BASIC APPROACH TO STRUCTURAL DESIGN FOR SEISMIC FORCES

by

S. B. Barnes*

History is full of stories of earthquake damage. Frequently this was considered to be Divine Retribution and, perhaps partially because of this, no scientific attempt to reduce damage from earthquakes has come to out attention until comparatively recent years.

I moved to California from Indiana in 1921. I experienced the earthquakes at or near Santa Barbara in 1925 and the so-called Long Beach Earthquake in 1933. Later we had the earthquakes in the Imperial Valley of 1939 and 1940 and the Tehachapi Quake in 1952. I saw severe damage in all of these. Engineers began to try to find a solution to protect structures subjected to earthquakes. The U.S. Coast and Geodetic Survey was directed to make studies in the field of seismology after the Santa Barbara Earthquake. They developed the strong motion seismographs that would give engineers the important earthquake characteristics such as ground acceleration, period, amplitude and direction. Some recordings were made in the Long Beach Earthquake of 1933.

In 1928 the California State Chamber of Commerce was sufficiently interested to propose a building code "dedicated to the safeguarding of buildings against earthquake disaster." No governmental agency, to my knowledge, ever adopted this early code but it made engineers think about seismic design. During this year I was in charge of designing two 13-story steel frame buildings in Los Angeles. Unbeknown to the Owner, I designed the frame to resist a 10% gravity horizontal force down to the second floor, and below this did the best I could. This mostly involved beam and column connections. I ignored the relative rigidities of the brick filler walls. Later it was discovered that I had spent some $10,000.00 of the Owner's money for this over and above minimum code requirements and I was properly "bawled out". Later, after the 1933 Earthquake, I was complimented.

After we had the somewhat meager strong motion seismograph records of the 1933 Earthquake, we began to learn a little bit about earthquake ground motion. We knew it was a random and dynamically complex motion. The so-called El Centro Earthquake was still better recorded. We began to know something about ground motion but still knew very little about building response to such motion. Seismic design criteria were incorporated in California codes. The California

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Riley Act and the Field Act were adopted in 1933. Los Angeles, in 1933, required a coefficient of 8% of dead load plus half live load. The Uniform Building Code followed this in 1935 with some variations for soil differences.

Studies of the records of the Long Beach and El Centro Quakes, along with research at Cal Tech and other universities, showed that the flexibility and periods of structures affected their response to a given ground motion. This was reflected in the City of Los Angeles Building Code in 1943, which roughly interpreted building period in terms of number of stories in height. In 1948, San Francisco adopted a building code which in format looked greatly different from that of Los Angeles and incorporated a recognition of building period in terms of height over width of building.

Actually, these two codes were not too much different in results but they looked different. People in other areas who looked to Los Angeles and San Francisco engineers for direction began to wonder why the differences existed. The Structural Engineers Association of California finally got the Los Angeles and San Francisco groups together in 1957 and the result was the "Recommended Lateral Force Requirements and Commentary" of the Structural Engineers Association of California. Except for the Commentary, this is the basis of the Uniform Building Code requirements of today. As Chairman of the Base Shear Committee, I was happy to get agreement after only one Martini around.

In any case, this "Recommen ded Lateral Force Requirements and Commentary" produced by the Structural Engineers Association of California, in my opinion, reflects the best thinking of engineers to date and has been adopted by the Uniform Building Code as a part of the building code. It is certainly not perfect. No code can properly describe, in a manner that is equitable to all materials and conditions, all the procedures necessary to produce reasonably earthquake resistant buildings. There is no real substitute for that frequently overworked term "sound engineering judgment".

There are bugs in this Uniform Building Code that are being discovered with use. There will be changes made. This is as it should be. I have been quoted as saying that I could design a building, technically in compliance with the Code, and guarantee to have it fall down in the next major earthquake. No one has paid me to do it yet.

Actually, we know very little about earthquakes and their effects upon structures. Our useful records go back only a few years. We know that the ground motion is random. We know that building response varies with distance from the epicenter and with the type of geological formations an earthquake wave shock passes through between the source of initial shock and a structure, but as yet we cannot accurately evaluate this effect in engineering terms.

We have recorded horizontal earth accelerations of, say, .3 gravity and even higher vertical accelerations. In the centuries before records there must have been greater earthquakes. We have long ground motion periods as found at Anchorage and short ground periods in other earthquakes. We know from experience and theory that the response of a structure is greatly affected by the relationship of the ground periods and the period or periods of a structure. Now we can compute the natural period of most structures when we consider the structural frames only. But the interaction of non-structural elements with structural elements is difficult to evaluate. The energy absorbed in cracking plaster or concrete and in elongating steel elements in the plastic range makes period computations very difficult.

In setting up building code coefficients some years ago, we were thinking of buildings with exterior walls of masonry or concrete. Today we have light, so-called window wall construction. We thought perhaps we might have a damping factor of 8% to 10% critical. This may now be reduced to 2% to 5% in some buildings.

The Code criteria for T or period is obviously highly empirical. We are now trying to measure existing building periods instrumentally. The results to date, in some cases, have been compared with both computed T values and the values obtained by the Code empirical formulas. Sometimes the empirical formula values come closer than the computed values. Also, since in most cases we cannot shake a building to destruction, our shaking impulses may be too small to properly evaluate the damping factor. We need more information on this.

The philosophy of the present Uniform Building Code is to prevent practically all damage in a minor earthquake, permit non-structural damage in a medium intensity earthquake and permit structural damage but not collapse in an earthquake of major intensity.

Because the El Centro Quake was well recorded and publicized most of us use this as a sort of standard. We know the ground acceleration reached .3g at El Centro but we design our short, stiff, masonry bearing wall buildings in Zone III for a base shear of only .133g. Where design and construction is carried out to the limit of allowable stresses as prescribed in the Code it is obvious that we could have severe damage to frangible or brittle elements and response in the plastic range where ductile elements are used, were we to have an earthquake of the intensity of that of El Centro. It is noted that an infinitely stiff building will have the same period as that of the supporting ground.

In this respect, earthquake design differs from wind force design. Wind forces prescribed in most codes are realistic in magnitude and when we design for these we expect that our structural elements will not fail and that these elements will not be stressed beyond the elastic limit. Earthquake Code design criteria anticipates response beyond the elastic limit in major intensity earthquakes.

For our taller buildings the Code has intended the same factor of safety or margin of safety as for the short, stiff buildings, but in some soil condition areas this may not be true. The long ground period at Anchorage, for instance, caused more damage to the taller buildings with longer natural periods than to the one and two-story masonry buildings of short natural periods. Nevertheless, we have found that well designed and well constructed buildings came through the El Centro, Tehachapi, and Alaskan earthquakes with minor damage. In using the term "well designed", I mean designed in accordance with the requirements and intent of the present Uniform Building Code or preceding codes of approximately the same requirements coupled with some good engineering judgment.

So much for the adequacy and philosophy of building codes. These codes must weigh cost and practical feasibilities with damage and even life. They are, however, minimum standards and there is nothing to prevent an engineer or architect from being more conservative than these minimum standards. If we were to design structures for the ultimate earthquake possible without damage, we would produce only hollow cubes or hollow spheres.

Seismic Resistant Elements:

The forces created by an earthquake are functions of mass and acceleration. Unlike wind, these forces originate at the mass centers. From the points of origin these forces must be distributed horizontally (in most buildings) to vertical elements which, in turn, transmit these forces to the ground. The horizontal distributing elements, usually called diaphragms, may be horizontal steel bracing, horizontal beams, or roof or floor systems which can be calculated by normal procedures or have demonstrated adequacy by tests. The vertical elements may be steel or concrete moment resisting frames or cantilevers or shear wall of concrete, masonry or wood.

Diaphragms of horizontal steel trussing, concrete floors or roof or horizontal beams are readily computable. Other diaphragm systems such as metal decks with or without concrete fills, plywood or diagonal sheathing, insulation boards, precast concrete elements, etc., are complex in action and frequently need to be tested to demonstrate ability to act as a homogeneous unit to resist lateral forces.

Relative Rigidities:

Where a structure has an infinitely rigid diaphragm at any level, it is obvious that when subjected to a horizontal force, the vertical elements connected to the diaphragm must deflect equally at that level (torsional effects excluded). This means that each vertical element must accept that portion of the total horizontal force at that level in proportion to its rigidity or stiffness as compared to the total rigidities of all the vertical elements involved.

This is not the case where we have a diaphragm that is infinitely flexible. In such a case the vertical elements must accept horizontal loads which are tributary to them and these elements do not have to deflect equally except where especially tied together. Actually, there is no such thing as either an infinitely rigid or an infinitely flexible diaphragm. In most cases a reasonably heavy and nearly square concrete diaphragm can be considered as approximately rigid but even this should be checked in some special areas.

In the new Tri-Service Seismic Design Manual which we have prepared for the U.S. Government we have attempted to categorize diaphragms of the most common materials into general classes of stiffness. In any case, except where diaphragms of concrete and some types of metal deck are used we have horizontal force distributing systems of some flexibility which may require consideration in determining forces to be assigned to vertical elements. The analysis then may be similar to that of an elastic beam, and yielding supports and the relative rigidities of vertical and horizontal elements must be compared. Sometimes such computations are long and difficult and a design can be made satisfying limits without sacrificing economy. See Figure 2-10, 2-11, 2-12 and Plates 2-2 and III-34 from Tri-Service Manual.

With the concept of relative rigidities we must appreciate the desirability of symmetry of resisting elements. Where the center of horizontal loads does not coincide with the center of resistance we have torsion or a twisting in a horizontal plane. Experience teaches us that excessive torsion produces severe damage. Theoretically we can compute these torsional effects similarly to the manner we compute stresses in rivet groups subjected to rotation but here again the ability of the diaphragms to transmit torsion must be weighed and the effects of damping and deformation of non-structural elements evaluated. Since non-structural elements may be here today and there tomorrow this imposes some degree of judgment. See Plate III-43, Tri-Service Manual.

Most of you have seen pictures of the J.C. Penney Building in Anchorage after the "Good Friday" Earthquake of 1964. Here was an example of severe torsional response. This sort of thing should be avoided. Because of the indeterminacy of both structural and non-structural torsional response, the Uniform Building Code requires an allowance for so-called accidental torsion. A classical example is a square building with crossed shear walls at its center. There is no computed torsion and yet we know that some ability to resist torsion must be provided. We had this classic problem in the design of the oval-shaped Satellites of the Los Angeles International Airport. We had crossed shear walls in the center of these structures. We used the perimeter structural steel frame to develop accidental torsional resistance. Actually, we doubled up on the code requirements on this at very little additional expense. See Plate III-81, Tri-Service Manual.

Fallacy of Ignoring Rigid Elements:

Some designers not too familiar with the actual response of structures in earthquakes have designed structures to resist earthquakes but ignored the effect of non-bearing but stiff and frangible filler walls. They have said to themselves these are just filler walls or partitions and we will ignore them in our computations. Unfortunately, no one has communicated with these walls and told them that they were to play a passive part in earthquake resistance. The U.S. Government Buildings at Elmendorf Air Force Base and Fort Richardson in Anchorage were full of 4-inch concrete block, non-reinforced walls which tried to act in diagonal compression or diagonal tension. Some of these exploded like shrapnel and had the earthquake occurred at some other time the loss of life would undoubtedly have been much greater. And, of course, by ignoring the rigidities of these walls the computations made were greatly in error, at least until these walls failed. Isolation of these walls is permissible, provided really effective isolation details are used. See Plate III-61 - Tri-Service Manual.

The ignoring of stiff masonry filler walls in seismic resistant calculations appears to be widespread. We recently had to review the design of a multi-story masonry walled building where these walls were not assumed to be acting in lateral force resistance. There is an apparent feeling that these filler walls may be allowed to crack up since the structural frame, if provided, will still support the structure. This is not necessarily true unless the structural frame is able to remain intact after the filler walls crack up.

Similarly, stiff vertical elements should not be by-passed when analyzing a diaphragm. If a rigid element is there it wil try to act. We found examples of such by-passing of rigid vertical elements in the design of some of the damaged buildings in the Anchorage area. The designer must properly visualize building response and carry his analysis through to completion. See Plate T - (not from Manual).

Effect of Diaphragm on Building Periods:

It is customary to compute the period of a building by analysis of the vertical resisting elements. However, there are times when the horizontal elements must be considered. Take, for example, a long suspension bridge. The vehicular deck acts as a diaphragm between piers or anchorage towers. This deck may have a very long period as compared to the stiffer towers or piers. The result is that the tail may wag the dog or involve severe harmonics. This sort of thing is not covered in seismic codes as yet but should be considered.

Interaction of Building Irregularities:

Buildings with wings shaped like L's, T's, or U's present special problems. We usually design a building to resist seismic forces about its two major axes. But the ground movement is rarely directioned this way. So we have components of forces in both directions and these components may be variable and out of phase. Thus we can expect trouble at intersection of L's or U's or T's. Separations here are usually advisable unless extra conservative precautions are taken. Here again we must fall back on some judgment. See Plate T.

Narrow Towers on Broad Bases:

This type of structure is enountered frequently in Southern California where parking areas are desirable and required by law. The tower usually has a moment resisting frame with no shear walls. The large base structure usually involves basement stories which obviously must have basement walls which have almost infinite rigidities as compared to the more flexible frame which carries through under the tower.

At the transition level then we need an especially heavy diaphragm to transfer lateral forces from the tower area to these perimeter basement walls and special attention must be given to strut-tie connections at this level.

Tall Cantilever Piers Connected with Short, Stiff Beams:

This problem arises all too frequently. Try to visualize an end elevation of a building, say, 160 feet high with windows at each floor level in the center of the end wall so that we have two tall piers connected by short spandrel beams of depth from head to sill of windows.

Under lateral loads parallel to this end wall, the two piers may deflect similarly to a single pier with holes in it or similarly to two independent piers or somewhere in between depending on the relative rigidities of the piers and the spandrel connecting beams. The shear induced in the spandrel beams to force the piers into unilateral deflections can become extremely large. On the other hand, if the connecting beams are designed as hinged tie-struts there may be severe torsional warping of the floors and the overturning action and stresses in the individual piers will be entirely different. This can, of course, be calculated but the solution is not simple. Two examples of this type of construction when subjected to lateral forces are the L Street Apartments and the Mt. McKinley Apartments in Anchorage.

The Floating Building Concept:

For some strange reason there appears to be a recent architectural trend to make a tall building look as though it had no support below the second floor. It is supposed to create the illusion of a building floating on thin air. The entire structure may be supported on two or three or four columns or clusters of columns. Such a building involves a reconsideration of basic seismic philosophy. Since this philosophy, for the typical building, anticipates responses in the plastic range to a major earthquake we must consider whether, in cases such as this, we can permit plastic hinges to form without collapse. A special committee of the Structural Engineers Association of Southern California has been set up in the Los Angeles area to advise building code authorities on unusual structures, and several structures of this general nature have been reviewed by this committee. This committee is advisory only, but in cases where the problem has arisen the committee has ruled in essence that where structural elements would cause collapse by large deformations in the plastic range, the design of these elements should be such that earth movements of the intensity of the El Centro Quake be resisted without exceeding yield point stresses. Here the higher strenth steels have been permitted.

Details:

Analysis of damage after a severe earthquake almost always points up the necessity for greater attention to details. In structural steel the problems involved in seismic design are generally the same as those involved in vertical load design except to see that horizontal stresses and forces are followed through a complete path. There is one basic exception. Where a ductile steel frame is required, the connections selected whould provide for such ductility. For instance, in a tall building with a moment resisting frame where girders are increased in size to control drift, the question may be raised as to the column and girder connection. If this connection is designed for stress only, does it provide adequate ductility? Or should such a connection be designed to develop the member? Test research is now under way on this problem. The probability of repeated reversals in stress is involved in this problem also.

Concrete has additional problems. The Portland Cement Association book, "Design of Multistory Reinforced Concrete Buildings for Earthquake Motions" by Blume, Newmark and Corning, shows that reinforced concrete can be made more ductile by techniques of confinement and patterns of steel reinforcement which will produce yielding in the steel prior to failure of the concrete. This is still being studied by Code authorities, particularly as to detail. Recent column-beam knee tests made in the laboratories in Chicago indicate some unusual problems at the core of such intersections where stresses beyond the yield are involved. It is probable that recommendations resulting from these tests will include a requirement for carrying column ties through such joint cores instead of stopping ties at top and bottom of the horizontal members.

It is sometimes difficult or impossible to anchor reinforcing bars in areas of compression in seismic design. Special consideration should be given to such cases. Welded connections may be preferable to laps.

Special attention should be given to column ties where anchor bolts are set or haunches are provided for anchor bolts. I saw a number of cases in the Anchorage area where failure was caused by pull out of anchor bolts where ties around the bolts just opened up.

Precast elements provide special problems. Where horizontal precast elements are to function as a diaphragm they must be tied or anchored together so as to act as a homogeneous system. Although codes do not prescribe consideration of vertical acceleration resistance, anchorage details between adjacent precast elements should provide for this possibility. This is related to differential vertical deflections between precast elements.

Clean construction joints are a must in reinforced concrete construction. Otherwise, shear values should be limited to those provided by the rebars acting as dowels. There must be good bond between old and freshly poured concrete.

Reinforced concrete block masonry has the same problems in detail as reinforced concrete. I do not concur in the concentration of horizontal steel at top and bottom of concrete block masonry now permitted by the Uniform Building Code just because it is more economical. In my opinion, some maximum spacing such as 6 feet should be required.

Old joints are particularly important. The shear value through a concrete block wall is no better than the joint between wall and concrete footing or beam.

Reinforced grouted brick has joint problems similar to concrete block except that there is no economic problem in providing horizontal steel.

In all kinds of concrete or masonry work it is well to remember that the shear value in a member subjected to tension also is not as good as where the member is subjected to compression. This can occur when vertical piers or columns have large overturning stresses.

In any case, the importance of following through in the field in seeing that details are actually carried out properly cannot be overemphasized.

Some Possible Code Changes:

The Seismology Committee of the Structural Engineers Association of California is considering some of the problems which have arisen in the light of possible code changes. Some of the items under discussion are:

(1) Adequate ductility requirements not only for reinforced concrete but also for high strength steels.

(2) Prestressed or post-tensioned concrete insofar as these elements are portions of moment resisting frames subject to reversals.

(3) Revision of the .80 K factor and 1.00 factor. This involves some clarification. The original concept here was to recognize that where a complete vertical load carrying frame was provided, or where a frame was provided with some moment resistant properties, a shear wall could sustain considerable damage before the building collapsed. However, since it is possible under the Code to take a 10-inch concrete wall and place 4 vertical bars with ties in this wall at intervals and place a couple of bars at floor line and, say 3 feet down from floor line and technically have a beam and column system, it is obvious that this does not provide a frame in the sense intended. If such a wall cracks up as a shear wall the cracks will run right through the so-called frame and we have provided no real benefit to the structure. Requiring the frame enclosing a shear wall to be able to take all the shear in the shear wall is one possibility. Elimination of these K factors is a possibility. The problem is not as simple as it sounds. The example given was for concrete but the principle is applicable also to reinforced masonry.

(4) Special requirements for connections between precast concrete elements are under discussion.

(5) Some computer work has been done that indicates that the overturning criteria set up in the Code needs re-evaluation. This is being studied.

(6) The effect of foundation soils conditions is being studied. What is the effect on building reponse of deep alluvial soils, bedrock foundations, and the geological formations between epicenter of earthquake and building location? Should we try to zone specific areas for these factors? This is being studied. We note the liquidization of sandy soils in Japanese earthquakes and the instability of the Bootleggers' Clay at Anchorage when subjected to earthquake vibration. We know that soils and geological formations are important. At some future time I anticipate that consideration of these factors will be incorporated in our design criteria.

In an era where new materials and new techniques are being developed it is necessary for any code to keep up with such developments. For this reason we must expect code changes. Our experience with seismic design codes is limited. It is hard to simulate earthquakes on large scale models and real earthquakes of large magnitude fortunately occur infrequently to test out theory. Even so, I feel that it will be impossible to write any code that will fairly, equitably, and economically insure that all our structures will not sustain severe damage in any earthquake. The range of imagination of architects and engineers is almost unlimited. Someone will always come up with something new that has not been covered. I do believe, however, that codes can form a fine background and basis for design, but such design should be tempered by a visualization of the response of the structure being designed and judgment used in seeing that all critical conditions are properly taken care of.

The original design concepts are frequently made by architects and sold to client-owners on the basis of functional space planning and esthetics before a structural engineer is called in. In seismic areas, it is my opinion that the original schematics should consider stability as well. An engineer should not be called in after schematics have been approved and told that "a good engineer can find a way to make it figure". Stability should be built into the original form and, in my opinion, this should not be forgotten by whomever has the prime design contract, whether he be archtect or engineer. I further believe that in a seismic area a look of stability is not incompatible with good archtectural and esthetic design.

TABLE 2-2

FLEXIBILITY CATEGORY	F	MAX. SPAN IN FEET FOR MASONRY OR CONCRETE WALLS	SPAN - DEPTH LIMITATIONS *			
			ROTATION NOT CONSIDERED IN DIAPHRAGM		ROTATION CONSIDERED IN DIAPHRAGM	
			MASONRY OR CONCRETE WALLS	FLEXIBLE WALLS ***	MASONRY OR CONCRETE WALLS	FLEXIBLE WALLS ***
VERY FLEX.	MORE THAN 150	NOT USED	NOT USED	2 : 1	NOT USED	1½ : 1
FLEXIBLE	70-150	200	2:1 OR AS REQ'D FOR DEFLECTION**	3 : 1	NOT USED	2 : 1
SEMI-FLEX.	10 - 70	400	2½:1 OR "	4 : 1	AS REQ'D FOR DEFLECTION**	2½ : 1
SEMI-FLEX.	1 - 10	NO LIMITATION	3:1 OR "	5 : 1	AS REQ'D FOR DEFLECTION**	3 : 1
RIGID	LESS THAN 1	NO LIMITATION	AS REQ'D FOR DEFLECTION**	NO LIMITATION	AS REQ'D FOR DEFLECTION**	3½ : 1

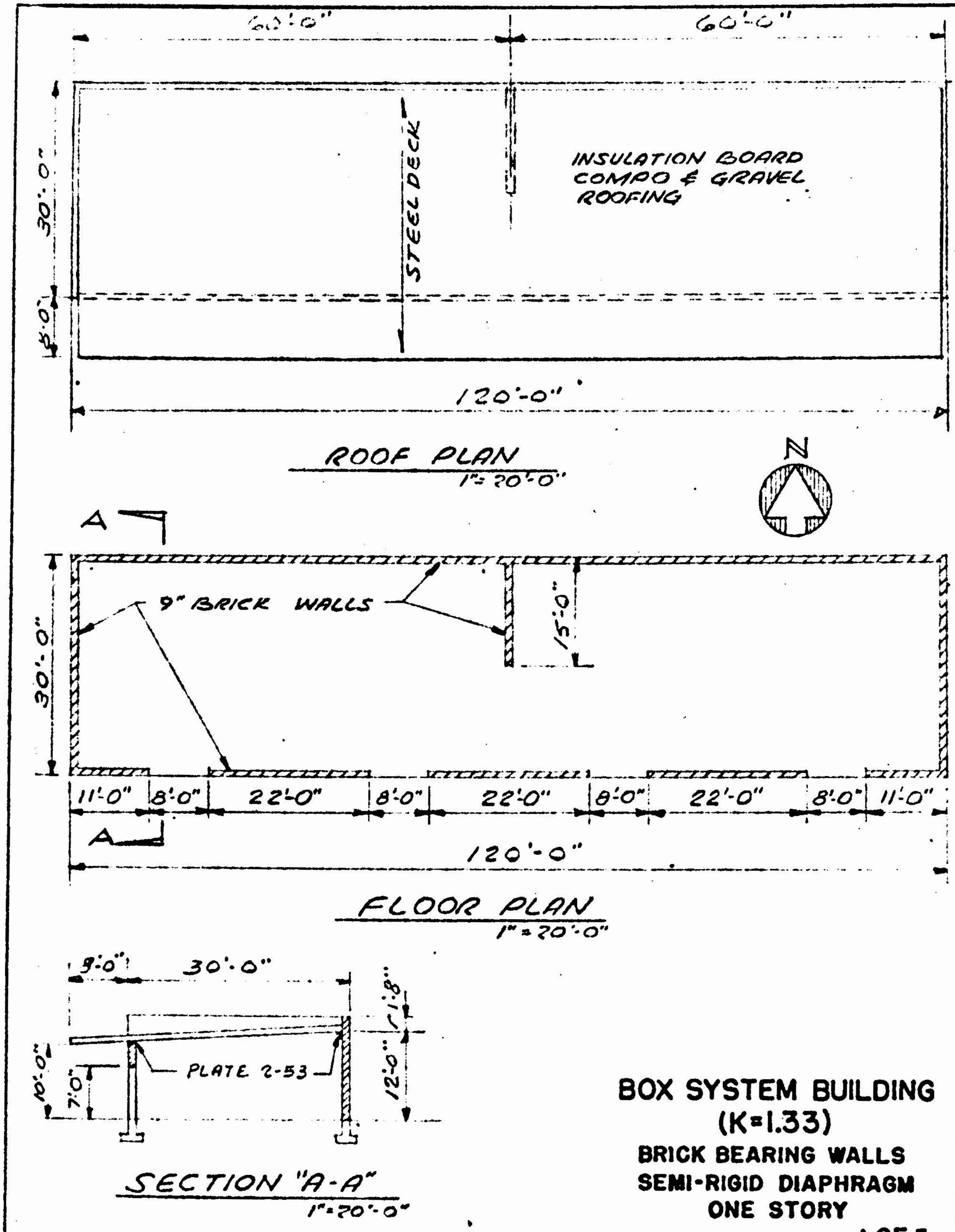
* SOME DIAPHRAGM SYSTEMS AS NOTED LATER ARE ASSIGNED LIMITATIONS IN A MORE STRINGENT CATEGORY.

** SEE PLATE 2-1 FOR DEFLECTION LIMITATIONS OF MASONRY AND CONCRETE WALLS

*** WHEN APPLYING THESE LIMITATIONS TO CANTILEVER DIAPHRAGMS, THE SPAN DEPTH RATIO WILL BE ½ THAT SHOWN.

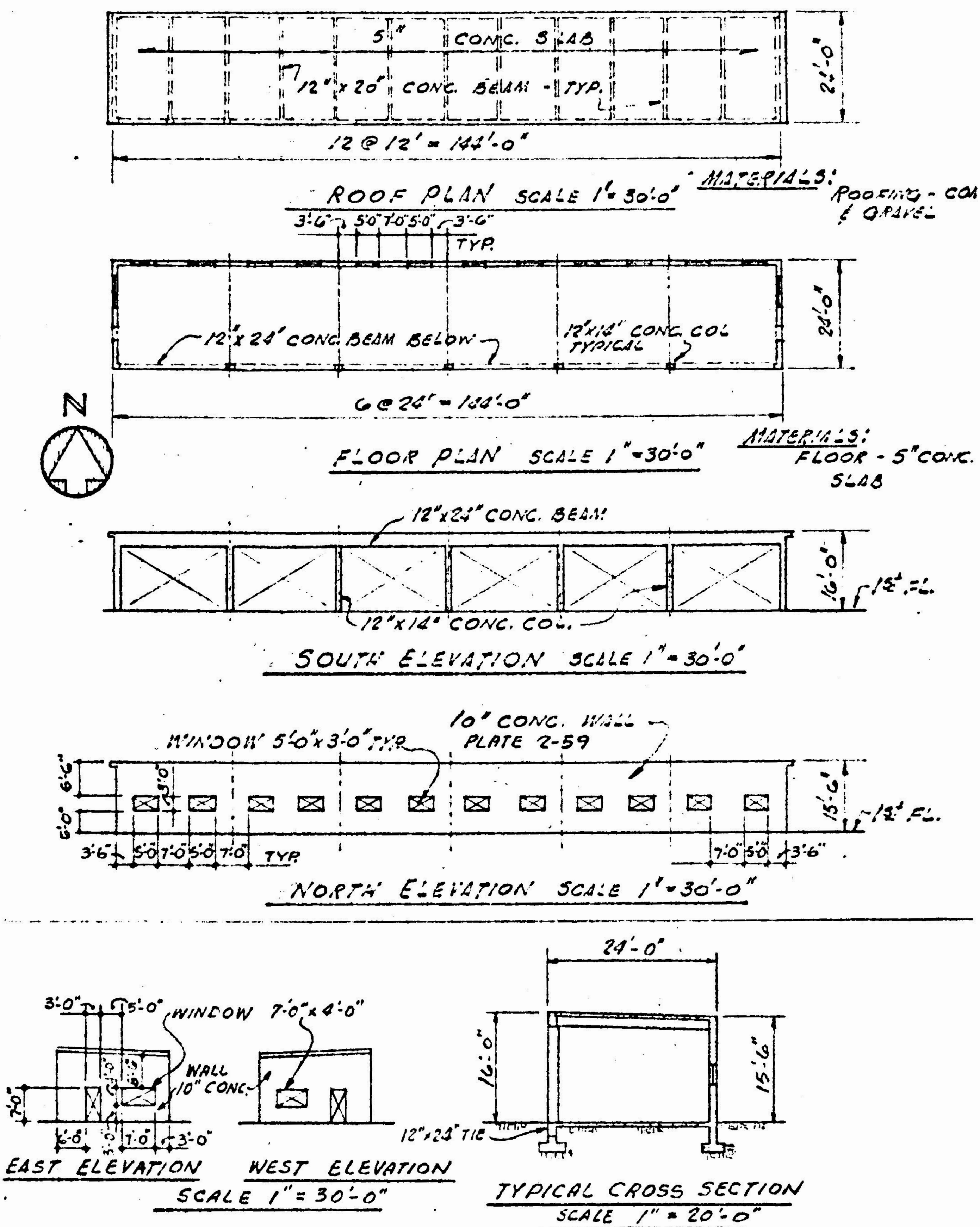
DIAPHRAGM
FLEXIBILITY LIMITATION

Building with Brick Bearing Walls and Semi-Rigid Steel Deck Diaphragm, One Storey: Two examples are given of a shear wall building with a semi-rigid diaphragm. Both are one-storey buildings with reinforced grouted brick masonry walls, and a long span steel deck having elevated plane of shear transfer (PLATE III-34, Page III-36). The steel deck has welded seams and is classified as semi-rigid. The first example, PLATES III-35 through III37 on Pages III-39, shows a building 30' x 120' with a partial cross wall in the center. Computations are made to determine the relative stiffness of diaphragm to center shear wall. In this case we find the load to the center wall approximates its tributary load. This implies that the shear walls are very rigid in comparison with the diaphragm. The second example, PLATE III-38 on Page III-40, illustrates the same building as in the first example except the shear walls have 100 times the flexibility. As before, computations are made to determine the relative stiffness of diaphragm to center shear wall. In this case the center wall would receive considerably less than tributary horizontal load and the distribution would be closer to the assumption of a rigid diaphragm. Where multiple interior cross walls are used, computations are similar to those of a foundation on yielding supports and much more complex than in the problems illustrated. It is frequently satisfactory to design the cross walls for the extreme limits of either case. With a rigid dia!hragm the walls receive loads in proportion to relative rigidities of the walls. With a flexible diaphragm the walls receive tributary loads. Where the difference in construction cost is neglible, such conservative assumptions are warranted. If not, the more rigorous analysis will be made.



BOX SYSTEM BUILDING
(K=1.33)
BRICK BEARING WALLS
SEMI-RIGID DIAPHRAGM
ONE STORY
1 OF 5

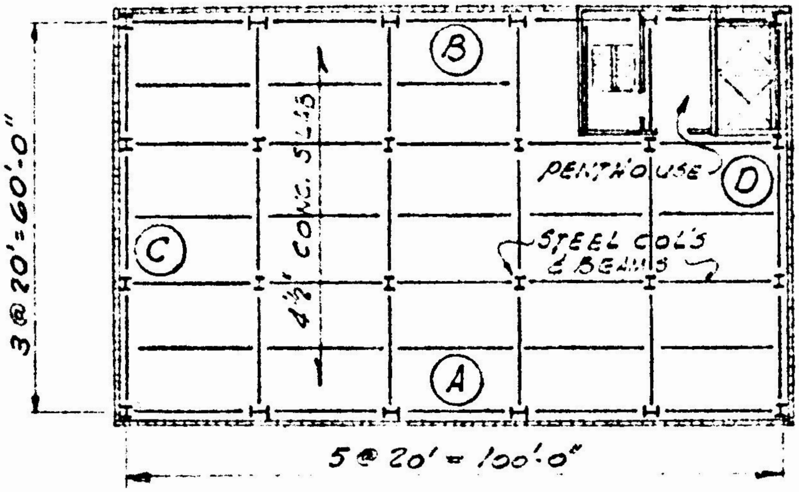
PLATE III-34



BOX SYSTEM BUILDING
(K=1.33)
CONCRETE SHEAR WALLS
CONCRETE FRAME ONE SIDE

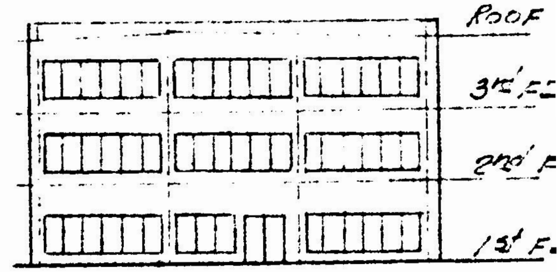
CONCRETE DIAPHRAGM
ONE STORY
1 OF 6

PLATE III-43

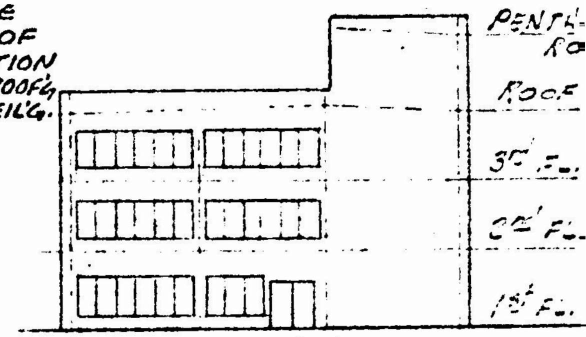


ROOF PLAN SCALE 1" = 30'

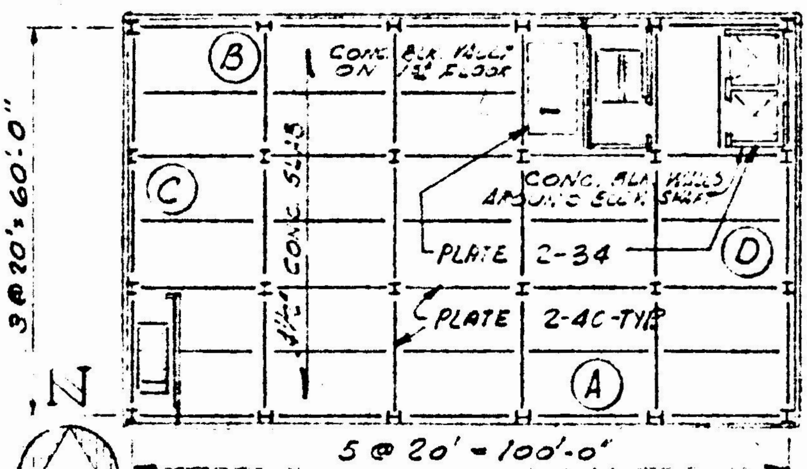
MATERIALS
CONCRETE SLAB ROOF
COMPOSITION & GRAVEL ROOF'G
PLASTER CEIL'G.



WALL "C"

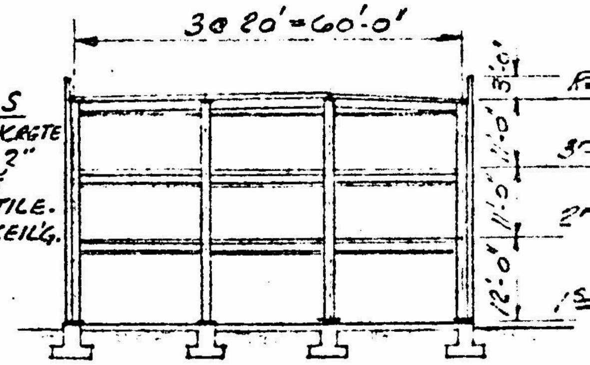


WALL "D"

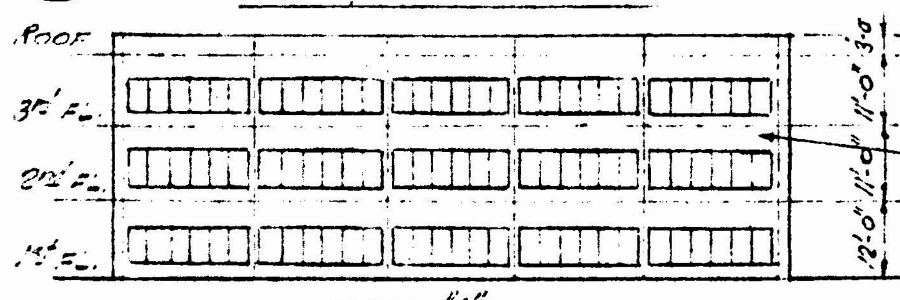


2nd & 3rd FLOOR PLAN

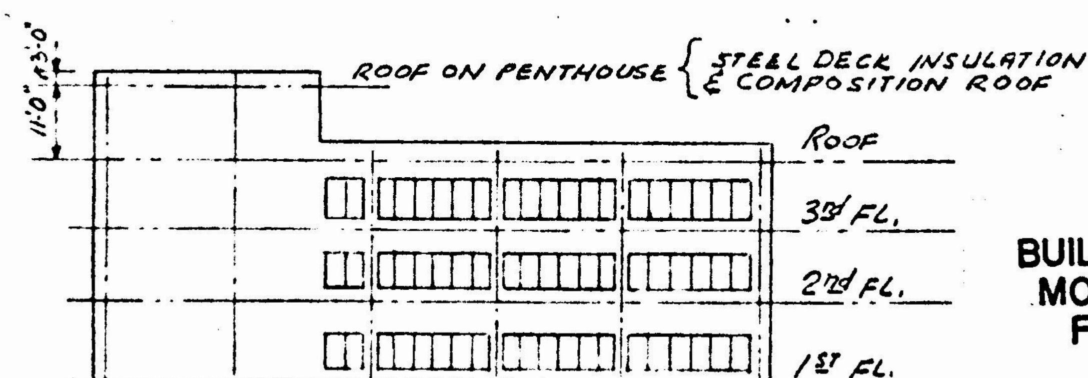
MATERIALS
FLOOR-CONCRETE SLAB WITH 2" TOPPING & ASPHALT TILE.
PLASTER CEIL'G.



TYPICAL CROSS SECTION
SCALE 1" = 30'



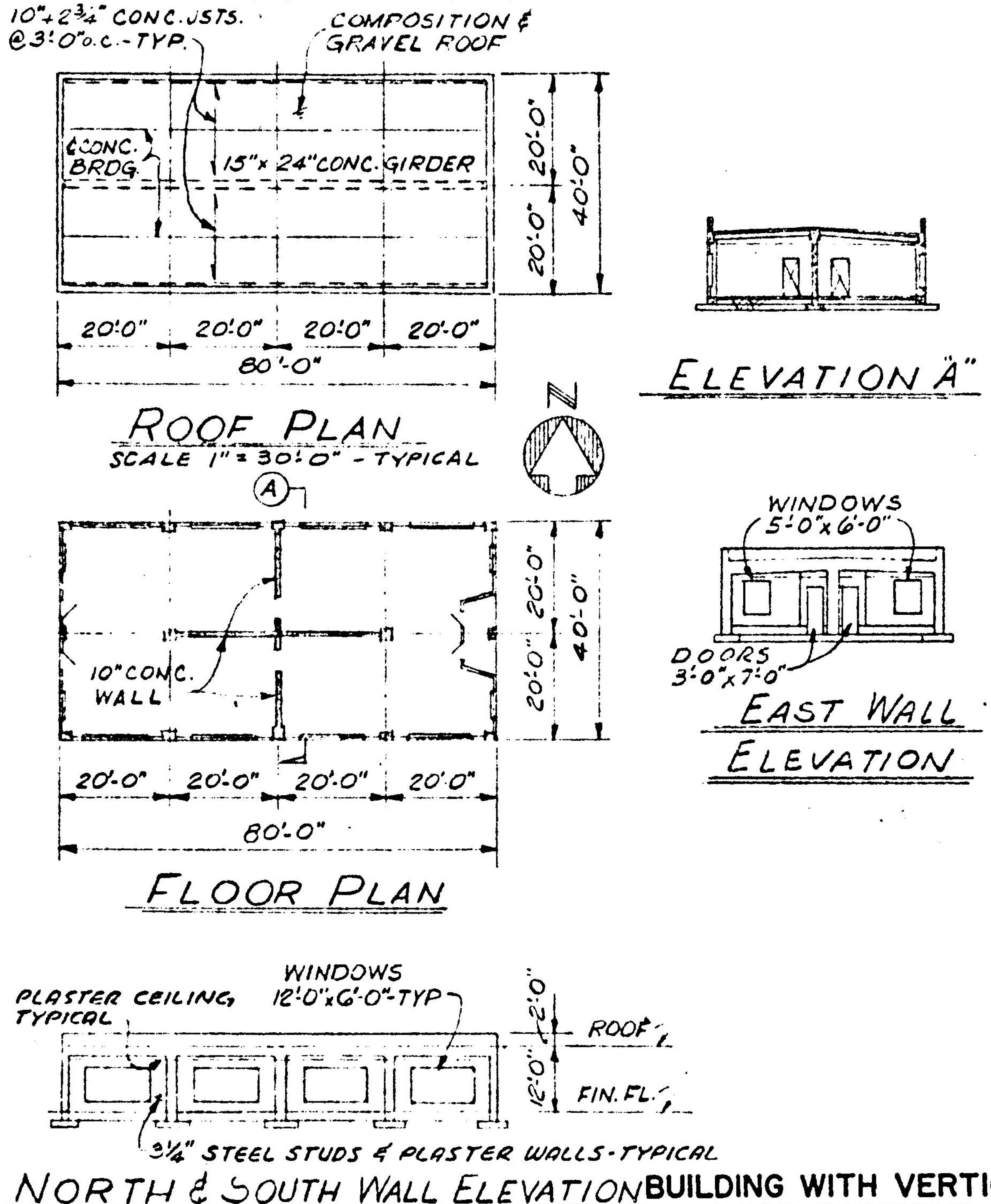
WALL "A"



WALL "B"

BUILDING WITH 100 %
MOMENT RESISTING
FRAME (K=0.67)
STEEL FRAME
ISOLATED MASONRY WALLS
THREE STORY

1 OF 4



BUILDING WITH VERTICAL LOAD CARRYING SPACE FRAME (K=1.00)

CONCRETE SHEAR WALLS WITH NO RESISTANCE TO ROTATION ONE STORY

1 OF 3

PLATE III-80

LOADS FOR ROOF

ROOF D.L.

ROOFING	=	6.0
JOISTS	=	56.0
GIRDER	=	9.5
COLUMNS	=	4.0
PLASTER CEILING	=	10.0
		85.5#/□

WALLS = 3.17 × 125 = 398
4.83 × 16 = 77
475#/.

N-S

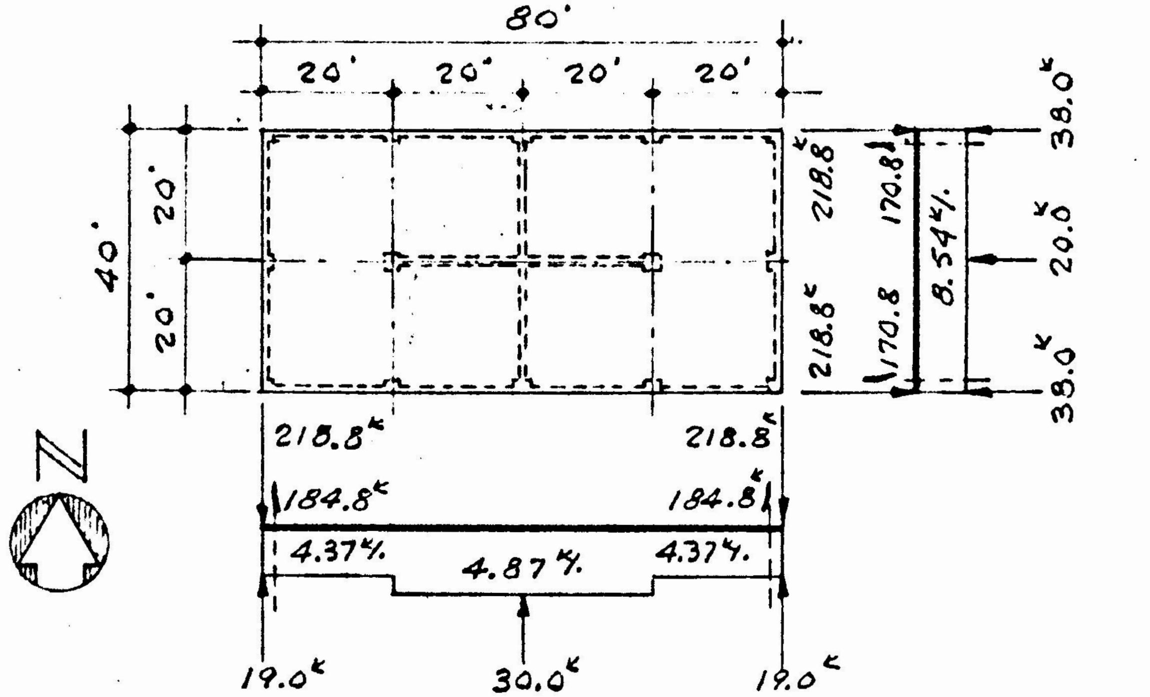
ROOF = 85.5 × 40 = 3420
WALLS = 475 × 2 = 950
4370
+WALL = 4.0 × 125 500
4870#/.

WALLS = 475 × 40 = 19,000#
6 × 125 × 40 = 30,000#

E-W

ROOF = 8.5 × 80 = 6840
WALLS = 475 × 2 = 950
7790#/.
+WALL = 6 × 125 = 750
8540#/.

WALLS = 475 × 80 = 38,000#
500 × 40 = 20,000#.



$W = 437.6^k$

$V = ZKCW = 1 \times 1 \times 0.1 \times 437.6$
$= 43.76^k$

BUILDING WITH VERTICAL LOAD CARRYING SPACE FRAME (K=1.00)

CONCRETE SHEAR WALLS WITH NO RESISTANCE TO ROTATION ONE STORY

2 OF 3

PLATE III-81

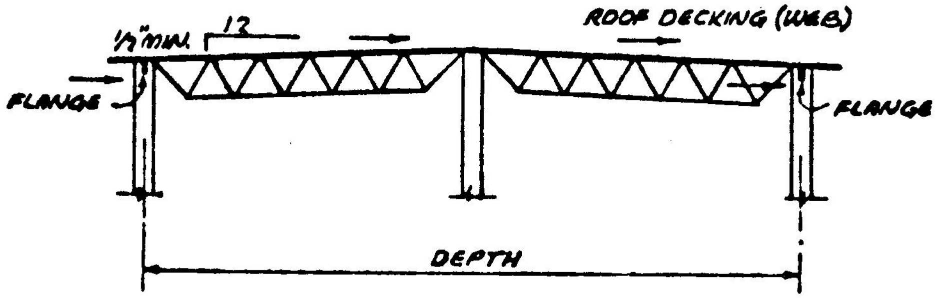


Fig. 2-5: ROOF DIAPHRAGM

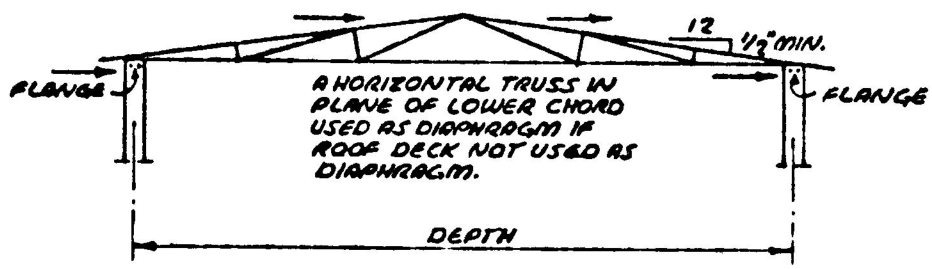
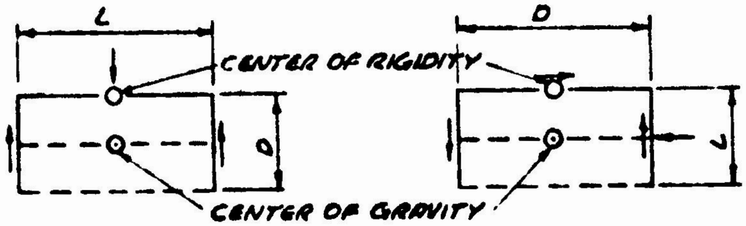
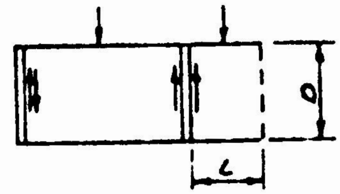


Fig. 2-6: TRUSS DIAPHRAGM

(1) Distribution of Forces Due to Rotation (Torsion): The magnitude of the torsional moment that is required to be distributed to the vertical resisting elements by a diaphragm is determined by the sum of the moment created by the physical eccentricity of the translational forces at the level of the diaphragm from the center of rigidity of the resisting elements and the arbitrary Code eccentricity of 5% of the maximum building dimension. The effect of torsion distribution on the diaphragm is noted as distinct from the somewhat similar action of a cantilevered diaphragm. The torsional distribution by the more rigid diaphragms to the resisting elements will be assumed to be in proportion to the stiffness of the elements and its distance from the center of rigidity. The more flexible diaphragms will not be used in torsional distribution. Cantilever diaphragms on the other hand will distribute translational forces to certical resisting elements even if the diaphragm is flexible. In this case, the diaphragm and its chord act as a flexural beam on supports (vertical resisting elements) whose resistance is in the same direction as the forces.





Cantilever Span

Fig. 2-7: NO ROTATION ON DIAPHRAGM

Fig. 2-8: ROTATION ON DIAPHRAGM

Fig. 2-9: CANTILEVER DIAPHRAGM

(2) Diaphragm Groups as Related to Stiffness: Diaphragms are classified into five groups, depending on their relative flexibilities. These are rigid, semi-rigid, semi flexible, flexible, and very flexible diaphragms. No diaphragm is actually infinitely rigid and no diaphragm capable of carrying load is infinitely flexible. All materials deflect under load.

(a) A rigid diaphragm is assumed to distribute horizontal forces to the vertical resisting elements in proportion to their relative rigidities. It is also assumed to distribute torsional forces to these vertical resisting elements in direct proportion to the stiffness of the elements and their distances from the center of rotation.

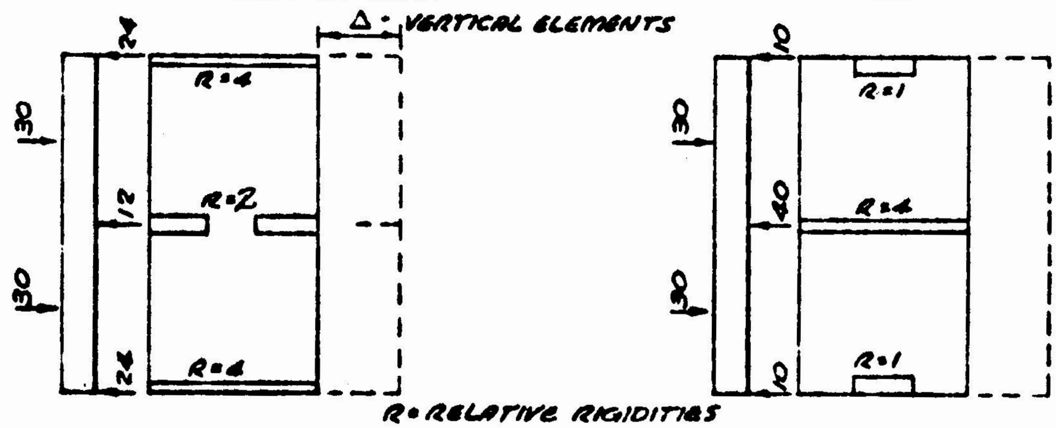


Fig. 2-10: RIGID DIAPHRAGM

(b) Semi-rigid and semi-flexible diaphragms are those which have significant deflection under load but which also have sufficient stiffness to distribute a portion of their load to vertical elements in proportion to the rigidities of the vertical resisting elements. The action is analogous to a continuous concrete beam system of appreciable stiffness on yielding supports. The support reactions are dependent on the relative stiffnesses of both diaphragm and vertical resisting elements. A rigorous analysis is sometimes very time-consuming and frequently unjustified by the results. In such cases a design based on reasonable limits may be used.

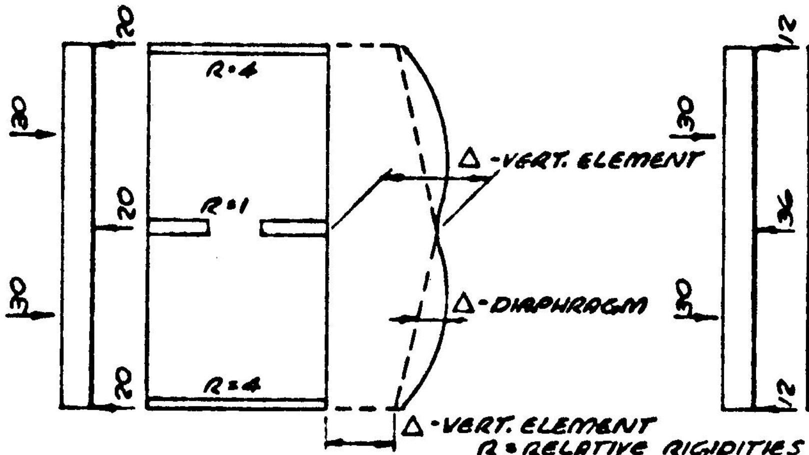


Fig. 2-11: SEMI-RIGID DIAPHRAGM

(c) A flexible diaphragm is analogous to a shear deflecting continuous beam or series of beams spanning between supports. The suppos ts are considered non-yielding, as the relative stiffness of the vertical resisting elements compared to that of the diaphragm is great. Thus a flexible diaphragm will be considered to distribute the lateral forces to the vertical resisting elements on a tributary load basis. A flexible diaphragm will not be considered capable of distributing torsional stresses resulting from concrete or masonry masses.

(d) A very flexible diaphragm is one which will exhibit comparatively large deflections under prescribed static loadings. Its use will be restricted to those cases where a large floor to floor deflection would be of no concern.

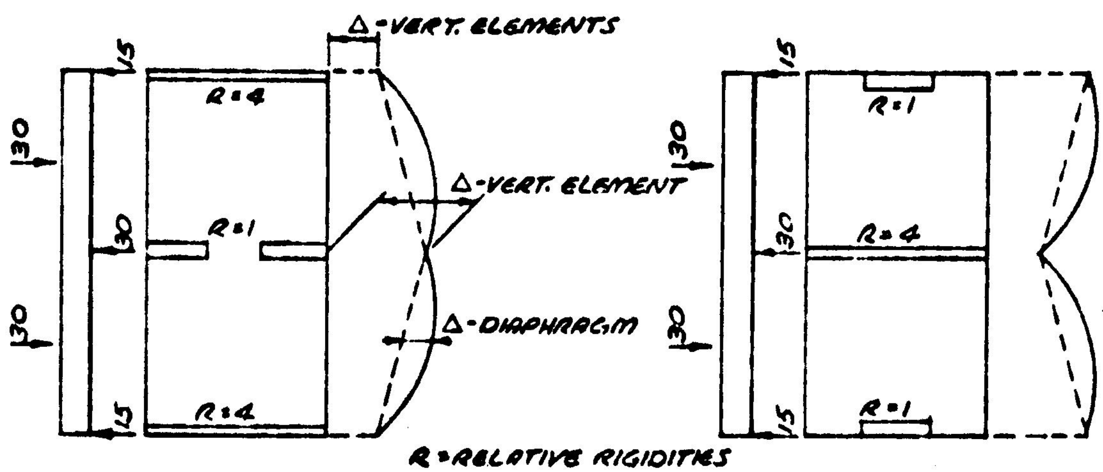


Fig. 2-12: FLEXIBLE DIAPHRAGM

(3) Diaphragm Deflections: A diaphragm will be designed to provide such stiffness and strength so that walls and other vertical elements laterally supported by the diaphragm can safely sustain the stresses induced by the response to seismic motion. The deflection of the diaphragm under the prescribed static forces will be used as the criteria for the adequacy of the stiffness of a diaphragm.

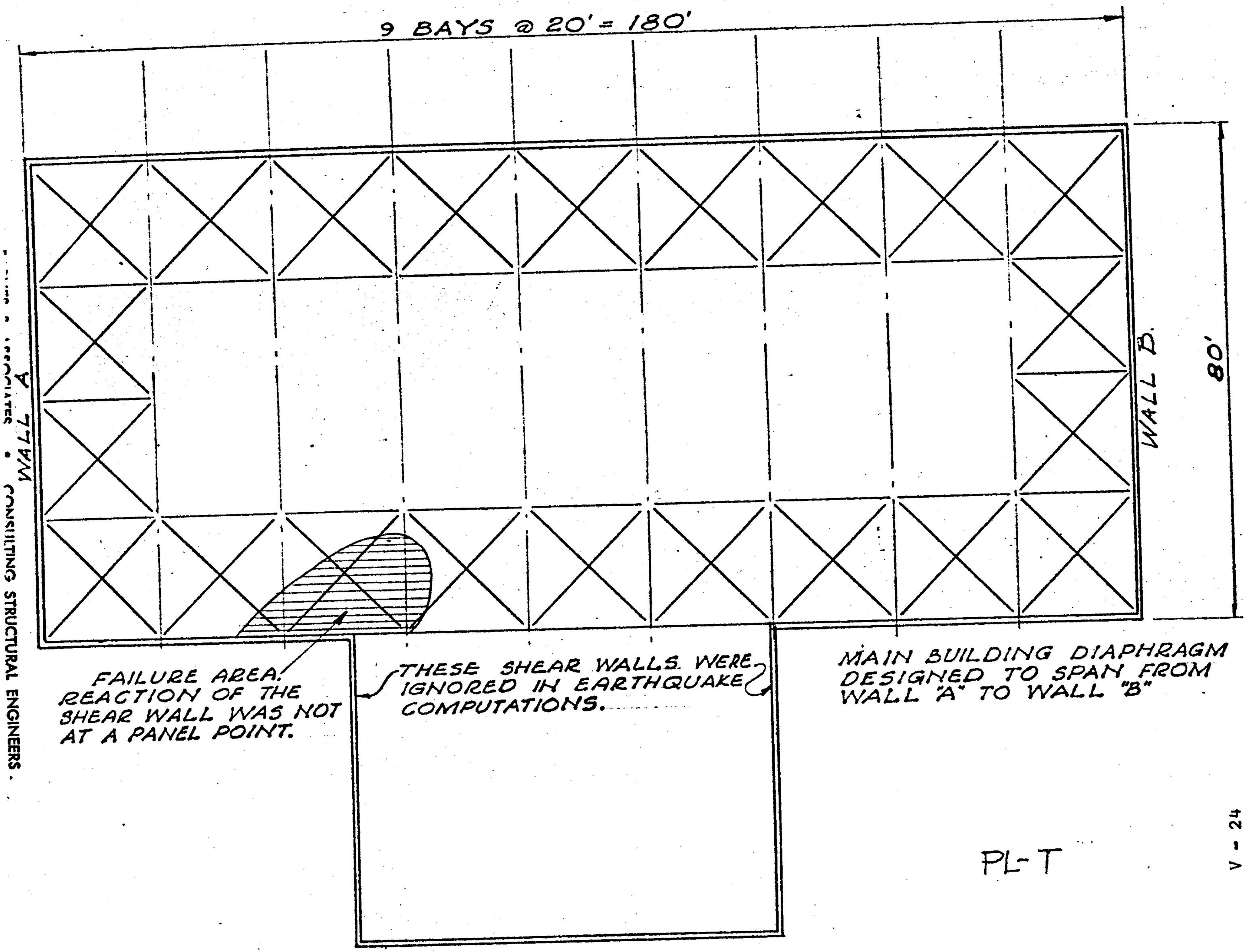
(a) The deflections of some diaphragms can be computed with reasonable accuracy. Other diaphragms have characteristic and fabrication variables making an accurate solution of deflection characteristics meaningless. Thus the methods of dtermination of the deflection characteristics for diaphragms of all materials given herein will be used to keep the range of diaphragm deflections within safe limits. This limit is the allowable amount prescribed and the floor below when the supported walls are of concrete or masonry construction. See Equation (2-1) on Page 2-3).

(b) The limitation imposed on diaphragms supporting flexible walls is a maximum span to depth ratio. If tomsional distribution is required a more restrictive limitation will be imposed. See PLATE 2-2 pn Page for these limitations. The span-depth limitations do not directly reflect deflections. But, if the diaphragm is designed with the proper ratio, the deflection requirements will be considered to be met unless an unusual building layout is used wherein deflection criteria would become critical.

(c) The total computed deflection of diaphragms under the prescribed static seismic forces (Δ_d) consists of the sum of two components. The first is the flexural deflection (Δ_f) of the diaphragm which is determined in the same manner as the deflection of beams. The assumption that flexural stresses on the diaphragm web are neglected will be used except for reinforced concrete slabs. For such slabs the proportional flexural stresses also may be assumed to be carried by the web. The second action is the web deflection (Δ_w) of the diaphragm. The specific nature of the web deflection will vary depending on the type of diaphragm used as will be described hereinafter.

(d) The determination and limitations of the deflections of a diaphragm is a dsign function. However, to specify a limitation on the total deflection to a fabricator of the diaphragm web imposes a difficult condition to meet as he may not have access to all the design criteria nor the flange (chorc) conditions of the diaphragm. In order to provide a means of properly classifying and identifying the stiffness of a diaphragm web, the factor "F" will be determined. The factor F is equal to the average deflection in micro inches of the diaphragm web per foot of span stressed with a shear of one pound per foot. Expressed as a formula this becomes

$$F = \frac{\Delta_w \times 10^6}{q_{ave} L_1}$$


9 BAYS @ 20' = 180'
80'
WALL A
WALL B
FAILURE AREA.
REACTION OF THE
SHEAR WALL WAS NOT
AT A PANEL POINT.
THESE SHEAR WALLS WERE
IGNORED IN EARTHQUAKE
COMPUTATIONS.
MAIN BUILDING DIAPHRAGM
DESIGNED TO SPAN FROM
WALL "A" TO WALL "B"

PL-T

Where

L_1 = Distance in feet between vertical resisting element (such as shear wall) and the point to which the deflection is to be determined.

q_{ave} = Average shear in diaphragm in pounds per foot over length L_1.

Δ_w = Web component of Δ_d.

Conversely, the web deflection will be determined by the equation

$$\Delta_w = \frac{q_{ave} L_1 F}{10^6}$$

(2-2a)

Using the factor F, the flexibility categories of diaphragm webs have designated values as prescribed in PLATE 2-2 on Page V - 13.