anticipated for bearing walls using standard framing methods. The designer is told to ignore veneer loads in determining vertical loads. The 600 pounds per foot can be exceeded in some two-story construction, and angle hold-downs may be required. The hold-down formula for angle hold-downs is given in Appendix A of the Report for this reason. When 600 pounds per foot is used, a 4-foot long wall, with only the double framing anchor hold-downs installed, is capable of resisting an overturning moment which would cause a shear in the wall of 250 pounds per foot. Only butt-jointed hardboard and plywood are capable of resisting this much load. It is unlikely that shear walls will be provided in many homes which not only have vertical loads in excess of 600 pounds per foot but also have walls so short that angle hold-downs would be required. For lighter uplift loads common to houses, the formula for strap hold-downs in Section 6.4D3 may be used.

When framing anchor or strap-type hold-downs are used, the formula presented in Section 6.4D3 assumes the hold-down force, F, as acting at the very end of the wall. The error caused by this assumption will normally not exceed about 4 percent (48 inches divided by 46.25 inches; 46.25 inches being the length of the wall minus 1/2 of the width of a 4 x 4). In determining the stability of walls using major hold-downs, the formula is more complicated and is therefore presented in graph form in the Report. As shown in the calculations following this section, the hold-down force, F, is assumed to be acting 6 inches from the end of the wall.

Hold-down loads were selected based on the capacity of commercially available hold-down devices as fabricated by a major manufacturer of building hardware. Since these allowable loads are predicated upon the bolt values, it was not felt that such an assumption led to the use of a proprietary item. In fact, angle-type hold-downs are detailed in the Report and may be used in lieu of pre-fabricated items. The manufacturer of the commercial hold-down anchors publishes the allowable design load values considering a 25 percent increase for metal side-plates. An asterisk in the table indicates that these loads may be increased one-third when used for the resistance of seismic loads. The 25 percent increase for metal side-plates has not been used in the Report in the design of any connections, and was therefore ignored in this case also. To avoid confusing designers using this or similar commercial hold-downs, the value as published in the table (with the 1/4 increase) was selected as the capacity of the hold-down in lieu of assigning the actual value (without the 1/4 increase) times 1.33 for seismic or wind loadings. In other words, the capacity of the angle hold-downs is listed with a 1/4 increase rather than the usual 1/3 increase allowed. It was deemed wiser to use the published values printed in the tables for such commercially available devices rather than to confuse the designer with values which were not readily apparent in the tables of design load values for such devices. A sample tabulation for no hold-down anchor required and for hold-down Number 1 is shown on the third page of the calculations.

Grade Beams

Probably no other section of the Report will be as subject to revision to meet local conditions as the requirements for grade beams. Since foundation requirements vary by locale, it is to be expected that local designers and contractors will wish to have designs prepared which reflect the type of footings installed in their area. Soil bearing value and frost line have an affect on the type of footing used. In addition, special considerations such as expansive soils and land fills over swampy conditions--to mention only two--have led to unique and varying solutions in different parts of the country. Since depth of footing plays a large part in the determination of the size and reinforcing of the grade beam, it was felt that the most conservative assumptions should be made in presenting information in the Report. It was assumed that this would be reflected in a 12-inch wide footing extending 12 inches below grade. As the calculations indicate, this footing size proves to be insufficient for most grade beams and, therefore, the assumption of a smaller minimum size would have had no effect on the calculations.

Short shear walls requiring hold-down anchors and grade beams are most commonly found along the exterior walls of a house, although they can occur at the interior as well. Along a given line of resistance (or given length of continuous footing), the shear wall can occur at either end or anywhere in between. Grade beams are most commonly installed as shown on Detail 29/4 of the Report. Calculations not presented herein show that the worst case occurs, however, when the shear wall is located at the end of the continuous footing with the horizontal load acting away from the corner. This condition is shown on page 85. The hold-down graphs essentially indicate the overturning moment which can be developed for a given length of wall by a specific hold-down anchor. Each wall height curve therefore represents the same moment, divided by that height, for any given length of wall. Since the moment is the same, the same grade beam can be used for a given length of wall regardless of its height, provided the vertical load per foot to the wall is also the same.

The UBC[2] allowable soil pressure (1000 psf increased 1/3 for seismic load) is not as critical as the grade beam length required to generate uplift resistance in determining the "a" distance required for the grade beam. For grade beams of the same weight, the "a" distance required is not much less, in order to provide the weight for uplift resistance, when the horizontal load is in the opposite direction. It was concluded that a single presentation, based on the formula shown, could be made in the Report. Tabulations were therefore prepared for varying lengths of wall to establish the locations at which grade beam reinforcing or size must be changed. This was done for each of the vertical loads per foot for which the hold-down graphs were prepared.

To account for differing foundation construction practices and allowable footing pressures throughout the country, it would be useful to revise Table 4.2 and Details 32/4 and 33/4 in the Report for the various locales.

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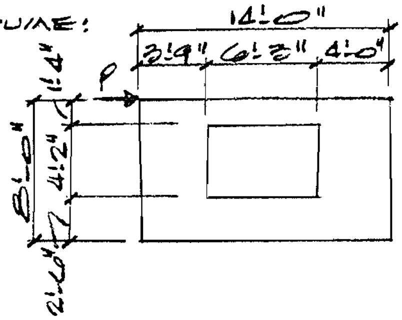
'SOLID WALLS'

ATTEMPT TO DETERMINE THE WAY IN WHICH THESE WALLS ACT SINCE, BY OBSERVATION, THEY DO NOT OVERTURN.

ASSUME:



WALL WT = 10 PSF
$w_{VERT} = 100$ #/'
THEN BELOW WINDOW: $w = 25$ #/'
ABOVE WINDOW: $w = 75$ #/'
WITH GYP. BD. SHTG.:
$v_{ALL} = 100$ #/'
$P = 4 \times 100 = 400$ #

IF IT IS ASSUMED HEADER ACTS IN BEARING AGAINST KING STUD THEN A COUPLE CAN BE FORMED WITH TOP PLATE WHOSE MAX. MOMENT IS:

$$M = 100\#/' \times 6.25' \times (16-2) = 8750''\# = 729'\#$$

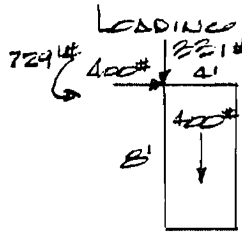
2x4 PLATE WILL TRANSFER $\frac{5.25 \times 95 \times 1.33}{1.5} = 443\#$

BUT LOAD CAUSING MOMENT IS:

$$75 L^2 \times 12/2 = 8750; \quad L = 4.41' \quad V = 4.41 \times 75 = 331\#$$

DUE TO THE FACT THAT A NAIL IS NEARLY ALWAYS PLACED AT A CORNER SUCH AS ABOVE AND BELOW THE WINDOW, AND SINCE SHORT LENGTHS OF WALL SUCH AS OCCUR ABOVE AND BELOW THE WINDOW AT THE VERTICAL JOINT SHOULD ALWAYS HAVE MORE NAILS THAN ARE REQUIRED TO DEVELOP THE SHEAR (DUE TO SPACING), SOME MOMENT CAN BE DEVELOPED BY THE SHEAR RESISTING MATERIAL ITSELF. IN THIS CASE, IGNORE THIS MOMENT.

LOADING DIAGRAM FOR WALL IS:



$OTM = 400 \times 8 = 3200$
$RM = 400 \times 2 + 331 \times 4 + 729 = \underline{2853}$
$= 347'\#$

½ SOLID WALLS'

ADD'L LOAD TO BE DEVELOPED $= \frac{347}{4} = 86.75\#$

BELOW WINDOW – $v = \frac{86.75}{2.5} = 34.7\#/'$ OK FOR GYP BD

WITH MAXIMUM ALLOWABLE LOAD

W/O HOLD-DOWN: $P_{MAX} = \frac{100 \times 14 \times 7}{8} = 1225\#$

$v = \frac{1225}{4} = 306\#/'$ 3/8" ST II – 8d @ 4,12

2x4 TOP PLATE WILL GOVERN SHEAR TRANSFER. IF ONE PLATE IS BUTTED AT OR NEAR LEFT EDGE OF 4' WALL;

$V_{ALL} = 443\#$ $L = \frac{443}{75} = 5.91'$ $M = 75 \times 5.91^2 \times \frac{1}{2} = 1310'\#$

$OTM = 1225 \times 8 = 9800$
$RM = 400 \times 2 + 443 \times 4 + 1310 = \underline{3882}$
NET O.T. $= 5918'\#$

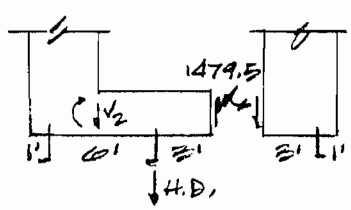
$\frac{5918}{4} = 1479.5\#$

FOR ASSEMBLY TO FAIL, BOTH PLYWD AND SOLE PLATE MUST FAIL. IF JOINT OCCURS IN SOLE PLATE DIRECTLY BELOW LOWER RIGHT CORNER OF WINDOW, THEN SILL BOLT WOULD OCCUR 12" MAX TO THE RIGHT OF THIS POINT AND ASSIST IN HOLD-DOWN. WITHOUT JOINT:

RESISTANCE $= 2.5 \times 380 + 443 = 1393\#$

$1479.5 - 1393 = 86.5\#$ – THIS COULD BE TAKEN BY TOE NAILS AT WINDOW SILL.

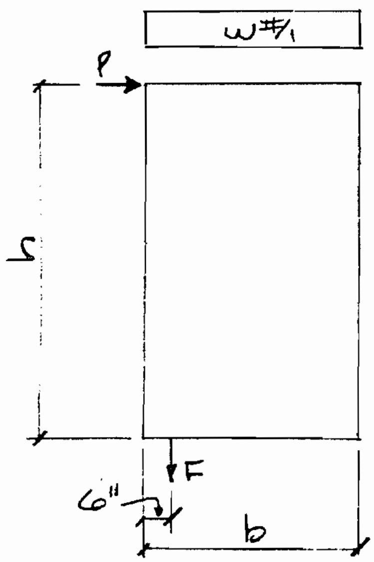
IF BOLT OCCURRED 1'-0" FROM RIGHT EDGE OF WALL THEN ACTION ON WALL WOULD BE AS SHOWN:



$M_{ALL} = 6.25 \times 380 \times 2.417 = 5740'\#$
$V_2 = \frac{5740}{6.25} = 918\#$ $(/2.5 = 367.2\#/')$
$V_{WALL} = 6.25 \times 25 = \underline{156}$
$1074\#$

WITH H.D. ACTING, THIS OBJECT IN BALANCE ($\Sigma M \neq \Sigma V \neq 0$). PLYWD IS THEREFOR OVERSTRESSED SOME INDETERMINATE AMOUNT.

HOLD-DOWN ANCHORS



HOLD-DOWN CAPACITY (F)	VERTICAL LOAD (w)
0	60 #/1
2520#	100 #/1
3610#	200 #/1
5410#	300 #/1
7380#	400 #/1
	500 #/1
	600 #/1

COMBINE EACH HOLD-DOWN CAPACITY WITH EACH VERTICAL LOAD TO OBTAIN ONE FAMILY OF CURVES FOR EACH COMBINATION. PLOT ONE CURVE FOR EACH HEIGHT CONSIDERED. PLOT WALL LENGTH VERSUS CAPACITY (P).

NO HOLD-DOWN

$$Ph = \frac{wb^2}{2} \qquad P_{ALL} = \frac{wb^2}{2h}$$

HOLD-DOWN

$$Ph = F(b-0.5) + \frac{wb^2}{2}$$

$$P_{ALL} = \frac{F(b-0.5) + \frac{wb^2}{2}}{h}$$

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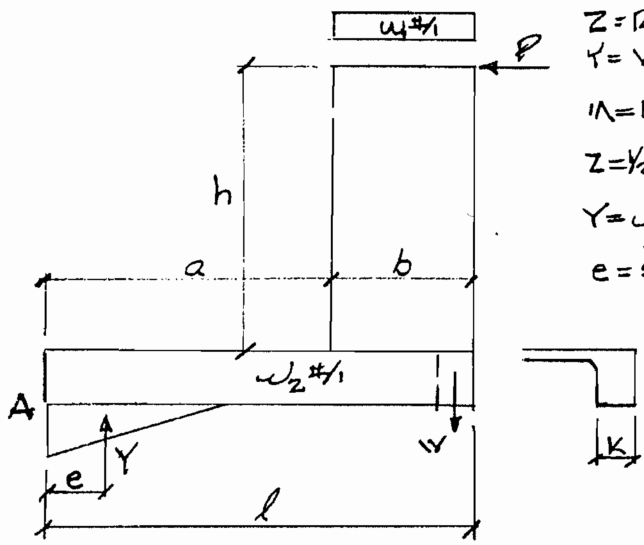
GRADE BEAMS

USE FOOTINGS AT CORNER AS WORST CASE (LESS MOMENT ARM). USE DIRECTION OF LOAD AWAY FROM CORNER SINCE SOIL PRESSURE IN THIS DIRECTION IS NOT TAKEN BY FULL WIDTH OF FOOTING AROUND CORNER.



M = OVERTURNING MOMENT
Z = RESISTING M ABOUT A
Y = VERTICAL LOAD

$$M = Ph$$

$$Z = \tfrac{1}{2} w_2 l^2 + w_1 b\left(l - \frac{b}{2}\right) + W\left(l - \frac{K}{2}\right)$$

$$Y = w_1 b + w_2 l + W$$

$$e = \frac{Z - M}{Y}$$

FOR 1'-0" WIDE FTG AND S.P. = 1000 PSF (x1.33 = 1333):

$$\frac{1333(3e)}{2} = Y = 2000\,e$$

$$e = \frac{Y}{2000} = \frac{Z-M}{Y} \; ; \; Y^2 = 2000(Z - M)$$

$$2000M = 2000Z - Y^2 \qquad M = Z - \frac{Y^2}{2000}$$

FOR FTGS WIDER THAN 1'-0":

$$M = Z - \frac{Y^2}{2000K}$$

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GRADE BEAMS

GENERAL PROCEDURE FOR FINDING M_{ALL}

CALL a DISTANCE REQUIRED TO RESIST UPLIFT a_U

CALL a DISTANCE FOR SOIL PRESSURE a_{SP}

I CHOOSE BEAM SIZE AND REINFORCING TO BE USED

A. SOLVE FOR MAX. ALLOWABLE MOMENT IN BEAM

B. USING BEAM'S WEIGHT PER FOOT SOLVE FOR a_U

1. $M_B = \frac{w_2 a_U^2}{2}$ $\quad a_U = \left(\frac{2M_B}{w_2}\right)^{1/2}$

2. a_U PROVES TO BE LONGEST a - USE FOR TABLE 4.2

3. DETERMINE $W = w_2 a_U$

II CHECK FOR SOIL PRESSURE

A. a_{SP} UNKNOWN

1. RIGHT END OF SOIL PRESSURE TRIANGULAR DISTRIBUTION MAY BE UNDER WALL
2. RIGHT END MAY BE BEYOND END OF WALL
3. MOMENT IN BEAM CAN NEVER BE AS BAD AS THAT CAUSED BY UNIFORM LOADING OF ALLOWABLE SOIL PRESSURE (- BM. WEIGHT) EXTENDING FROM LEFT EDGE OF WALL OUTWARD
4. DETERMINE a_{SP} FOR CONTROLLING CONDITION:

$$M_B = \frac{(333 - w_2)a_{SP}^2}{2}; \quad a_{SP} = \left(\frac{2M_B}{333 - w_2}\right)^{1/2}$$

5. SINCE $333 - w_2 > w_2$, $a_{SP} < a_U$

III - USING INFORMATION DEVELOPED ABOVE, SOLVE FOR M THAT CAN BE ACCOMMODATED USING a_U AND a_{SP} AS LENGTHS OF BEAM BEYOND WALL

A - $\ell = b + a_{SP}$

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GRADE BEAMS

III B SOLVE FOR Z IN TERMS OF b

1. $Z = \frac{1}{2} w_2 (b + a_{sp})^2 + w_1 b (b + a_{sp} - \frac{b}{2}) + W(b + a_{sp} - \frac{K}{2})$

2. $\;\; = \frac{1}{2} w_2 (b^2 + 2ba_{sp} + a_{sp}^2) +$ " $+$ "

3. $\;\; = \frac{1}{2} w_2 b^2 + w_2 b a_{sp} + \frac{1}{2} w_2 a_{sp}^2 + w_1 b^2 + w_1 b a_{sp}$
$- \frac{1}{2} w_1 b^2 + Wb + W(a_{sp} - \frac{K}{2})$

4. $Z = \frac{1}{2}(w_1 + w_2) b^2 + [W + a_{sp}(w_1 + w_2)] b + [\frac{1}{2} w_2 a_{sp}^2$
$+ W(a_{sp} - \frac{K}{2})]$ (ALL KNOWN EXCEPT b)

C SOLVE FOR Y IN TERMS OF b

1. $Y = w_1 b + w_2 (b + a_{sp}) + W$

2. $Y = (w_1 + w_2) b + (w_2 a_{sp} + W)$ (ALL KNOWN EXCEPT b)

D SOLVE FOR M IN TERMS OF b

1. INSERT ALL KNOWNS INTO EQUATIONS B4 AND C2

2. INSERT EQUATIONS OBTAINED INTO $M = Z - \frac{Y^2}{2000K}$, COLLECT TERMS AND OBTAIN EQUATION FOR M ALLOWABLE

E ESTABLISH WHERE GRADE BEAM CAN BE USED

1. SINCE HOLD-DOWN CURVES AND OTM ARE BASED ON $M = Ph$, $P = \frac{M}{h}$ AND M IS THE SAME FOR ALL CURVES ON ONE GRAPH REGARDLESS OF h (AT ANY GIVEN LENGTH)

2. WORK SEPARATELY WITH EACH VALUE OF w_1 USING SAME VALUES AS USED FOR DEVELOPING HOLD-DOWN GRAPHS

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GRADE BEAMS

III E 8 USE ENOUGH LENGTHS, b, TO ESTABLISH THE CURVE FOR HORIZONTAL LOAD. DIVIDE MOMENTS OBTAINED FOR EACH LENGTH BY 18'. PLOT VALUES OBTAINED ON WORK COPY OF EACH APPLICABLE HOLD-DOWN GRAPH.

a. IF PLOT IS ABOVE CURVE FOR h=18', GRADE BEAM IS GOOD FOR THE ENTIRE GRAPH

b. IF PLOT CROSSES CURVE, GRADE BEAM IS GOOD FOR ALL HEIGHTS UP TO THE LENGTH WHERE THE PLOT CROSSES THE h=18' CURVE (READING FROM THE LEFT).

c. IF PLOT IS BELOW CURVE, GRADE BEAM IS NOT APPLICABLE

F REPEAT FOR ALL VALUES OF W, STARTING WITH STEP III B

IV REPEAT ALL STEPS FOR EACH SIZE GRADE BEAM

NAILING SCHEDULE

Table 25-P of the Uniform Building Code specifies the nailing to be utilized between various wood framing members. To avoid conflict with this Nailing Schedule, the schedule presented in the Report for the most part sets forth the same requirements but extends required nailing to items not specified by the table in the UBC.

In determining the shears for which nailing should be provided, it is necessary to make conservative assumptions and yet attempt to arrive at connections that are not unduly expensive. For this reason, the nailing specified in the schedule approaches but does not always provide the design shears specified in the calculations. It must be recognized that the example calculations shown at the end of this section have attempted to arrive at the worst possible case that might ordinarily be expected. Some connections will obviously be over-stressed due to special conditions such as the installation of masonry fireplaces, fully veneered homes in combination with interior lath and plaster, or other special loadings. Homes for which lightweight materials are used throughout would be penalized, however, had the schedule in the Report not specified limits for the nailing which may be used with lightweight materials.

Sheathing

The nailing for plywood sheathing specified is the same as that in the Uniform Building Code for 3/8 inch sheathing with 8d nails for roofs and 1/2 inch sheathing with 10d nails for floors. This is as required for designed diaphragms as specified in Table 25-J of the UBC rather than the general nailing specified in Table 25-P. The nailing for straight sheathing is essentially the same as required by the UBC with the exception that straight sheathing is required to be nailed along its length to blocking at supports.

Roof and Floor Framing

The toe nailing of rafters and joists to top plates and the face nailing of ceiling joists to parallel rafters is as specified by the UBC. The toe nailing of joists and rafters to the top plates does not provide sufficient shear resistance and for this reason the toe nailing of blocking has been added to the Table. In Zone 3, the actual number of toe nails required for the blocking provides resistance in combination with the toe nailing of the rafters or joists to the plates. In accordance with the calculations given at the end of this section, total nailing provided is intended to provide resistance of approximately 150 lb/ft at roofs, 250 lb/ft at second floors or the first floor of one-story structures, and 300 lb/ft at the first floor line of two-story structures. A minimum of two 16d toe-nails per block is specified. In some cases, this creates allowable shears higher than the minimums indicated. When exterior finishes continue past the blocking, the footnote allows the nailing specified to be reduced by 50 percent since it is assumed that this exterior finish will provide at least half the total resistance of the shear wall. In Zone 2, the design shears are one-half of those specified for Zone 3. Because of the lower shears entailed, partial blocking has been allowed for roof framing in Zone 2.

Wall Framing

The Uniform Building Code presently stipulates nailing between top plates to be 16d at 16 inches on center with two 16d at laps and intersections. The Report's Nailing Schedule has added the requirement that a minimum of nine 16d be placed between laps and/or intersections. This is further discussed in this report under the heading "Chord Splices." Sole plate nailing has been revised from that specified in Table 25-P of the UBC[2] to more nearly reflect the design shears used for determining the toe nailing of blocking.

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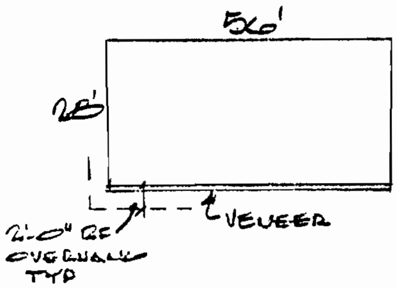
NAILING SCHEDULE

DETERMINE SHEARS BASED UPON ONE STORY HOUSE WITH 2:1 DIAPHRAGM RATIO AND TWO STORY HOUSE WITH 1½:1 DIAPHRAGM RATIO

ONE STORY - "HEAVYWEIGHT"



ASSUME STUCCO AND PLASTER WALLS & CLGS, GRAVEL ROOF & FRONT WALL VENEERED, REDUCE VENEER LD TO 32 PSF TO COMPENSATE FOR STUCCO CONSIDERED

	D.L.	2/3 LAT		D.L.	2/3 LAT
RFG -	6.0	0.8	CLG -	8.0	1.067
SHTG -	2.0	0.267	FRING -	2.0	0.267
FRING -	2.0	0.267		10.0	1.333
	10.0	1.333	INT WALLS -		0.500
			EXT WALLS -		1.500
					3.333

SEIS. LD. = RF - 32×30×1.333 = 1280
'CLG' - 28×28×3.333 = 2613
3893
VENEER - 28×21.33×¾ = 448
4341#

$\frac{4341}{28} = 155\#/'$ $\frac{3893}{28} = 139\#/'$

"LIGHTWEIGHT" DESIGN

ASSUME 10 PSF WALLS, 3 PSF CLG. ADD VENEER SEPARATELY.

RFG -	6.0	0.8	CLG -	3.0	0.4
SHTG -	2.0	0.267	FRING -	2.0	0.267
FRING -	2.0	0.267		5.0	0.667
	10.0	1.333	INT. WALLS -		0.25
			EXT WALLS -		0.75
					1.667

NAILING SCHEDULE

ONE STORY – 'LIGHTWEIGHT'

SEIS. LD. = RF – 32x30x1.333 = 1280
CLG' – 28x28x1.667 = 1307
2587#
VENEER – 28x21.33 = 597
3184#
VENEER 2 SIDES 597
3781#

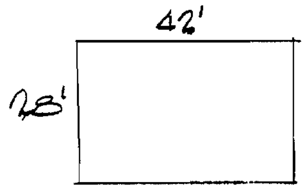
$\frac{2587}{28} = 92.4$#/' $\frac{3184}{28} = 113.7$#/' $\frac{3781}{28} = 135$#/'

VENEER TO ½ HT ON 'FRONT' WALL

P = 2587 + 597 x ¼ = 2736# $\frac{2736}{28} = 97.7$#/'

TWO STORY – 'HEAVYWEIGHT'

ROOF & CLG SAME AS 1 STORY



2ND FLR – FLRG	– 1.0	0.133
SHTG	– 2.0	0.267
FRMG	– 4.0	0.533
CLG	– 8.0	1.067
	15.0	2.000
INT WALLS		– 3.0
EXT WALLS		– 3.0
		8.000 PSF
		3.333
		11.333

SEIS. LD. – RF – 32x23x1.333 = 981
2ND – 28x21x11.333 = 6664
7645#
VENEER – 21x¾x69.33 = 1092
8737#

$\frac{7645}{28} = 273$#/' $\frac{8737}{28} = 312$#/'

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NAILING SCHEDULE

TWO STORY – 'LIGHTWEIGT'

2ND FLR. – FLRNG – 1.0   0.133
SHTG – 2.0   0.267
FRMG – 4.0   0.533
CLG – 3.0   0.400
10.0   1.333
INT WALLS – 1.500
EXT WALLS – 1.500
4.333
1.667
6.000 PSF

SEIS LD. – RF – 32×23×1.333 = 981
2ND – 28×21×6.000 = 3528
4509#
VNR 1 SIDE – 21×69.33 = 1456
5965#
VNR 2 SIDES   1456
7421#

$\frac{4509}{28} = 161$ #/' $\frac{5965}{28} = 213$ #/' $\frac{7421}{28} = 265$ #/'

WITH VENEER AT 1ST, FRONT, ONLY:

$P = 4509 + 21 \times 21.33 = 4957$ $\frac{4957}{28} = 177$ #/'

ALLOWABLE NAIL VALUES – 2510 f AND T. 25-G - 1973 UBC

FOR SEISMIC –

8d COMMON – FACENAIL – 78×1.33 = 104#
8d BOX – " – 104×.75 = 78#
8d COMMON – TOENAIL – 104×2/3 = 69.33#
8d BOX " – 78×2/3 = 52#
16d COMMON – FACENAIL – 107×1.33 = 142.7#
16d BOX " – 142.7×.75 = 107#
16d COMMON – TOENAIL – 142.7×2/3 = 95.1#
16d BOX " – 107×2/3 = 71.33#

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CONNECTIONS

USING COMMON NAILS -

NAILING	RAFTER OR JOIST SPACING				
	16"	24"	48"	72"	96"
3-8d T.N.	156#/1	104#/1	52#/1	34.67#/1	26#/1
3-8d + 1-16d TN	227.33#/1	151.5#/1	75.8#/1	50.5#/1	37.9#/1
2-16d TN	298.67	199	99.5	66.4	49.8
3-16d TN	370	246.7	123.3	82.2	61.7
4-16d TN	441.3	294.2	147.1	98.1	73.6
16d T.N. @	16"	12"	8"	4"	5"
	71.33#/1	95.1	142.67	285.33	228.24

USING BOX NAILS

NAILING	RAFTER OR JOIST SPACING				
	16"	24"	48"	72"	96"
3-8d T.N.	117#/1	78#/1	39#/1	26#/1	19#/1
3-8d + 1-16d T.N.	170#/1	114#/1	57#/1	38#/1	28#/1
2-16d T.N.	224	149	75	50	37
3-16d T.N.	277.5	185	92.5	62	46
4-16d T.N.	331	221	110	73.5	55
16d T.N. @	16"	12"	8"	4"	6"
	53.5	71.3	107	214	142.7

STUD TO SOLE PLATE CONNECTIONS

Perhaps no other requirement of the Report exists for so many different reasons as does Detail 24/4 and yet calculation of loads justifying its use are not possible. The following circumstances played a part in the determination that some type of special connection was needed between wood studs and the sole plate at the first floor:

1. Observations of damage resulting from the San Fernando earthquake frequently revealed that a single exterior corner of a residence incurred damage well beyond that found elsewhere in the structure. This phenomenon was noted in other types of construction as well. It has been variously ascribed to overturning, inherent weakness at corners, torsion, etc. It is felt here that none of these explanations can be fully justified. Perhaps the best theory espoused to date is that a major motion of the earthquake (or perhaps the first motion) reaches one corner of a structure first and places stresses on that corner prior to the remainder of the structure being so affected. This would imply that the damage occurs almost instantaneously. Since little is known about how this damage occurs it is not possible to provide an analysis of it.

2. It was generally observed during the San Fernando earthquake that the weakest point in home construction was the connection of the studs to the sole plate. When forces were perpendicular to the wall, shear wall failures elsewhere in the house allowed the studs to separate from the plate and move outward. In most cases, such movement was minor, resulting only in the cracking of the exterior finish material. In a few cases, usually at the first floor of two-story construction, the wall moved completely away from the sole plate.

3. Because shear walls themselves have weight and therefore contribute to the seismic load, the greatest load developed in a shear wall occurs at its base. In many cases, it was also suspected that whatever shear resisting material was present had been inadequately connected at this location. When damage was slight, investigators were loathe to do further damage by removing finish materials to make this determination, and when damage was heavy, it was difficult to determine what the original connections had been since severe damage causes nails to pop, etc. In new construction built since the San Fernando earthquake, it has been observed that if shear resisting materials are improperly fastened at all there is a great tendency for such inadequate fastening to occur at the sole plate. Being close to the ground, this is the most inconvenient location to fasten and also requires continued stooping while nailing.

4. In a few cases shear walls actually tore loose from the sole plate in either uplift or overturning. In all such cases observed, the separation occurred where the studs adjoin the sole plate; that is, the sole plate remained in place relatively undamaged while the wall itself moved. Pictures of such damage are shown in Chapter 1-2 of the Report.

Various arguments can be espoused for not requiring this detail. If proper shear walls are provided for instance, it might be assumed that deflections allowing walls perpendicular to the load to be pulled off their support would not occur. It might also be argued that inspection could assure the proper fastening of shear resisting materials at the base of shear walls. This has been taken into account in the requirements for the use of Detail 24/4 in the Report. Framing anchors in all cases are required on both sides of one corner stud in each direction for all houses in Zones 2 and 3. In addition, a single framing anchor is required at the first two studs from the corner. Two-story houses in Zone 3 are required to have framing anchors placed at four feet on center when plywood sheathing is not used. It cannot be stated that the addition of the framing anchors at the corners will prevent corner damage, but such a connection certainly will strengthen what appears to be the weakest point at the corners. In addition, the installation of the framing anchors will offer some resistance to uplift caused by the vertical component of seismic motion. They will also provide a small degree of resistance to overturning of the wall. The principal reason for the addition of the staggered framing anchors at four feet on center in two-story construction is that two-story houses generate much more seismic load. These framing anchors should help compensate for any improper nailing when it occurs, but primarily will provide a small additional factor of safety for such shear walls. In view of the many functions these framing anchors serve, and especially considering the fact that they are installed at the "weak link" as exposed by the San Fernando earthquake, it is felt their use is entirely justified.

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STUD TO SOLE PLATE CONNECTIONS

NO WAY TO FIGURE REQUIRED LOADS TO THESE ANCHORS. WHERE USED AT 4'-0" o.c. (2 STORY-ZONE 3) THEY WILL PROVIDE THE FOLLOWING:

$\frac{400}{4} = 100$ #/1 UPLIFT RESISTANCE

$\frac{313}{4} = 78$ #/1 SHEAR RESISTANCE

$\frac{267+400}{8} = 83$ #/1 BASE REACTION PERPENDICULAR TO WALL

CHORD SPLICES

In comparison with larger commercial buildings, diaphragm chords are relatively unimportant in residential construction. This is true partly because of the size of residences but primarily because of present construction techniques. The top plates of exterior wood frame walls are usually doubled, with splices in these plates staggered. One plate from intersecting perpendicular walls is usually extended across and nailed to the otherwise continuous exterior wall plates. The exterior plates act as the chords for the roof and floor diaphragms. Intersecting crosswalls provide some support, even when these walls are not designed as shear walls. If the exterior walls perpendicular to the chord are considered to be fully designed walls with zero relative deflection, other interior crosswalls can then be assumed to be additional supports having in most cases less rigidity than the exterior walls. The situation might be likened to a beam over several supports with only the end supports unyielding and all others mounted on springs with varying spring constants. In most cases, interior walls provide sufficient support to the chord to reduce diaphragm deflection and thereby reduce any tension or compression in the chords.

Wings of houses are usually much smaller than the main body of the house and therefore develop lower chord stresses even though the length to width ratio in plan view may be greater than occurs in the portion of the house which might be considered as the primary load. For this reason, the accompanying calculations at the end of this section assume a 25'-0" depth as being the minimum depth for which it might be assumed that two interior walls parallel to the chord would be encountered. These calculations indicate that for the heaviest loads which might be expected, the upper plate splice required would be acceptable for spans up to 31 feet with no intermediate supports. As mentioned above, it is unlikely that interior crosswalls would not be encountered within this distance. These interior crosswalls will provide some support for the chord and reduce deflection to a degree sufficient to further extend the 31 feet calculated. The Report frequently discusses the fact that it is felt that the ceilings of conventionally framed houses act as the principal diaphragms, with the roof diaphragm carrying only its own weight and some contribution from exterior walls. This consideration not only substantially reduces the seismic weight taken by the overall "diaphragm" and thereby transferred to the chords, but also reduces the span since interior walls not connected to the exterior walls also act to reduce total deflection of the structure. It is felt that the chord splice developed is sufficient for all cases mentioned and, in addition, supplies splices capable of transmitting significant strut loads, when seismic direction is perpendicular to that which would require the top plates to act as a chord.

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CHORDS

TRY 9-16d BETWEEN SPLICES SINCE TOP PLATES MUST FREQUENTLY ACT AS A STRUT AS WELL

9-16d - $P_{ALL} = 9 \times 107 \times 1.33 \times 3/4 = 963$#

IF 25' IS CONSIDERED A MINIMAL DEPTH OF MAIN PORTION OF HOUSE:

$M_{ALL} = 25 \times 963 = 24075$'#

USING MAX LOADS AND 2 INT. PARTITIONS

W_{SEIS} = ROOF - $29 \times 20 \times .133$ = 77.3
CLG - $25 \times 10 \times .133$ = 33.3
WALLS - $4 \times 4 \times 20 \times .133$ = 42.7
VENEER - 2×21.33 = 42.7
196.0 #/'

$196 L^2/8 = 24075$

$L^2 = 982 \quad L = 31.3'$

CHORD OK FOR SPANS TO 31' BETWEEN PARALLEL WALLS

9-16d BOX

STRUTS

As stated in the Introduction, calculations have not been shown for simple connections of the type designed in the office on scratch or directly on a calculator. The struts indicated in the Report develop very low compressive or tensile forces in the wood members and, hence, no calculations have been shown for these members or their connections. Roof live load is commonly not required to be considered as acting together with seismic load, and, in addition, a one-third increase is allowed for lumber stresses. As stated in the section in this report title "Split-Level Ties", a 62 percent increase could be taken for the lumber instead of the 1.33 increase and still have the same factor of safety as is used for vertical load based on 1 to 10 years duration of loading. Although it is impossible to ascertain exactly what stresses will be developed in individual rafters, floor joists, top chords of trusses, etc., it is reasonable to assume that the omission of the roof live load and the one-third increase will more than compensate for the relatively low stresses induced by the struts shown in the Report. The worst case indicated occurs for Detail 44/4 in the Report wherein solid blocking is required to take a load of 2400 pounds. In this case, of course, there is no vertical load figured on the blocking and the compressive stress in a 2 x 4 would be approximately 457 psi. For the two alternate Details shown as Detail 41/4, the truss has been doubled in Detail 41a/4 and the rafter is indicated as being 2 x 6 minimum in 41b/4. In this case, the stress in the 2 x 6 (worst case) is 291 psi.

Although floor members must be figured using the live load plus the lateral seismic load, the 40 psf live load results in a larger floor joist size with the result that the one-third increase is the same increase in stress, while the axial stress due to strut action is decreased because of the increase in the net cross-sectional area of the member. The result is that stresses for the struts shown are so low as to be acceptable "by inspection."

SPLIT-LEVEL TIES

There is little question that split-level homes suffered much greater damage in proportion to their number than any other category of home construction. The most spectacular damage to split-level homes was caused by factors unique to a particular tract, but even when this tract is discounted, two specific mechanisms of failure peculiar to split-level houses were apparent. One of these is the normal lack of ties between the mid-level and the two-story portion of such homes. The second, lack of shear wall at the garage, is discussed under "Rigidity Analysis."

In concentrating on split-level homes and referring to these connections as split-level ties, it is apt to be forgotten that the same principles apply in any structure where there is a vertical offset in roof or floor diaphragms. Because of the offset in levels, the flexibility of the wood studs at the point of connection, possible differences in the period of vibration of the two sections of the structure and other similar problems, it is difficult to develop a tie that is applicable to offset roof levels in general.

In one tract severely damaged by the San Fernando earthquake, two split-level houses collapsed. Another house in the same tract was of interest to many engineers, inspectors and building contractors, in that a strap tie had been placed between a framing member at the mid-level and a header at the common wall between the two sections. Although memory no longer allows a complete visualization of this detail, it was apparent to all who viewed it that the strap was still under considerable tension. The point is that this house, although severely damaged, did not pull apart at the juncture of the two levels although oriented in the same direction and only two or three doors away from an identical house which had collapsed. The lesson here seems to be that it is far more important to provide ties of some sort than to determine their exact capacity or the required load to be transferred.

As the accompanying calculations indicate, an attempt has been made to determine the required load for which the ties should be designed. If Model C as shown in the Report is used as the typical split-level home, total load transfer would indicate a load per foot of 106 pounds. This house represents very light finish materials on both walls and roof. If the heaviest materials available were used the total seismic load would be more than double. If stucco exterior and lath and plaster interior finishes were utilized, along with a slightly heavier roof, the load would be approximately double.

A second viewpoint might be to assume that the loads generated by the walls and ceilings were taken by the mid-level walls and the transferred loads would be equal to the roof weight only. The heaviest roof would generate an equivalent seismic load of 2.667 psf as shown by the calculations at the end of this section. If ties were provided for 100 pounds per foot, the entire roof load could be transferred if the diaphragm length did not exceed 37.5 feet.

The last attempt to reconcile loads to be transferred through the ties was to assume that all walls acted together despite the fact that the mid-level and the first story of the two-story portion are a half story apart. In this case, it was found that the load to be transferred would be approximately 51 pounds per foot for Model C or 100 pounds per foot for houses with heavier finish materials. Based upon all these considerations, it was determined that the ties should be designed for a load of 100 pounds per foot in Zone 3 and 50 pounds per foot in Zone 2. Rather than vary the details, connections perpendicular to the rafters are required to be spaced at 8 feet on center in Zone 3 and at 16 feet on center in Zone 2. In this way, each tie is required to have a capacity of 800 pounds. The connections given in the Report were designed for this load. The studs to which the connections are made do not have the same capacity as the connection, however. In addition to the 1.3 factor of safety normally assigned to wood, an additional factor of 1.62 is applied for short term loadings. Even though this 1.62

factor was used in place of the normal one-third increase for seismic loads, the required stress in a single stud was found to be 1865 psi. This would require the use of special studs at these connections and provide not only complaints from builders but also the chance for confusion in installation of studs of lesser grades. For this reason, 1600f was chosen as the required strength for the studs in anticipation that most contractors would halve this and use double studs of 800f. These stresses are only developed when the connection occurs at the exact mid-height of the stud. Since most roofs using this detail will be sloping, some of the connections will actually come much closer to the 800 pounds (at working stress) than others.

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SPLIT LEVEL TIES

PURPOSE OF TIES IS TO REQUIRE THE TWO SECTIONS OF THE HOUSE TO MOVE TOGETHER - NO WAY TO FIGURE ACTUAL LOAD REQ'D TO BE TRANSFERRED IN ORDER TO ACCOMPLISH THIS. DEFLECTION OF STUDS WILL ABSORB SOME ENERGY AND BENDING STRENGTH OF STUDS MAY LIMIT LOAD.

AT MODEL C - TOTAL SEISMIC LOAD AT MID-LEVEL
IS 1080 + 1680 = 2760#

$$\text{LOAD/FT} = \frac{2760}{26} = 106\#/'$$

IF LATH & PLASTER, STUCCO, A HEAVY ROOF ETC. WERE USED LOAD WOULD BE APPROX. DOUBLE

IF ~~ROOF~~ LOAD ALONE WERE TRANSFERRED

HEAVIEST ROOF SEIS. LD # CONC TILE - 2.133
SHTG - 0.267
FRMG - 0.267
2.667 PSF

IF ALL WALLS ACTED TOGETHER

TOTAL SEIS LD = A - 2134
B - 1080
C - 3054 (ADJ. ELIMINATED)
D - 1680
E - 2134
10082

TOTAL WALL L = A - 13.0
B - 12.0
C - 18.33
D - 13.0
E - 4.0
60.33

SPLIT LEVEL TIES

IF WALLS ACTED TOGETHER (CON'T)

LOAD TAKEN BY MID-LEVEL $= \frac{25}{60.33} \times 10082 = 4178^{\#}$

LOAD TO BE TRANSFERRED $= 4178 - 2760 = 1318^{\#}$

LOAD/FT $= \frac{1318}{26} = 50.7\ \#/1$

IF LOADS WERE DOUBLE $-$ 101.4 #/1

DESIGN TIES FOR 100 #/1 (ZONE 3)
50 #/1 (ZONE 2)

TRY TIES @ 8'-0" o.c. (ZONE 3) & 16'-0" o.c. (ZONE 2)

$P = 800^{\#}$

CHECK STUDS $-$

$L = 8'\text{-}1" - 3 \times 1\tfrac{1}{2}" = 7'\text{-}8\tfrac{1}{2}"$ $\quad M_{max} = 800 \times 7.71 \times 12/4 = 18504''^{\#}$

$f = \frac{18504}{3.063 \times 1.62} = 3729/2 = 1865\ psi$

1.62 IS USED RATHER THAN 1.33 SINCE WOOD FACTOR FOR SHORT TERM LOADING IS 1.62.

TRY 1600 f $-$ $P_{ALL} = \frac{1600}{1865} \times 100 = 85.8\ \#/1$

BUT 5% OF STUDS WOULD THEORETICALLY BREAK AT $85.8 \times 1.3 = 111.5\ \#/1$

USE 1600 f SINGLE STUD OR 800 f DOUBLE

LOADS PERPENDICULAR TO MASONRY WALLS

Section 2313 of the Uniform Building Code states "concrete or masonry walls shall be anchored to all floors and roofs which provide lateral support for the wall. Such anchorage shall provide a positive direct connection capable of resisting the horizontal forces as specified in this Chapter or a minimum force of 200 pounds per lineal foot of wall, whichever is greater. Required anchors in masonry walls of hollow units or cavity walls shall be embedded in a reinforced grouted structural element of the wall." The Uniform Building Code does not specifically exempt residences from the quoted requirements. The One- and Two-Family Dwelling Code, 1971 Edition, has been authorized by BOCA and the Southern Building Code in addition to the ICBO, and also requires connections at the top of masonry walls adequate to resist 200 pounds per foot.

In this instance, the Report is less conservative than the other publications. Commercial buildings with nine inch exterior brick walls having a height of 16 feet to the roof sheathing and a three foot parapet, would almost exactly develop the 200 pounds per foot required. This type of commercial building is very common and offers no factor of safety over the minimum requirement. It is felt that 8 or 9-foot high masonry walls in residential structures offer much less of a threat than the aforementioned higher walls. The actual load developed at the top of a masonry wall 8'-8" high (maximum height recommended in the Report) is shown by the accompanying calculations to be 78 pounds per foot maximum. By requiring ties good for 100 pounds per foot for Zone 3, it is felt that the ties as detailed in the Report are more conservative than those present in many commercial structures. Using similar reasoning, the ties in Zone 2 have been designed for 65 pounds per lineal foot. This is predicated on a 15 psf wind load. In wind zones higher than 15 psf, it would seem logical to require the ties stipulated for Zone 3 regardless of the seismic zone. Since the Report is not primarily designed for wind loads, this statement has not been made therein.

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LOADS PERPENDICULAR TO MASONRY WALLS

ASSUME 9" BRICK WALL AS WORST CASE

$W_{SEIS} = 90 \times 0.20 = 18$ #/1

FOR TYPICAL CONST – $P_{ROOF} = 4 \times 18 = 72$ #/1

AT 8'-8" HIGH – $P_R = 4.33 \times 18 = 78$ #/1

UBC REQUIRES ALL MASONRY WALLS BE TIED AT TOP FOR 200 #/1 AND ONE AND TWO FAMILY DWELLING CODE INDICATES THE SAME FOR ZONE 3 (NO DETAIL FOR ZONE 2) BUT REINFORCED WALL HEIGHTS ARE ALLOWED TO 17'-0", SINCE REPORT DEALS WITH WALL HEIGHTS OF 8'-8" MAX, USE <u>100 #/1 IN ZONE 3</u>

<u>IN ZONE 2</u>

$\frac{1}{2}$ SEISMIC = 50 #/1

USE 15 PSF WIND AS GOVERNING

$4.33 \times 15 =$ <u>65 #/1</u>

BASEMENT CONNECTIONS FOR EARTH PRESSURE

In many areas the backfill placed against basement walls provides an equivalent fluid pressure of 30 pounds per foot per foot. Since most basement walls are 8 feet in height, the Report has provided a connection adequate for the reaction at the top of the wall created by this equivalent fluid pressure over a height of 7'-4" and stipulates that it may be used for earth heights up to 7'-6". This is virtually the only connection shown in the Report which is not directly associated with seismic activity. Many basement walls presently have little or no connection at their top, but remain standing with little, if any, distress. Seismic activity does not appear to overstress basement walls of houses designed for the earth pressure mentioned above. None of the damage literature reviewed[4] indicated that earthquakes caused problems for basement walls of houses. In Southern California, there were relatively few basements and in the Alaskan earthquake, it appears principal problems were caused by earth subsidence rather than shaking. This requirement can and has been supported by engineering calculations, but in view of past performance, is considered to be one of the least critical items detailed in the Report.

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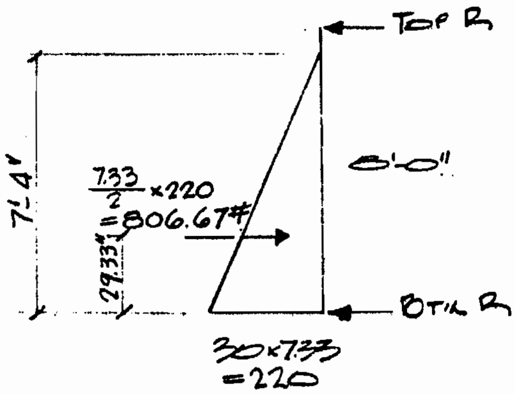
BASEMENT WALLS - SUPPORT FOR RETAINING ACTION.

ASSUME 'TYPICAL' CONDITION - 8'-0" HIGH
30#/'/' EQUIV. FLUID PRESSURE



$$\text{TOP } R = \frac{806.67 \times 29.33}{96} = 246.48\#$$

$$\text{BTM } R = 806.67 - 246.48 = 560.19\#$$

$$\text{TOP } R @ 16'' = 246.48 \times 1.33 = 328.64\#$$

$$@ 4'0'' = \quad '' \quad \times 4 \quad = 985.92\#$$

FIREPLACES AND CHIMNEYS

As stated under Item 10 on page 10 of Reference 4: "Attempting to devise strap ties which seem reasonable and still work in every case is virtually impossible." In the accompanying calculations at the end of this section, the fire box itself is assumed to be stable on its foundation with only the chimney or the equivalent weight of chimney contributing load to second floor or roof diaphragms. Assuming a slightly larger than average chimney, it was found that the weight of the chimney times the normal Zone 3 seismic coefficient resulted in a load of approximately 600 pounds to the building. Normally the chimney ties would be designed for 1.5 times this load. After reviewing a number of fireplaces and chimney details and considering the calculations herein included, it was felt that the Los Angeles City Detail provided all requirements reasonable to assume. The ties in this detail are adequate for a load of 1200 pounds, or twice the calculated load. The detail itself is duplicated in the Report with information not pertaining to seismic requirements deleted. Additional cross-sections are provided to make the detail clearer. Because diaphragm shear values vary the length of attachment for the strap ties must be made partially on judgment.

Despite these somewhat severe requirements, it is anticipated that masonry fireplaces constructed in accordance with the Report will continue to experience distress, although it is anticipated that overall damage to these structures will be reduced. Strap ties will continue to be left unconnected and/or poorly bonded unless close inspection is made, large or heavily veneered fireplaces will generate loads in excess of those for which the strap ties have been designed, reinforcing will be incorrectly placed due to the configuration of a particular fireplace and in a few instances, strap ties themselves will fail. Despite this prediction, it is questionable that is is practical to require much more than the detail in the Report offers.

Although some fireplaces will generate loads in excess of the 600 pounds required to be added into the structure when masonry fireplaces are added, it is doubtful that overstress to shear walls created by this additional load will have a large effect upon the overall structure. Earthquakes vary markedly in intensity and the loadings from these variations will play a much larger role in dwelling performance than the 8 or 10 percent possible overstress to shear walls caused by larger than average fireplaces.

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FIREPLACE & CHIMNEY

ASSUME FIREBOX WILL ACT AS A CANTILEVER OFF FOOTING — THEN WORST CASE IS 9'-0" HIGH CHIMNEY LOAD TO 2ND FLOOR OR ROOF.

ASSUME SLIGHTLY LARGER THAN AVERAGE CHIMNEY — 2'-0" x 2'-6"

WT OF CHIMNEY = 6.33 x 80 = 507 #/'
SEIS. WT = 507 x .133 = 67.5 #/' TO BLDG
P TO BLDG = 67.5 x 9 = 608 # — SAY 600 # - ZONE 3

CONNECTIONS — DESIGN FOR DOUBLE RATHER THAN 0.20

WATER HEATER TIES

It is hoped that this detail will be widely adopted in areas of heavy seismic activity, as opposed to its requirement for homes built with FHA guaranteed loans only. An attempt has been made to make the detail as simple as possible in order to achieve this goal. It is recognized that other ties may be developed which are either better or less expensive to fabricate or install, and that, in addition, the detail as shown may not work for all water heaters. To achieve economy, the detail is predicated on the manufacture of a pre-fabricated item which does not presently exist. It is believed that the requirement of this detail for homes built with FHA financing will provide a market large enough to warrant the manufacture of this item.

The connection itself is quite light but should provide sufficient staying to prevent the overturning of virtually any water heater. It cannot, of course, assure that the legs on the heater will not buckle under seismic loads, but even in this circumstance, it should allow the water heater to remain in a semi-upright position. The connection was initially intended to be required only when flexible couplings were used, but a statement in Reference 3 indicated that rigid connections were not a deterrent to overturning. Since water heaters are frequently replaced and are not of uniform height, it will not be surprising to find, in a future earthquake, that these ties have not been installed on replacement heaters in many instances. It is not believed that this realistic assessment should prevent the attempt to have them installed as frequently as possible, however.

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WATER HEATER TIES

ASSUME 75 GAL. HTR. AS LARGEST

WEIGHT - WATER - 75×8.34 = 625
TANK - SAY 100
725#

SEISMIC LD. AT TOP OR BTM = 725×0.2×1/2 = 72.5#

FOR 2 1/4×2 1/4×16 GA L -

$(2.19\times.0598)\times1.125 = 0.13096 \quad 0.14733$
$(2.25\times.0598) = 0.13455$
0.26551

$x = y = \dfrac{0.14733}{0.26551} = 0.555''$

$I_x = I_y = 1/3(.0598(2.25-0.555)^3 + 2.25\times0.555^3$
$-2.19(0.555-.0598)^3 = 1/3(0.29121+0.38465$
$-0.26594) = 0.13664$

$r_x = \sqrt{\dfrac{.13664}{.26551}} = 0.717$

$I_z = 0.13664 - (.13664)^2 = 0.11797$

$r_z = 0.667 \qquad \text{MAX } L = 200\times0.667 = 133'' = 11'\text{-}1''$

L OK

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