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Structural forms in steel for earthquake resistance

La rèsistance de structures en acier aux tremblements de terre

Erdbebensichere Stahlbauten

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SUMMARY

The performance of steel structures in strong earthquakes is highly dependent on form — both exterior and interior form. The exterior form includes the basic concept of the building, shape, symmetry, continuity or lack thereof, and such architectural features as flexible first story. Internal form includes redundancy, choice of details, and moment frame as compared to the use of braced frames and shear walls.

RESUME

La résistance de structures en acier aux tremblements de terre dépend essentiellement de la forme extérieure et intérieure de la structure. La forme extérieure comprend le concept de base du bâtiment, sa forme, sa symétrie, sa continuité ou discontinuité, et aussi des aspects architecturaux tel qu'un premier étage souple. La forme intérieure comprend la redondance, le choix des détails constructifs et le système de cadres rigides ou contreventés ou parois reprenant les efforts tranchants.

ZUSAMMENFASSUNG

Das Verhalten von Stahlbauten während eines starken Erdbebens hängt grösstenteils von der äusseren und inneren Form des Tragwerks ab. Die äussere Form beinhaltet das grundlegende Konzept des Gebäudes, die Ausbildung, die Symmetrie, die Kontinuität oder das Fehlen derselben und architektonische Eigenschaften wie Flexibilität des ersten Stockwerks. Die innere Form umfasst statische Unbestimmtheiten, die Wahl von Details, Rahmentragwerke mit und ohne Aussteifung sowie Schubwände.

Traditionally, the form of a building consists of a roof and floors supported by exterior walls and possibly using interior columns as additional supports. In earlier days, walls were of masonry. This type of construction was limited in height and/or the number of floors due to the large thickness of the walls which supported the floors around the perimeter of the building. This form of construction is still the most prevalent in low and moderate height buildings although in many countries, concrete or reinforced concrete has replaced the masonry. Often the building is rectangular in plan but many shapes are used, as for example, in churches.

Cast and wrought iron structures (both bridges and buildings) with masonry walls or piers were developed in the late 18th and 19th Century, both in Europe and the United States. The reconstruction after the Chicago fire of 1871 led to a new style of building construction using the newly developed steel in a framework that included columns in the walls to carry the vertical loads permitting the walls to be much smaller and lighter.

At about the same time as the modern steel frame was developing, the Mino-Owari, Japan earthquake of 1891 led to the first modern-day attempts to design structures to rationally resist earthquakes. The successful performance of steel framed structures to resist both the San Francisco 1906 and the Tokyo 1923 earthquakes encouraged engineers to demand this type of construction for all truly large structures. Subsequent earthquakes have tested much larger structures and so far the performance has been very good.

FORCES AND MATERIALS

Earthquake forces - unlike most other forces that engineers must consider - are really unknown but are known to be so large that it would be prohibitively expensive to design the resisting elements in the elastic range. When we say that the forces are unknown, we mean that all of the nice straight or curved line plots we see and use as design guides are "averages" of widely scattered plotted points that look like a shotgun pattern using a log-log graph as a target. This "average" may be raised a standard deviation or two as an attempt to account for the large data spread. For certain critical areas, such as close in records of very large earthquakes there are not even any points to plot, so assumptions are made. Different researchers use different assumptions and arrive at very different results. But we do know that the forces are large, cyclic and erratic, so that the knowledge of the performance of the structural material far beyond the elastic limit is of prime importance.

In the design of structures to resist earthquakes, it is the quality of toughness or ductility that is more important than mere brute strength. Few engineers seem to realize this. It is a characteristic of modern steels that they can be strained dependably and repeatedly far beyond yield. It is this quality of steel that makes it the preferred material for earthquake resistance. The hysteresis curves at large strains are remarkably stable. This is important in resisting earthquakes since the structural material is strained above the elastic limit in both directions for many cycles. The number of cycles depends upon the length of time of shaking. The length of time of shaking in turn varies somewhat with the size of the earthquake.

Steel framing has other advantatages. It ties the building together so that it acts as a unit. It is light as compared to the loads it must resist and is probably the most uniform and dependable of all structural materials. It is easy and fast to erect.

Steel framing, however, has some disadvantages. It must be protected from corrosion and from exposure to fire. The very thick steels that are necessary for large buildings may not be as uniform as engineers may desire. The ductile properties of the more usual sizes and thicknesses of steel - say up to 1-1/2 or 2 inches - are often lacking in the thick welded steel details where triaxial strains become important. Steel framed structures that do not use braced frames or shear walls can be quite flexible and if current drift limitations are used as a design standard, the PΔ forces may be such as to to cause instability.

One of the disadvantages of structural steel concerns its availability in many parts of the world. Except for the largest, most unusual or critical structures, local economic conditions may dictate the use of other materials. Those of us who practice engineering in regions where steel is plentiful and our technicans and workmen know how to fabricate it are fortunate.

TYPES OF FORM

The performance of any structure in a major earthquake is very dependent upon the form of the structure and this is no less true of a steel structure than it is for structures using other materials. The form of the structure can have two aspects - the exterior form such as shape, continuity, regularity and the distribution of mass and resistance and the interior form such as redundancy, type of resisting elements, choice of details, etc.

Concerning the exterior form, the ideal, of course, is a symmetrical, regular, rectangular shape without set-backs and with regularly distributed mass and resistance. Because of utilitarian restraints and architectural fashions, we rarely encounter such a structure although our basic codes, standards and design guides are based on that concept. This discrepancy may not be of great importance for gravity or wind loadings where the forces are known (or can be approximated) and where the load resistance is expected to be within the elastic limit of the structural material. However, for design earthquake forces that are expected to strain the structure beyond the elastic limit by a factor of several times, many ordinary concepts familiar to the structural engineer are not valid. The importance of this limitation is specially great for the non-symmetrical, irregular building.

ANALYSIS

Consider first the principle of superposition - a principle which we all use and is valid for elastic structures. A structure can be analyzed for each loading condition separately and the resulting stresses or forces combined as desired. Loadings can be repeated and each loading cycle will cause comparable stresses. Loads can be reversed by changing sign. This is especially convenient when designing complicated, irregular structures. However, after the elastic limit is reached and hinges form, loads cannot be combined in this way. Repeated cycles of combined loadings may not give repeating stresses, i.e. the stresses may be different for each cycle of loading. A more detailed discussion of this problem with illustrations, can be found in Reference 1.

At the present time, there are no non-linear computer programs that are reasonable and adequate for use in a building design engineering office. Universities and research firms have studied and reported the effects of earthquake loadings on regular structures. They have made parametric studies of a few of the irregularities. Some of the results of the studies on regular buildings have been incorporated into codes and standards but there is little quantative data for the design of the irregular building. As a result, the designer of an irregular building has to recognize that he must pay a premium of (1) less accurate analysis and (2) increased safety factor to account for the uncertainties caused by his choice of form.

Some of the most troublesome problems are those arising from required irregularities in the distribution of mass, strength, and stiffness throughout the height of a tall building. The most common illustration of this problem is the building having a high flexible first story. It is similar to a massive, heavy, stiff, building supported on stilts. A similar discontinuity can exist at any location in the height of a building, where the regular framing is changed for some reason or other.

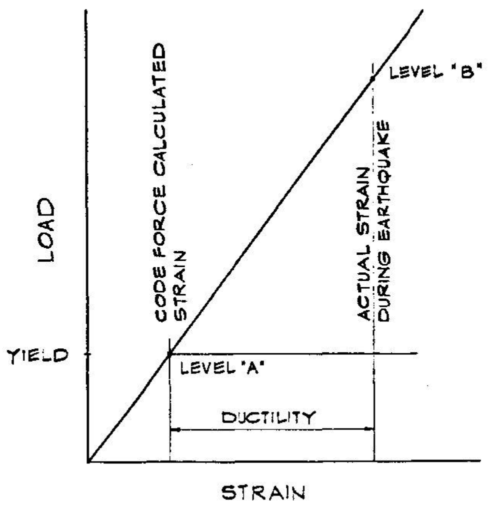
The normal building code or standard requirements are based on a uniform vertical distribution of strength, stiffness and mass. When applied to the irregular building these usual formulae for period and vibration characteristics are not applicable and consequently the base shear and distribution of the lateral loads throughout the height are not valid. If the loads are small enough so that the strains are within the elastic limit of the structural material, the usual dynamic analysis, either by time-history or modal analysis, can furnish a reasonable basis for design. However, when the strains are beyond the elastic limit, even the usual dynamic analytical methods do not indicate the true response. The actual deformations at the critical locations are generally much greater than the elastic dynamic analysis would indicate. If the engineer were to design strictly according to the results of his dynamic analysis as well as to the requirements of normal codes, the structure would be seriously overstressed at certain critical locations. When the earthquake hits, failures occur as they did at the Olive View Hospital in San Fernando and the Terminal Hotel in Guatemala City. This weakness has been observed in many earthquakes.

For those whose everyday work is not constantly related to earthquake resistant design, a quick review of the ductility effects in earthquakes may be in order. The ductility required to resist a given earthquake is illustrated in Figure 1.

If a given single degree of freedom structure is constructed using a strong elastic material and subjected to the ground motion of the design earthquake, it would respond at a load and deformation at level "B". If the material is ductile and is not strong enough to reach level "B", it will yield at level "A" and "stretch" to the same deformation as "B". This bilinear stress-strain curve is the "ideal" ductility usually assumed in many analytical studies.

Some material may increase in load capability under progressive cycles in cyclic loading and some material may decrease. But for simplicity in calculations, most research and analytical work has been performed assuming a bilinear or trilinear stress-strain curve.

If the material in the structure is ductile enough to reach the actual strain at level "B" (neglecting secondary stresses such as PΔ), the structure will not fall. This has been proven for single degree of freedom structures and is assumed to be true for multi-degree-of-freedom structures.(2) A corollary



CALCULATION OF DEFLECTION OR DRIFT

FIGURE 1

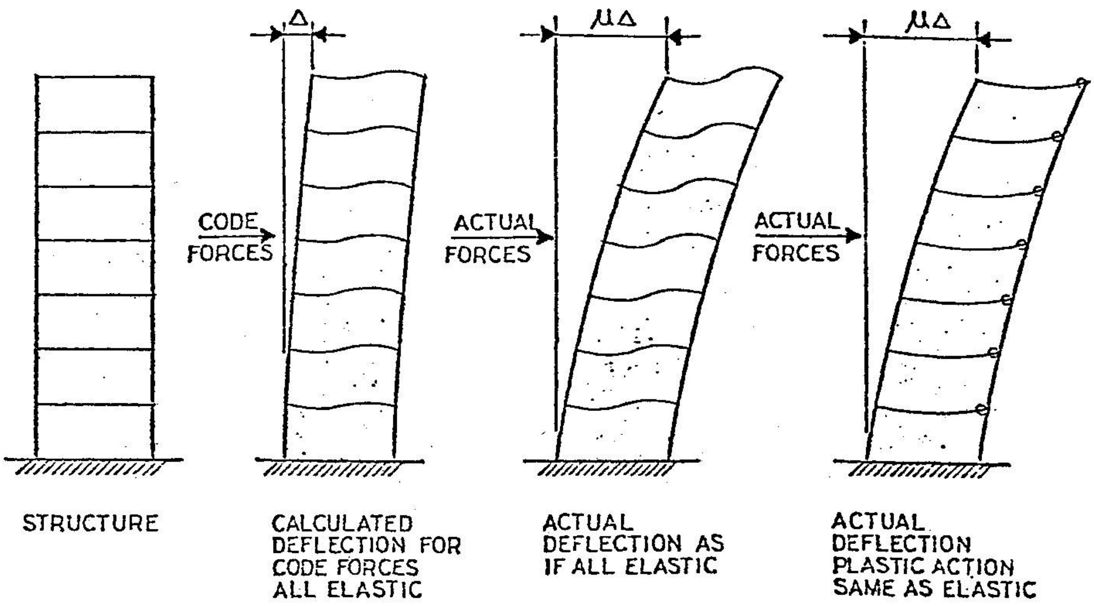
of the above statement is that if code level forces are specified at level "A" to account for ductility, it must be recognized that the actual deflections or drift of the structure in the design earthquake will be at level "B" - generally several times that calculated for the code level forces.

EXTERIOR FORM

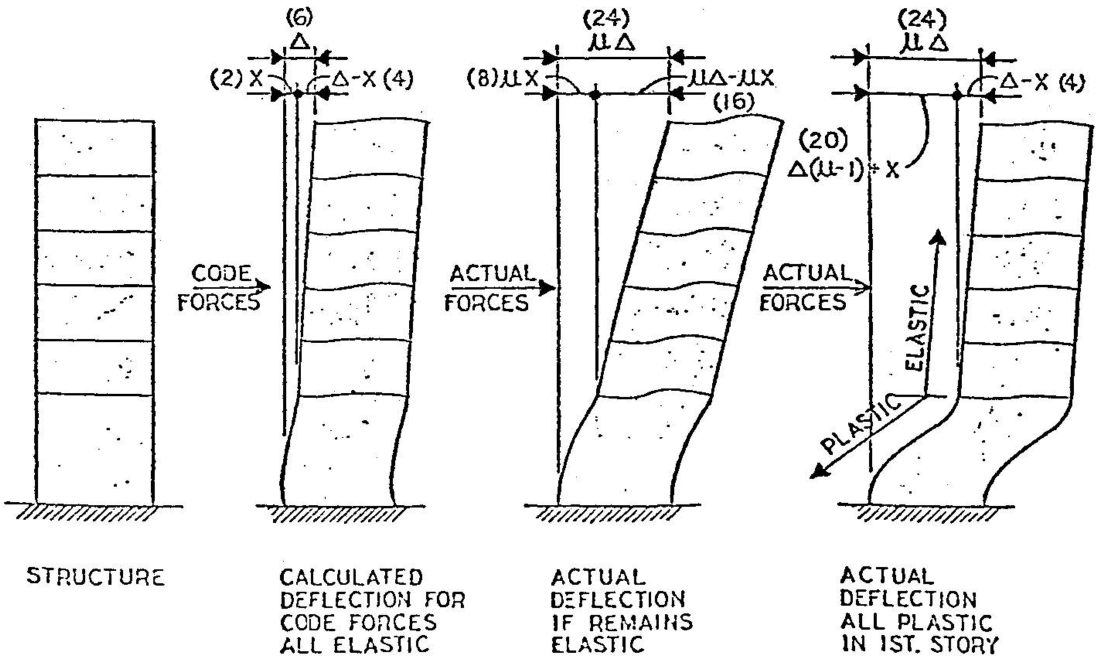
How does ductility influence the choice of exterior form of the building?

As an example consider a typical building frame for the seven story structure shown in Figure 2. If we consider the average structure in the top portion of the figure where the mass, stiffness and strength are well distributed along the height, the code forces will indicate a deflection at the top equal to Δ. But we know that the actual forces are greater by the ductility factor and from the principles discussed above in Figure 1, the actual deflection will be the ductility factor times Δ (point B instead of point A in Figure 1). If the properties of the structure are uniformly distributed throughout the height, this increase in deflection is more or less uniform, permitted by the hinging of the girders (or columns) throughout the height of the building.

Now if we consider a similar structure with a "soft" first story as shown in the lower portion of the figure, the code forces will give an elastic deflection as shown at the left. This is made up of the elastic deflection of the first story "X" plus the deflections from the rest of the structure Δ-X. The actual earthquake forces will again cause the total structure to deflect by the amount of ductility times Δ as indicated in the center bottom. But if the structure above the first story is so stiff and so strong as compared to the first story that the first story must absorb all of this excess deflection in



AVERAGE STRUCTURE



SOFT FIRST STORY

ASSUMED $X = 2$, $\Delta = 6$, $\mu = 4$

FIGURE 2

the plastic range and the top remains elastic, it can be seen that the first story is deflected much more than an elastic analysis would indicate.

If a given strength and ductility combination are satisfactory for the uniform building and the upper stories of the flexible base building, the first floor of the flexible base building would require much more strength and ductility. The same principle applies to any other major discontinuity in the height of a building. The elastic analysis does not indicate the true dangers of these discontinuities even when dynamic methods are used and many engineers are trapped unknowingly by their architectural clients who like their buildings to appear as if they had no visible means of support. Every recent earthquake has its examples of the poor performance of this form of building.

Without going into detail, buildings with setbacks may have similar problems. Usually a dynamic elastic analysis should give results as reliable as for a more regular building, but the designer must be very careful at major stress concentrations and transfers.

In the earthquake analysis of a structure, it is usually assumed that the entire base of the structure is subject to the same ground motion and the motions of all portions of the base are in phase. Actually, this is not true, since the various earthquake waves come from some source and have a finite velocity. This has two effects. One, described by Yamahara(3) is that high frequency waves tend to be out-of-phase, considering the foundation as a whole, and so reduce the response of the superstructure. This is beneficial. The other effect is to induce torsion into even a perfectly symmetrical structure.(4) The amount of research on torsion in multistory structures has been limited, and in the usual dynamic analyses of buildings, the various modes of vibration are uncoupled. As a result there is a considerable amount of uncertainty as to the actual torsional effects.

This uncertainty is greatly magnified when the form of the building is such that large known torsions must be resisted. To provide some measure of protection, many seismic codes require that a building be analyzed and designed for a minimum "accidental" torsion. In practice, this is often ignored.

In view of the various factors discussed above, as well as others, it can be seen that the external form chosen by the building designer has a major effect on the performance of the building in a damaging earthquake. Known loads can be reliably resisted where all factors are known. Sufficient research has been performed on regular, symmetrical buildings of the "usual" proportions so that the effects of various uncalculated parameters can be estimated by the experienced engineer. However, when an unusual shape is encountered, where there are discontinuities in the vertical distribution of mass, strength or stiffness, or where a building has an inherent imbalance between the location of loads, and the location of resisting elements thereby causing torsion, the engineer has to make estimates - really guesses - based on his experience because the analytical methods readily available are not valid in the load and deformation levels reached in damaging earthquakes.

INTERIOR FORM

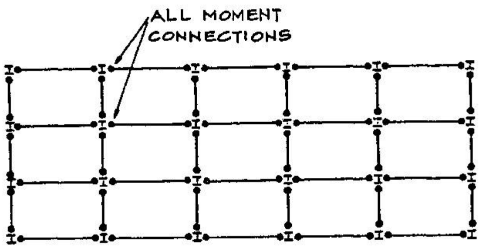
The other aspect of seismic resistant design relates to the interior form including such factors as choice of framing system, redundancy, choice of and inter-dependency of resisting elements and choice of details.

The first factor to consider in a structural steel building is the choice of framing system. Three general choices are available: 1) all moment frame, 2) shear wall, or braced frame, and 3) a combination of "dual" system, combining the moment frame with either shear walls or braced frames. Each has its advantages and disadvantages.

MOMENT FRAME

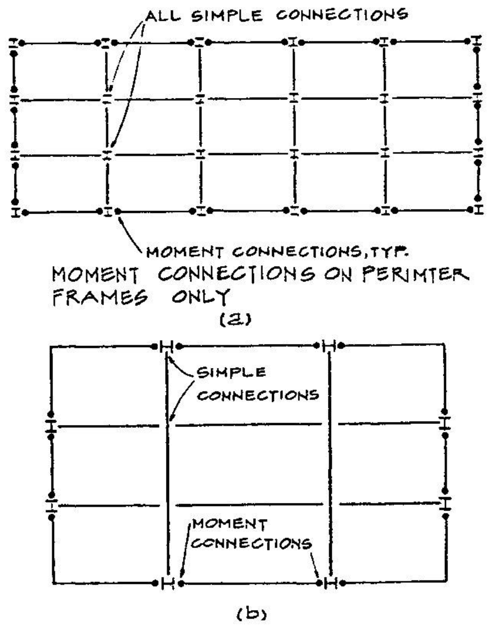
The moment frame, wherein all lateral forces are resisted by moment resisting connections between columns and beams is the system generally preferred by architects since there are no permanent structural walls nor diagonal members and so permits the greatest freedom in space planning. It is also the system generally discussed in research papers and because of its relative simplicity is the easiest to analyze and design. On the other hand, it is by far the most flexible permitting greater movements and non-structural damage in strong earthquakes and requires more steel to resist a given size of earthquake. Because of the greater deformations that it will undergo, it is subject to larger secondary stresses and may be subject to damaging mechanisms that have not been foreseen in present and past research studies of damage observations

Traditionally, in the moment frame system, all columns and beams were moment connected as in Figure 3, thereby attaining the maximum redundancy and reliability. In recent years, the tendency has been to make only certain members parts of the resisting elements, as shown in Figure 4. This reduces redundancy to the minimum possible in order to reduce costs. Obviously, for certain loadings as for earthquakes, if all other factors are equal, the greater redundancy reduces risk of collapse. An extreme case is shown in Figure 4(b) where the collapse of a single column ensures the collapse of the building. Note that, when using cast-in-place concrete frames, all connections are moment resisting. In a structural steel frame, the designer has the choice of making his beam connections either simple or moment resisting. This may be one case where a freedom of choice can lead to an inferior building.



PLAN OF TYPICAL BUILDING WITH ALL COL.-BM. CONNECTIONS MOMENT RESISTING

FIGURE 3



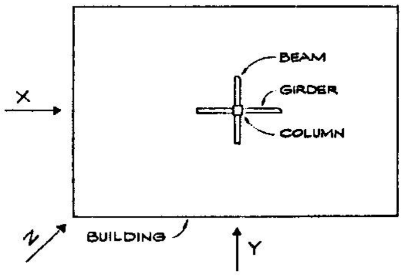
PLAN OF TYPICAL BUILDING

EXTREME CASE OF PERIMETER MOMENT FRAMES

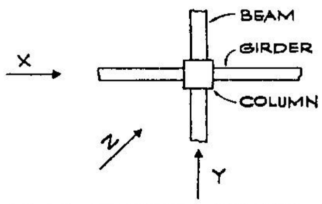
(b)

FIGURE 4

In the past, it has been a common practice to consider wind or earthquake loadings to occur in two orthogonal directions but not simultaneously. With the present emphasis on clean moment frames, matching the code forces with available ductility, and the effort to keep hinges out of the columns, it becomes necessary to review the stresses when the earthquake forces are oriented in an intermediate direction. From Figure 5, it can be seen that, as far as beams and girders are concerned, the loads on the main axes create the maximum stresses, but the diagonal direction requires a greater demand for strength or ductility.



PLAN OF BUILDING



PLAN OF BEAM, GIRDER & COLUMN

CALCULATED STRESS IN ELEMENT

	COLUMN	BEAM	GIRDER
100% FORCE IN X-DIRECTION	100%	0	100%
100% FORCE IN Y-DIRECTION	100%	100%	0
100% FORCE IN Z-DIRECTION			
X-COMPONENT	71%	0	71%
Y-COMPONENT	71%	71%	0
TOTAL	142%	71%	71%

FIGURE 5

SHEAR WALL SYSTEMS

The bracing system that is in most common use for low and medium height buildings is the shear wall system. It provides considerable stiffness for protection of non-structural elements and is usually quite economical. The category "shear wall" is very broad and there is no adequate definition at present. Figure 6 shows four general types that are often used by architects. All are shear walls but the performance of each in earthquakes are quite different. Type A, the inverted pendulum, has performed badly in the past - similar to the Four Seasons Building in the Alaska, 1964 earthquake. Type B, with the small piers, has also performed badly as illustrated by the performance of schools in the 1968 Tokachi-Oki earthquake. Type C, with large piers performs excellently and is the forerunner of the coupled shear wall system which is being advocated both in the United States and New Zealand. Type D also performs excellently. In earthquake prone regions there is usually a height limit on buildings braced entirely by shear walls. The counterpart of the concrete shear wall in structural steel is the braced frame. The most economical bracing systems are those with concentric connections using any of

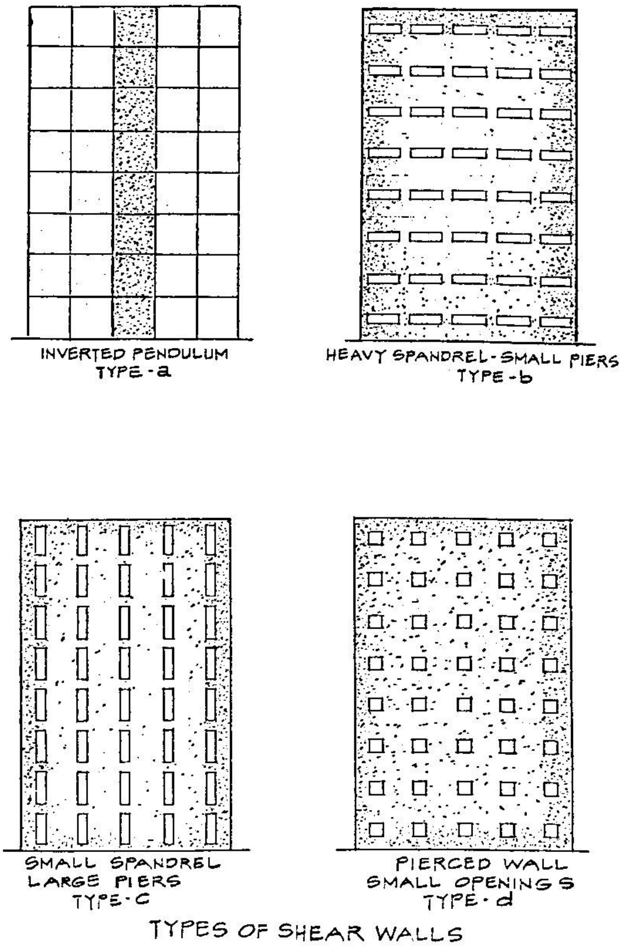
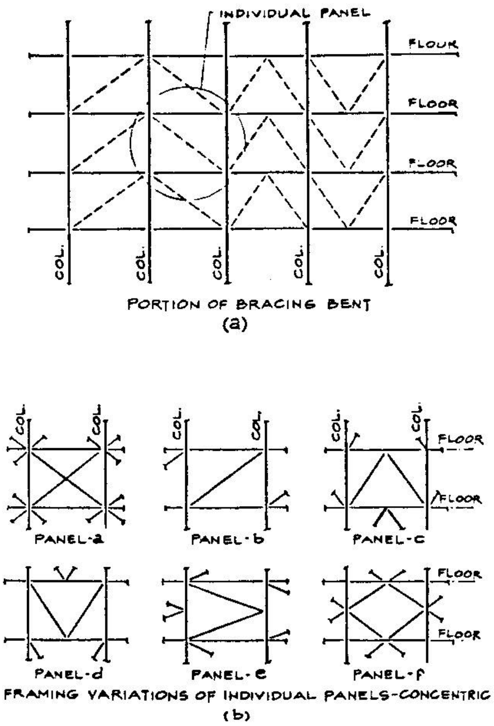


FIGURE 6

the patterns indicated in Figure 7. While these systems can readily be designed to resist code forces, they do not have much ductility. In past earthquakes, members have buckled and connections torn apart. Recently a considerable amount of study has been directed at making the joints eccentric as shown in Figure 8 in order to be able to absorb large amounts of energy. A more detailed discussion is presented in Reference 1 and the references given therein.

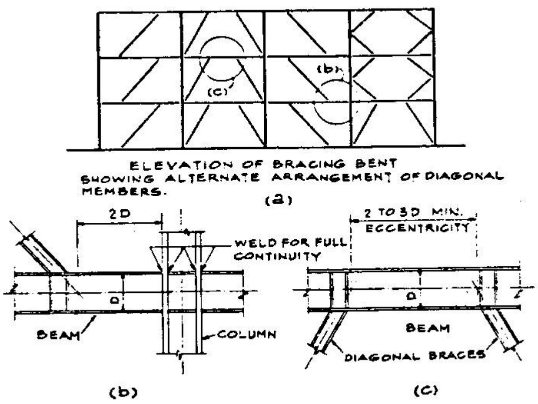
DUAL SYSTEM

In the writer's opinion, the best, most efficient and safest framing system for moderate and high rise buildings in earthquake country is the dual system.



Generalized layouts of normal braced frames with (a) showing general layout where each individual panel may have various configurations as shown in (b).

FIGURE 7



"Braced frame" bracing arrangements using eccentric connections to absorb energy. (a) Elevation of bracing bent showing alternate arrangement of diagonal members.
(b), (c) Enlargements of areas shown in (a).

FIGURE 8

This combines the shear wall or braced frame to give stiffness with the moment frame with the moment frame furnishing the backup strength with a ductile moment frame. It is this general type of framing that has given the best performance in past earthquakes and has led to the excellent reputation of structural steel framing in earthquakes. It is also an example of a system furnishing ductility as opposed to the material only. It can be seen that this system has the maximum amount of ductility.

DETAILS

After selecting the exterior form of the building and the general type of framing system, a choice of type of detail must be made. In structural steel, there are two general types of details used that depend on the form of the column. Connections are or should be moment resisting and today they are usually welded. In low and medium height buildings the columns are usually H sections. In high rise construction, they may be either H sections or box sections. The H section columns usually have the traditional type of moment connection using stiffeners in the bosom of the column. It is necessary here to accommodate the shear stresses in the panel portion of the joint. One disadvantage of this connection is the fact that bending capacity in the weak direction of the column is only about one third that of the strong direction. However, this type of detail has been tested in earthquakes and has been found to be reliable.

In an effort to balance the bending capacity in both directions, the box column has been developed in recent years. This is a welded box made up of plates. The stiffeners needed at the top and bottom flanges of the beams generally must be placed on the interior of the box and this creates some difficulties in welding. Very often the plates needed to form the column must be quite thick and the restrained situation at the stiffeners has caused plate cracking on several projects. While this system is theoretically more efficient than the H section system, it has not been tested in a major earthquake. The combination of very thick plates and welding raises some apprehensions as to the ductility that may be available.

SUMMARY

From the discussion above, it can be seen that the forms chosen in the design of a building have a major effect on the building's performance in a major earthquake.

The exterior form - its regularity, distribution of mass and strength, and arrangement of resisting elements - may have more effect on the building's performance in an earthquake than the engineer's calculations. The regular compact, symmetrical form has fewer unknowns whose effects the engineer must estimate.

Similarly, the choice of framing scheme and details - those items making up the interior form - determine the reliability and redundancy of the structure. Under loadings that cause major strains such as major earthquakes, these choices are of more influence on performance than the size of earthquake coefficient.

REFERENCES

1. H. D. Degenkolb, "Practical Design (Aseismic) of Steel Structures" Canadian Journal of Civil Engineering, Vol. 6, No. 2, 1979, pages 295-298 and pages 303-307.

2. R. Clough and J. Penzien, "Dynamics of Structures" McGraw-Hill, 1975, page 602.

3. H. Yamahara, "Ground Motions During Earthquakes and the Input Loss of Earthquake Power to an Excitation of Buildings" Soils and Foundations, Vol. X, June 1970, No. 2, page 145 - The Japanese Society of Soil Mechanics and Foundation Engineering.

4. J. E. Luco and H. L. Wong, "Response of Structures to Non-Vertically Incident Sesimic Waves" Bulletin Seismological Society of America (In Press).

5. H. J. Degenkolb, "Sesimic Design-Structural Concepts" Summer Seismic Institute for Architectural Faculty, Page 65, 1977 AIA Research Corporation.

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