EARTHQUAKE RESISTANT DESIGN OF SMALL BUILDINGS

By

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The problems concerning the design of small buildings to resist lateral forces have been largely neglected in the engineering literature. Most of the research and the development of mathematical tools for studying earthquake response have concerned themselves with the larger structures or the structures whose dynamic properties can be rather simply expressed in mathematical terms. In addition to this, engineers and architects in most parts of the United States, at least, even today do not consider lateral loading on small structures to be an engineering problem, and many building codes completely ignore it. From the newspaper accounts of building failures in various parts of the world, it is probable that this same situation with regard to lateral loads on small buildings is common in other countries. It is somewhat strange that this condition should exist, since in every country in the world, without exception, the small buildings make up the vast majority of the total construction. This fact alone should concentrate research and study on the problems of the small building for two important reasons. First, the majority of the population live and work in and are otherwise concerned with the safety of small buildings. More lives are lost in earthquake failures of small buildings than in any other kind. Secondly, more total construction cost is involved in the construction of the small buildings than in any other class of construction. As a consequence, research into possible economies in the design or construction of small buildings offer more possibilities of fruitful return than in any other construction field.

Let us consider some of the fundamentals upon which any engineering design is based. In approaching the design of any specific structure there are two general basic approaches; the first of these is the trial and error approach - the proof of time. In using this approach if it is necessary to span a certain distance with a wood joist floor, a certain size joist is tried. If it falls down or deflects too much, a larger size is used and the process repeated until a satisfactory floor is obtained.

Long before we had our formulae for bending moments, maximum stress, etc., builders were constructing bridges, buildings, towers, domes and other structures. If one fell down, that was unfortunate and the next one was built differently, or heavier, and eventually one was built that stood up. Experience was gained and empirical rules of thumb were developed and major structures resulted. Some of the longest timber bridges ever built were built under this method. The domes of the old cathedrals were built in this manner and many are still standing today after centuries of use. It is not a method to be dismissed lightly, as it formed the foundation for our engineering profession.

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The second approach to structural engineering problems consists of determining the forces that exist, and by applying certain laws of stress and the ability of a material to resist that stress, determine mathematically what structure is needed to resist that force with an adequate factor of safety. Instead of just applying a load to a floor joist in order to see if it will stand up, we now calculate a bending moment, and by using an appropriate fibre stress in a formula based on Hooke's Law, we determine the proper size of joist that will safely carry the required load. When all factors, relationships and data are known, this second method, of necessity, must agree with the first elementary method. When we do know all the factors, we can more closely predict the results that can be obtained only expensively and laboriously by tests.

We engineers are mathematically inclined by nature or we would not have survived the type of training we were given, and we therefore appreciate and endeavor to use this second method. While this second method is certainly valid and appropriate when all factors are known, it can certainly be misleading if certain portions of the data are not known. Yet, from our basic inclination, at times we endeavor to apply the second method before all the data is available.

Now just how do these basic approaches affect our lateral force design? At the present time our lateral force design is based on the first method. Coefficients are based on observation and past experience. The requirements as to parapets, fastening of walls and ornaments, foundation ties and general attitude of design are all based upon observation of structures that either survived a destructive earthquake or were demolished by it.

In recent years a large amount of effort has been expended in gathering data for applying the second method to the art, not science, of seismic resistant design. Several agencies in various countries have developed, installed and maintained instruments to measure basic earthquake motion and several good records are now available. Various investigators have studied the application of this data to idealized simple structures of various types. Studies have been made of how the findings on these idealized structures may compare with actual prototype structures. The records of earthquake motion indicate ground accelerations of the order of magnitude of 30 to 40 percent of gravity, or even more. By applying the spectrum response of idealized structures to various earthquake records, Professor Housner(1) has found that this has been amplified through resonance to accelerations of the order of several hundred percent. Studies of internal damping and some foundation effects have reduced this theoretical maximum again to the order of almost 100 percent of gravity. However, studies to date do not lower these theoretical figures appreciably and there is increasing pressure, in the United States, that our building code design values should be increased to a level more closely approaching this theoretical figure. In the face of these measurements and studies, earthquake experience tells us that many well designed structures have withstood major earthquakes with rather nominal percentages of calculable earthquake resistance. We have seen numerous instances of steel or timber buildings having negligible resistance to lateral forces coming through major earthquakes with absolutely no damage at all.

Certainly, to date, the results obtained by the two approaches as related to earthquake design do not agree. Based only upon observations, a rather small coefficient is adequate for many types of structures, and our theoretical second approach has not yet developed to the point where these small coefficients can be explained in useable mathematical terms.

Before proceeding further, it should be made clear that I am not disparaging any research that has gone before or is to be done in the future. If we are ever to graduate from the first crude approach to design to a true engineering approach, we will need all research possible in all directions that indicate any promise. This does not change the facts, however, as regards present day knowledge and conditions.

A fundamental characteristic of small buildings as a class is the diversity and complexity of their construction, both as regards to material, shapes, and methods of framing. Large structures are usually constructed with structural steel or reinforced concrete as the basic structural material. Small structures, on the other hand, may be built of almost anything. In the United States, timber is probably the most common construction material. It is often used as the basic structural material but it is also often used in conjunction with other materials such as masonry units in its various forms, and all of the other basic materials. While timber is basically an elastic material, when fastened together in assemblies commonly used for lateral resistance it becomes highly plastic, especially when the commonest connectors, nails, are used. As Jacobsen found in conducting dynamic tests upon timber shear walls(2) "It was soon found that the action of the nails was such that the individual tests could not be repeated on the same model with the same results; in other words, the elastic characteristics of the models, due to the loosening of the nails, changed considerably from blow to blow."

Lateral load tests on diagonally sheathed shear walls and roof diaphragms also indicate that a considerable variation in deflection may exist in repeated static loadings. Many of the other building materials common to small structures exhibit the same tendencies to a greater or less degree. The difficulties of determining definite dynamic properties of a single building composed of several different materials, none or few of them elastic, are almost insurmountable.

Small buildings come in an infinite variety of shapes and framing schemes. There is no such thing as a typical small building, but Figures 1 to 3 give some indication of the range of complexity that is characteristic of this entire class of structure.

Figure 1 shows a small bank building using a wood roof with concrete exterior walls. The lower level roof and the mezzanine diaphragms have very large span depth ratios but by creating shear walls connecting them with the roof diaphragm and tying together with moment connections, the deflections could be held within acceptable levels. It would be very difficult to determine the dynamic properties. Figure 2 shows a building containing a bowling alley with the floor at five different levels, a small mezzanine and various relative roof elevations. Any dynamic properties determined at one end of the building would have little relation to those found at any other location. Figure 3 shows an all concrete building housing a public utility. The

complex shape was required by the shape of the lot as well as the peculiar requirements of the equipment and economic factors. The thought of determining the various periods of this structure for a theoretical dynamic analysis is appalling.

It can be seen from the above that, since our research and theoretical analysis of earthquake response has not developed to the point where it is practical for use in the small buildings, present practice relies almost entirely on the first, trial and error, method of analysis. Our methods of providing lateral force resistance in small buildings is empirical, based upon observation of many buildings that have performed adequately in past earthquakes as well as those that have failed. It is common to provide for 50% or 100% G for parapets for example since observation tells us that they are extremely vulnerable to earthquakes. Moreover, any failure of a parapet creates an especial hazard on sidewalks and public ways and so extraordinary precautions are necessary to protect the public.

The same precaution, to a somewhat lesser degree, is observed in the attachment of walls since past earthquakes have indicated that many walls have fallen out of buildings, endangering the public.

The coefficients chosen for the design of the building are averages of the judgments of the engineers who have observed the effects of past earthquakes. Undoubtedly, in specific cases such as the flexible buildings illustrated in Figures 4 and 5, these may be too high, while in other cases there is quite a possibility they are too low. But the range of the problem with small structures is so great and the indeterminate factors so overwhelming, that it is the best that can be done practically at the present time. One non-scientific factor is also of major importance when evaluating our design methods. Almost without exception, small buildings are restricted economically. There are few monuments built today and the purpose of creating the building is to return some benefit to the builder, often monetary, through rent or other commercial usage. This restriction, through economy, extends to the design phases of the construction. The engineer or architect must employ some method in design that will turn out an economical safe structure with a minimum of analysis consistent with the problem at hand.

Now, in view of the above general background, we must evaluate some of the requirements necessary for adequate design to resist earthquakes. First and foremost, the whole attitude of the engineer must be changed somewhat. In vertical load design, the loads are known. The designer can analyze his loads and how they are to be carried with considerable certainty. From years of research, development and study as a background, the requirements of the actual building can be readily estimated and the performance of the building and its resultant factor of safety can be predicted with a reasonable degree of precision. To a somewhat lesser extent, the same situation exists in designing for wind loads. When considering earthquake effects, however, the direct opposite is true. The effective loads on the building are not known and cannot be predicted. In every instance, the designer must bear in mind that his assumed forces - the forces specified in building codes - may be considerably exceeded at least in parts of his structure. The solution to

this problem does not lie in choosing high design coefficients which would make the small structure economically impossible. It is rather in the attitude of the designer in providing a tough structural system which can resist shock well, even though it may not withstand the high static forces often assumed. One of the most typical and important results of this approach is the proven necessity for tying the various parts of the structure together. Probably the commonest immediate cause of structure failure in an earthquake is the lack of adequate coherence of the structural parts as illustrated by the disintegration of unreinforced unit masonry, the falling off of parapets, collapse of walls, and separation of wings from the body of the structure.

Second in importance only to the attitude of the designer, are the materials of construction and the related framing scheme chosen. Certain materials and framing methods have considerable inherent resistance to earthquake loadings. Figures 4 and 5 illustrate certain buildings that had no earthquake damage at all in spite of having negligible calculable lateral resistance. They went through the same earthquake that caused the failure of the building shown in Figure 6 which used materials and details proven to be particularly vulnerable in many earthquakes.

There are two basic method of providing for the horizontal forces created by the earthquake ground motion. In specific structures, the designer may choose one method or the other depending upon conditions, but, because of specific utilitarian requirements it is probable that combinations of both methods will be required in many structures. For simplicity here, we will consider only each specific method and not the combination. The first method resists these forces through moment connections or knee braces as illustrated in Figure 7. This is typical of many types of mill structures, light sheds and other places where shear walls cannot be used for architectural or utilitarian reasons. The forces are carried to the ground through bending of the columns and girders. This bending resistance must be developed at the joints.

This method has the advantage that large open spaces can be created without interference from permanent bracing walls. It is often forced upon the engineer in order to achieve the open effect of large glass areas prevalent in modern architecture. The framing can absorb large amounts of energy and can probably withstand a larger earthquake with less chance of total collapse than any other type of framing. It has two major disadvantages. In general, it is a more expensive method of framing since the member sizes are often larger than those required by vertical loads alone and the connections are more complicated and hence more expensive. However, its major disadvantage lies in the fact that it is usually somewhat flexible. This flexibility denies the proper support of the more expensive finishes common to many of our buildings. Plaster, tile, masonry, and veneers of all types are usually quite rigid and brittle and must be supported with a minimum of deflection.

The second base method of providing for lateral forces is through shear walls and diaphragms. The action in this method, illustrated in Figure 8, resembles the action of a box. In most cases, this is the most economical

method of providing for lateral forces. It is by far the stiffest method and therefore gives the best protection against property damage especially to brittle finishes such as plaster. Its economy lies in the fact that it utilizes building components such as walls, roofs and floors already provided in the building. In the United States, this method of bracing is used in the vast majority of the small buildings. It is so prevalent, and the engineering literature is so meager, that it would be well to review the basic design principles here.

Consider the idealized simple structure shown in Figure 8, where we have a simple, box-like, one-story structure. The inertia loads of the side walls (or wind pressure and suction in the case of wind) act on the walls. The wall framing spans from the foundation to the roof as a beam and must be designed as such in addition to the column action due to the vertical loads in the roof. The top of the wall thereby produces horizontal loads against the roof framing. The wall must be attached to the roof framing strongly enough to resist the forces involved. The roof framing now acts as a large girder taking the horizontal reactions of the walls and the roof's own inertia horizontal loads and spans to the end walls which must resist these reactions.

The roof system that carries these loads is called the diaphragm. In some cases, it is actually framed with diagonal rods or angles to form a bracing truss, which is the only case generally considered in engineering literature. From observation, a few tests, and considerable engineering intuition, the custom has developed of using the actual roof framing, that is, the wood or plywood sheathing, concrete or metal deck or other construction materials to serve as the structural element to resist these lateral loads. How is this done?

Considering this roof diaphragm as a girder with the external loads shown, we see that there are the usual shear and moment forces to resist. Generally, chords are provided to resist the moment stresses. The roof material itself resists the shear stresses. This sounds simple, and in some materials such as with concrete or plywood, it is.

In Figure 9, the roof is spanned in the short direction with roof rafters, while sheets of plywood form the roof sheathing. The stress action is similar to that of a large plate girder with the plywood resisting the shear forces and the continuous chords on the long sides resisting the moment forces. It is usually assumed that the web does not help to resist the moment forces. The nailing of the plywood is calculated from the shear forces similar to the web rivet design for a steel plate girder. Many tests of this type of construction have been made in the United States by the Douglas Fir Plywood Association.(3)

Figure 10 illustrates the type of diagonally sheathed diaphragm which has been tested most often to date. Here, too, the roof is spanned in the short direction by roof rafters, while sheathing boards span the space between rafters at a 45° angle. These sheathing boards are usually 1" x 6" or 1" x 8" boards nailed at each joist with two or more 8 penny nails,

depending upon the calculated stress. Again, the continuous chords on the long sides resist the moment forces, while the diagonal sheathing resists the shear forces. As shown in Figure 10, the sheathing boards are in tension but since lateral loads may come from any direction, the loading can be reversed and the boards would be in compression. Although there may be considerable bending in the sheathing boards, tests verify (4,5) that the primary load resistance in an efficient diaphragm is due to the axial stress in the sheathing boards. The nailing is calculated from this axial stress. As may be seen from Figure 10, there is a component of stress perpendicular to the end posts that puts it in bending. The end post must be connected to the longitudinal chords adequately to resist this stress component. There are similar bending components perpendicular to the long continuous chords, but as in this example, the effect is usually minor because of the comparatively small spacing between rafters, which resist these perpendicular components.

Figure 11 illustrates a similar type of framing, using a double layer of diagonal sheathing to resist the shear forces. This system has not been tested since 1935.(6) It will be noted that in this system one layer of sheathing is in tension while the other layer, at right angles to it, is in compression. The forces perpendicular to the end posts cancel each other and so no bending tendency results. This system can be completely analyzed by simple statics and the maximum strength is limited only by the nailing room provided at the various contacts and intersections. Structures at the Golden Gate International Exposition in 1939 successfully resisted shears of over 4,000 lbs. per foot by this method.

Figure 12 shows a most common method of framing a roof which, to date, has not been tested except for one full size test in Los Angeles in 1952.(7) However, by comparison with tests on the diaphragm shown in Figure 10, the following inferences may be drawn:

The loads from the side walls are delivered to the longitudinal girts which span to the trusses. The end reactions of the girts are delivered through the trusses to the diaphragm, usually through blocking over the truss chords. The girts are spliced for axial load to provide chords to resist the moments on the diaphragm as a whole. The shears on the diaphragm are resisted by the sheathing boards, again primarily by their axial loads, similar to those illustrated in Figure 10.

In this case, the force components causing major bending stresses in boundary members occur at the girts. It can be seen in the figure that as the sheathing boards take their axial tensions as shown here, the girts are pulled inward, similar to the end posts of Figure 10, and must be designed for this bending force. The axial forces from the sheathing at the end walls also tend to pull these walls in, but here the span is only between purlins. The purlins should be spliced throughout the length of the diaphragm to balance the similar forces at the opposite end of the building.

The tests to date and the analysis so far have assumed that the diaphragm lies essentially in a plane so that forces normal to the sheathing

may be neglected in analyzing the diaphragm action. However, it is common practice to use either bowstring trusses or pitched trusses of one type or another and, consequently, the diaphragm is either pitched or curved. Some studies have been made indicating the secondary forces that may be caused by pitching or curving the diaphragm.(8) Similar tests have been run on diaphragms of other materials such as metal decks and poured gypsum.(9)

At this stage of the discussion, it is appropriate to consider what properties are desirable in a diaphragm that will make it most suitable to resist earthquake forces. Naturally, since it is serving as part of the structure, it should be adequately strong to resist the normal vertical loads, it must not deteriorate with time or weather, and it must be reasonably economical in its application. The diaphragm must have some reserve of strength so that it can act after normal deflections have taken place. For example, in Figure 13 when reasonable foundation settlement takes place or truss deflection occurs, its value should be relatively unimpaired. The assembly as a whole must be capable of some degree of analysis in order to predict its strength, and its strength must be proven by test. And the diaphragm must be able to resist deflection.

As seen in Figure 14, the loads on the diaphragm will cause it to deflect. This deflection allows the walls, in turn, to deflect. If the wall is stiff, is short, is brittle, or is fixed at the base and the diaphragm deflects too much, the wall can be cracked.

By assuming the wall to be fixed at the base, it can be shown that

$$\Delta = \frac{96H^2 f}{Ed}$$

where Δ is the deflection in inches, f is the fibre stress in pounds per square inch and E the bending modulus of elasticity, H is the height of wall in feet and d is thickness in inches.

In a condition where the wall is fixed as by a loading dock in Figure 15, or by construction in a story below, this or a similar formula may be very useful in determining the limiting deflections. However, no stretch of the imagination could indicate any validity in the other cases shown such as in a wood stud wall where rotation takes place in the plates and not bending of the studs, or in a masonry wall where the foundation construction does not fix the base. As a result, larger deflections might safely be allowed than indicated by the above formula.

At present, there are no suitable criteria regarding the desirable properties in a diaphragm. As can be seen, the designed faces a dilemma in choosing the proper material for diaphragm action. On the one hand, as can be seen from the discussion above, a rigid diaphragm is desirable and may even be necessary to provide adequate support for the various walls and finishes. On the other hand, it appears that a little flexibility aids a structure to survive a major earthquake, especially when the damaging

"forces" are not known. In fact, the loads may vary inversely with some function of the flexibility. We have seen instances where well designed buildings with rigid elements have failed in such a manner as to indicate that it required a large force to cause failure. Permitting the diaphragm to deflect as in Figure 14 relieves the load on the diaphragm because of the flexibility. And we have numerous instances of where this flexibility must have actually permitted buildings with very little lateral resistance to successfully weather rather large earthquakes. So it is quite probable that some flexibility is desirable, but how much is the question. The flexibility also permits a secondary action with regard to walls and parapets, probably increasing the amount of strength and attachment necessary.

At the present time, there is no accepted criteria for judging the relative merits of diaphragms and each case must be evaluated by the structural engineer based primarily on his judgment. It is to be hoped that some reasonable criteria as to strength, rigidity, "toughness", all as related to proper design coefficients, can be determined soon by some responsible group.

Shear walls can be designed in a somewhat similar manner as the roof diaphragms. It is important that all stress transfers are adequately provided for in the design and that the overturning forces are taken care of. The overturning forces can be quite a problem, especially when walls of light construction materials are involved. Tests of timber shear walls have been conducted by the Forest Products Laboratory.(10) These tests reveal the importance of proper details at corners and joints if a strong, rigid wall is to be obtained.

It may be assumed from the foregoing discussions that the art of design of small buildings for earthquake forces is in a very elementary and unsatisfactory state at the present time. It is true that it is in an elementary state as compared to the status of vertical load design. However, our empirical methods of design have furnished a reasonable basis for many practical design problems because it has been based on observation.

Just because the design method is empirical does not, per se, mean that it is poor. Even with its faults, it satisfies a need that is present now. Many structures that have performed satisfactorily in recent earthquakes have been built by these methods. The empirical method now used has the major advantage that it is rather simply applied and can be used on the most complex structures. It is a method that is suitable in the average design office because it can be quickly and economically used, and on small structures especially, this is an important factor.

In 1933, as a result of the Long Beach earthquake(11, 12), the State of California enacted the so-called "Field Act"(15) requiring that public schools built thereafter in California be designed for earthquake forces to insure safety of construction. Since that date, there have been several destructive earthquakes in California, notably Imperial Valley, 1940, (14), Kern County 1952, (15,16), Eureka, 1954, (17), and San Francisco, 1957, (18). In these earthquakes, there was considerable damage to the older schools

built before 1933 and in other structures as well. Although many new schools had been built in the area of major damage, not one school constructed under the Field Act suffered any structural damage to the degree that would have created a hazard to life or limb. In a few notable examples, pre-Field Act schools, badly damaged, were adjacent to or very near newer schools that were completely unharmed. In general, it can be stated that any of the small buildings built in accordance with the various building codes in California, empirical and crude though they may be in their earthquake requirements, performed very well.

CONCLUSIONS:

From the general discussion presented above, the following conclusions can be drawn:

1. Our present lateral force coefficients and design practices are generally based on observations of structural behavior. Many of the old buildings used rather flexible diaphragms. New materials should have basically similar mechanical properties to those used in the past in order to use the same coefficients. If the mechanical properties of new materials are substantially different, other and possibly larger coefficients may be necessary to assure safe construction.

2. The art of theoretical analysis has not yet developed to the point where mathematical predictions of structural behavior will agree with the actual behavior in major earthquakes.

3. When loads are known, such as wind forces, diaphragms should be as rigid as possible.

4. Under seismic and shock loadings some flexibility or at least toughness of a structure is desirable, based on observations of past performances.

5. Diaphragms are useful, economical tools in designing structures to resist lateral loads.

6. Diaphragms must serve to economically resist vertical loads, without deterioration through exposure, and must have sufficient reserve strength to function adequately in spite of secondary stresses caused by normal deflections, foundation settlement and other circumstances.

7. It is of great importance that the various components of a structure be adequately tied together so that the structure will act as a unit.

8. In the final analysis, the producing of safe earthquake-resistant structures for the public depends basically on the experience and

knowledge of the designing engineer rather than on detailed building-code requirements. The engineer who knows the requirements of a project must choose the proper materials for each specific job. He must connect the materials together so as to provide a coherent unit with sufficient lateral resistance to satisfy the particular requirements and yet he must maintain strict economy for his client. The problem is too complex for the provisions of a "handbook" building code.

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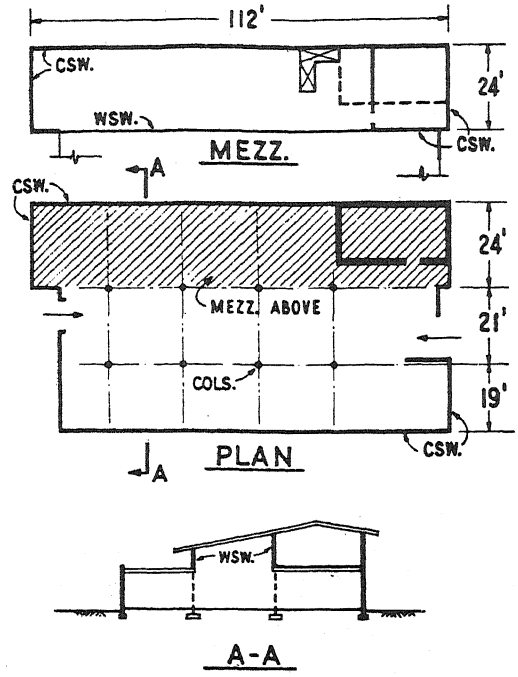

CSW. = CONC. SHEAR WALL
WSW. = WOOD SHEAR WALL

FIG. No. 1

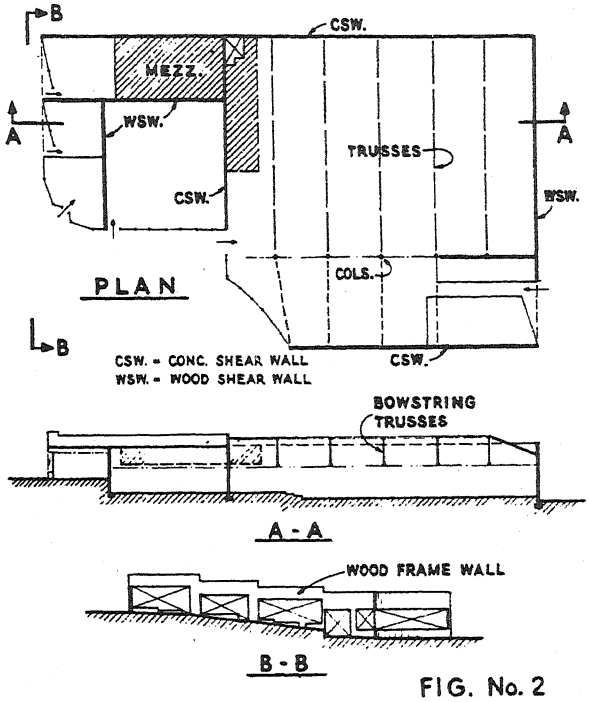

FIG. No. 2

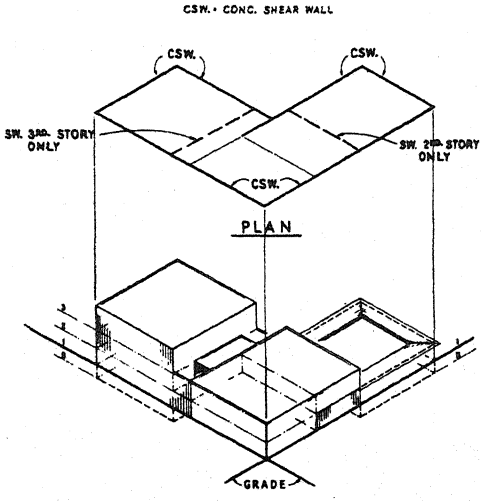

FIG. No. 3

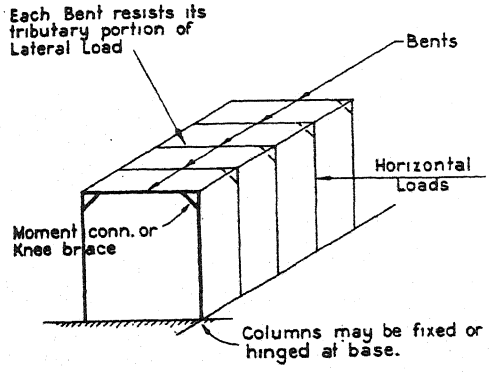

FIG. No.7

FIG. No. 4

Corrugated Iron Warehouse, Tehachapi, July 21, 1952. This structure has a timber frame without bracing and was not damaged in the 1952 Earthquake.

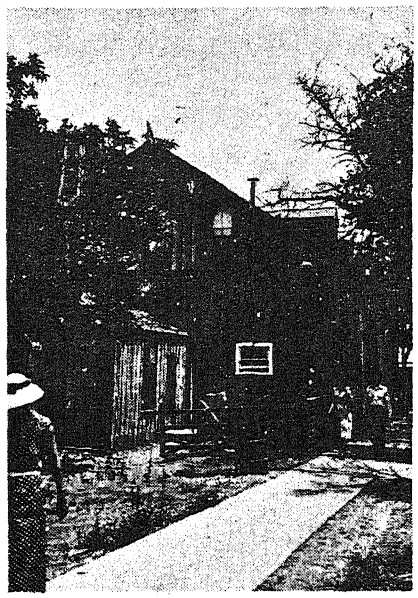

FIG. No. 5

Clark Hotel, Tehachapi, July 21, 1952. This old Board and Batten wood structure several inches out of plumb and with no calculable bracing was undamaged by the 1952 Earthquake.

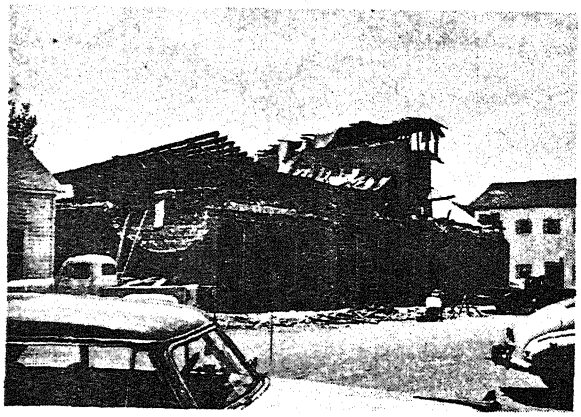

FIG. No. 6

Lodge Hall in Tehachapi, July 21, 1952. Ordinary construction using unreinforced brick walls and wood roof. The roof over the 2nd. floor Auditorium now rests upon the piano and chairs after the walls collapsed.

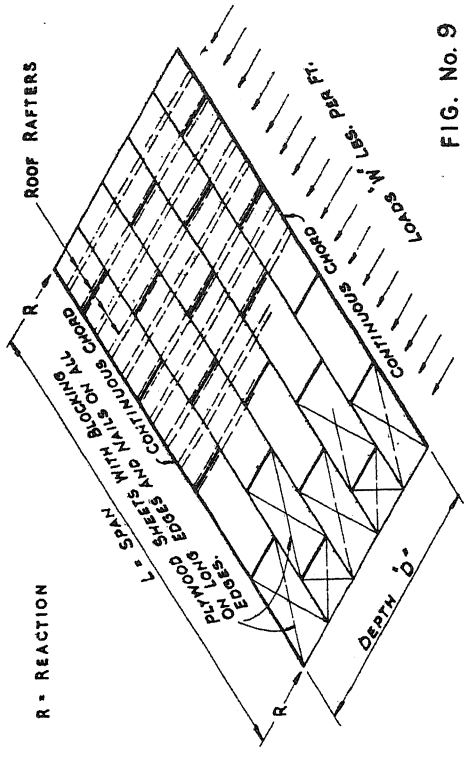

FIG. No. 9

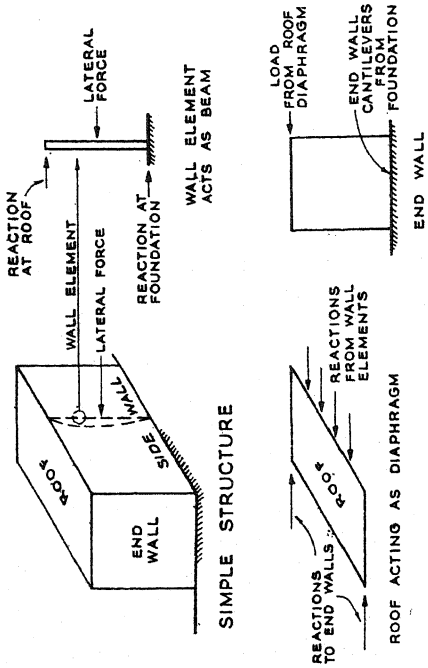

FIG. No. 8

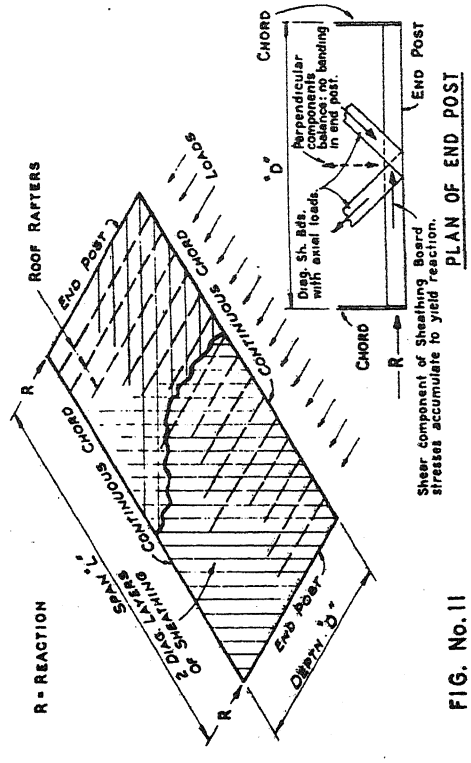

FIG. No. 11

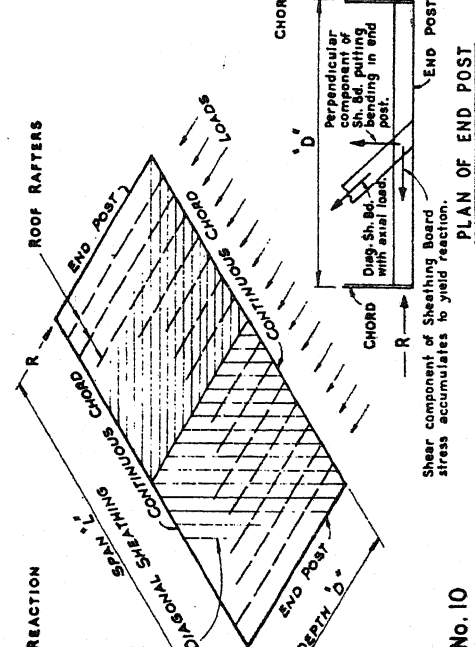

FIG. No. 10

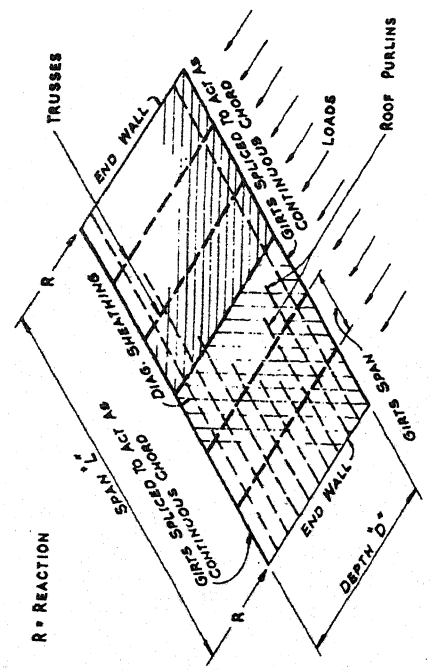

FIG. No. 12

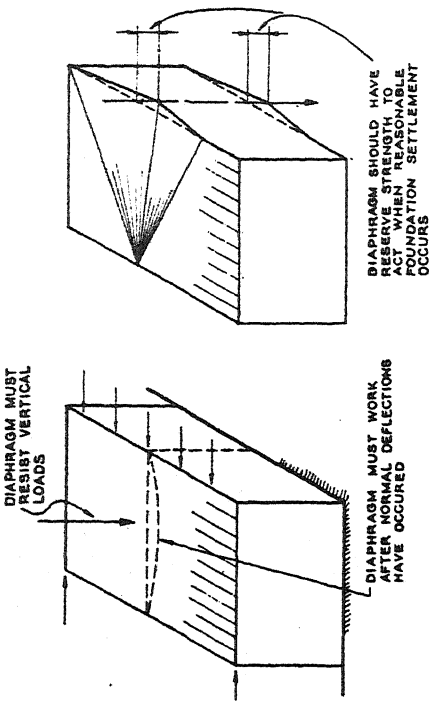

FIG. No. 13 A

FIG. No. 13 B

FIG. No. 14

FIG. No. 15