LIMITATIONS AND UNCERTAINTIES OF PRESENT STRUCTURAL
DESIGN METHODS FOR LATERAL FORCE RESISTANCE

by

Henry J. Degenkolb (I)

ABSTRACT

Some of the limitations and uncertainties in structural earthquake resistant design can be classified as those due to (a) lack of fundamental knowledge; (b) knowledge that is available but not in the form where an average structural engineer can use it in an average building; (c) the limitation on available design time due to size of design fee or budget and, (d) construction practices and techniques available in the area where the project will be built.

Some of the technical uncertainties concern the stresses and consequent strength of joints of both concrete and structural steel; the amount of vertical load variation in the design of columns with bending, especially due to the fact that vertical accelerations are neglected; relative rigidity of resisting elements, especially under higher modes of vibration; required load factors and ductility ratios for any given motion; and, fundamental to all, the lack of basic ground records for strong motions.

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It is appropriate, after considering some of the uncertainties and limitations relating to ground motion, foundation problems and structure-soil interaction, to examine some of the problems relating to the detailed structural design of a building, especially as practiced in a design office.

First, let me state that the end result of all of our research, study, conferences, and the day-to-day work of every one of us is to provide structures that will be safe for the public to use and occupy. They should be safe even during a strong earthquake. Hopefully, they may even be economical.

Too often, the seismologist, the geologist, the foundation engineer and the dynamic analyst think that if they could only give the structural engineer accurate information relating to loads, strains, periods and motions, the engineer will be able to provide for them safely and economically with little or no uncertainty.

As structural engineers, our best reply is that possibly our public relations program has been too successful among fellow professionals even if not very successful with the public. Our limitations are probably of the same order of magnitude as those of the other professional groups, even if many of the engineers are not completely aware of that fact. These limitations can arise from various sources.

Looking to our primary goal of providing safe structures for public use, we must relate our limitations to the actual practice of structural engineering.

The first area of limitations to consider would be those concerned with the lack of fundamental knowledge. This limitation relates to those areas where even theoretical research or practical research is either inadequate or missing. Examples could be the detailed design and performance of certain joints in either concrete or steel; the relative stiffness of various elements under various modes of vibration; and even the basic ground motion of the foundations as related to soil type, earthquake type and distance of structure from energy source, interaction, etc. In some cases the engineer has not even known that there is a limitation - it has been completely overlooked.

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The second group of limitations includes those items where the basic knowledge may be available but where it has not been adapted in a manner that the average structural engineer can use for the design of an average building. For example, there are many solutions to problems in column stability, shear flow, fatigue, stress concentrations, etc., that are available for use to the aeronautical engineer but are not in practical form for use by the structural engineer.

A third limitation relates somewhat to the last one but is so highly variable and is so important from a practical point of view that it deserves a separate listing. This is the economic limitation on the amount of design and engineering time that an engineer can afford to furnish any specific project. His time is limited by his professional fee if he is in private practice or by his engineering budget if he works for a public body or major corporation. This limitation relates directly to the practice of structural engineering - but what level of practice? The level of practice in an area of the world that considers vertical loads only, which is minimal; the practice that designs for vertical and wind loads but not for earthquake effects; the average design office in earthquake prone areas which supposedly considers all these effects; or the level of practice of a design office that also does considerable research and study in the field of earthquake resistant design and is aware of the problems and limitations? The design costs and, therefore, the required design budget increases greatly as we raise the level or quality of practice. How much safer our buildings could be if fees and design budget were increased to the point where this limitation could be eliminated by giving each design, each detail, and each structure the time and effort it really deserved in the overall situation - not only the effort that can be afforded from an often inadequate portion of the total cost of a project.

A fourth limitation concerns local construction practices. No engineer, no matter what his knowledge or qualifications, can move too far ahead of his contemporaries. If a certain construction practice is accepted or condoned in any given area, it is extremely difficult and it takes a very courageous engineer to design counter to these practices. One example is reinforced masonry. On the West Coast of the United States, it is customary to require steel reinforcement in masonry walls - either in reinforced grouted brick masonry or in reinforced filled-cell grouted concrete block construction. However, in most areas of the United States - the entire East, Midwest and South - this reinforced masonry construction is virtually unknown, labor for constructing it would be inexperienced, contractors' prices would be very high, and its use would be quite impractical. As a matter of fact, while we in the United States consider 8-inches or 9-inches to be the minimum thickness of brick walls many countries consider a single width of brick - about 4-inches - to be normal. Another example concerns the use of a single layer of reinforcing in concrete walls. Many areas of the world consider a single layer of reinforcing as normal in 6-inch, 8-inch, and even 10-inch thick walls and this is used occasionally even in California although it is highly unsuitable for earthquake resistant construction. All U.S.A. building codes permit it, and in some areas, an engineer who would consistently require two layers of reinforcing in his shear walls would soon be out of business for lack of clients. There are many more aspects to engineering a building for earthquake forces than the mere dynamic analysis of a bare moment resisting ductile frame.

Moving on to the more technical aspects of our topic, let us first consider two aspects of concrete construction that nominally concern bond stresses - at least in the normal design practices of most structural engineers. The allowable bond stress in concrete, for transferring stresses between concrete and reinforcing steel, has usually in the past been determined in two or three ways. First, there is the pull-out test as shown in Figure 1. Here a tension is applied to one end of a reinforcing bar, a reaction is supplied by compression on the concrete, and the bar is pulled out of the concrete. From this, the average bond stress around the perimeter of the bar can be computed, and a permissible value assigned to the contact area. You will note that a closed force system results. Incidentally, most of you are probably not aware of the fact, that in arriving at our bond stresses for the common "Hi-Bond" bars now used, that spirals of #6 or #7 gauge wire at a pitch of 1-1/2" centers were placed around these bars to prevent a low failure due to splitting of the concrete. When Ferguson, Breen and Thompson ran comparable tests on #14 and #18 high strength bars, as reported in the August, 1965 ACI Journal, 1/2-inch ø spirals with 2-1/2-inch pitch were used. According to the authors, the failures with spirals are less sudden, more tough and probably occur at higher stresses than specimens without spirals. How many engineers put similar spirals around bars in areas of high bond stresses in practical structures?

Another method of determining allowable bond stresses is through the testing of concrete beams as shown in Figure 2. Here again there is a closed force system, whereby the shear forces contribute to a change in compression stress in the concrete and tension stress in the reinforcing steel. The change in stress in the reinforcing steel is a measure of the bond stress.

Other methods of testing bond, or development length have been used such as lapping steel in beams, casting bars eccentrically in the block and in various positions, but all employ a closed force system, and where lapping occurs, the test specimens are symmetrical.

With this brief background in bond stresses, let us examine two situations in some detail.

Our first consideration is the joint as shown in Figure 3. Consider forces acting from the right so that this corner, marked "A" in the diagram, has tension on both the vertical and horizontal bars at the inside, re-entrant corner. The usual method of design is to simply anchor these opening trim bars a certain bond length past the opening. This may be about 16 bar diameters if you believe ACI, or a certain distance with a radius hook, or, if you want to be conservative and have been informed on the importance of anchorage in earthquake loadings, you might even provide 40 bar diameters. Let us examine the stresses within this intersection, however.

Figure 4 shows this joint intersection with some of the forces resulting from a lateral load acting to the left. The tension in the horizontal bar T_b must equal the concrete compressive stress due to bending C_b plus the shear on plane 0-X, V_a. Similarly, the tension force in the vertical bar T_a must equal the concrete compressive stress C_a plus the shear V_b. It is probable that the shear force V_b and the transfer of the compressive force C_a is distributed more or less uniformly along line 0-Y, so the tension in the bar T_a must be accumulated somewhat uniformly along line 0-Y with zero tension at Y and the maximum tension at 0. The required length of the reinforcing bar as shown in the portion shown solid, therefore, has nothing to do with bond stress but is a function of the dimensions of the joint, the minimum reinforcing in the wall, and the tension allowed in the concrete itself. Looking at it in another way, the sum of the tension stresses in the reinforcing bars result in a component force S acting away from the joint upward and to the right, while the sum of the compressive forces result in a component force Z acting down and to the left. The only way this force system can be closed is through tension in the concrete in the joint. It can be seen that no bond test - either beam, pull-out, embedment, or lapped splice has any relationship whatever to the length the bars must be extended past the opening.

While the above analysis may seem quite reasonable, there are absolutely no tests on such joints to prove or to disprove either this or any other theory of action of such joints. Nowhere do textbooks treat the design of this joint. No papers have been written on the subject. Very few design offices consider these stresses which may be critical.

In the discussion of bond stress, let us examine another situation which is common to all reinforced concrete construction. How many engineers check the stresses in the concrete reinforcing bars when they are doweled out or lapped? How many tests have been performed on a lapped bar construction as shown in Figure 5? What are the actual field tolerances to limit the eccentricity as shown here, and what is the effect of those tolerances?

When a wall or column is in compression under vertical load, the concrete takes most of the load and there is little concern for possible eccentricity. However, when a portion of a shear wall is in uplift as happened in several buildings in Anchorage or when a column goes into tension as happened in Caracas, there is a major eccentricity and the wall tries to act as in Figure 6. Point "A" shifts to where it tries to line up with point "B" and there is a splitting action along plane X-X which is in shear and tension. The importance of this splitting action can be understood when it is realized that the reinforcing steel did not go into yield at the core walls of the Cordova Building nor in the core walls of the Four Seasons Apartment building in Anchorage, although the walls in both buildings failed.

This lap joint and the tolerances in field placing are one of the primary reasons that walls reinforced with a single curtain of reinforcing steel are completely unsuitable in seismic areas if they might go into tension and could not be justified by any means - regardless of code provisions. Again, it is obvious that the bond considerations usually used for the design of this lap are not the critical considerations at all.

In a slight variation of this bar lap, the bond strength of the concrete can be eliminated entirely. As shown in Figure 7, some engineers feel that lap welding the bars together will eliminate their troubles. The amount of lap is calculated from the length of weld required for a direct stress transfer. Again, this completely neglects the eccentricities involved. As the bars tend to straighten out, large side thrusts "x" are introduced into the concrete. Unless the length of lap is quite long, these side thrusts are greater than the concrete can withstand in either bearing of the concrete or splitting of the concrete section.

There are several factors to consider in the design of steel connections, but I would like to emphasize only one point that is commonly overlooked.

In Figure 8, two loading conditions are shown that inexperienced designers frequently overlook. At the left is shown the shear condition in the column web between the beam flanges. Regardless of whether this is a bolted or welded joint, the stiffener plates have tension at one side and compression on the other (considering only the lateral loading portions of the beam moments) so that the shear on the weld to the column web is measured by <u>twice</u> the beam flange stress. Similarly the shear force in the column web is approximately twice the beam flange stress.

When the column is in bending in the weak direction as shown in plan, only the welds to the flanges of the column are fully effective and again the total welding to the flanges must take about twice the beam flange stress.

In considering the level of practice in a structural design office, however, the simple statement that the shear in the panel labeled 2T may be critical, does not indicate the amount of design time and labor that is necessary to investigate this. Just reflect on the office procedures. First, all beams and girders are designed with end details as required to develop the members. Then the columns are designed for the various critical loads and bending moments, The stress in the column-beam panel varies with the column size and stress, the size of the beams and girders in <u>each</u> direction and the proportion of vertical load (which causes little or no panel stress) to the lateral load (which causes most of the panel stress). As a result, the addition of this single critical condition increases the design calculations a major amount - and the engineer who may be operating on a nominal design budget may be inclined to overlook these niceties of structural analysis especially if he knows the truth that few of his competitors are looking at these details. And, worse yet, if his calculations indicate an overstress, the corrective measures are rather unusual and costly from the contractor's point of view, so the careful engineer gets only criticism for his unusual and conscientious efforts. He is going beyond the customary construction practice.

One of the major omissions in most present requirements for lateral force design involves the situation of combined bending and direct stress in columns when the direct compression may be substantially smaller than anticipated. In Figure 9, we can see diagramatically that reducing the direct compression from P to P', increases the eccentricity for a given moment M, and can cause tension stresses in the reinforcing steel where none were calculated previously. As an illustration of what can happen, let us examine a 24-inch x 24-inch spirally reinforced column designed in accordance with the provisions of ACI 318-63, working stress methods, as shown in Figure 10.

Assume 5000 lb. concrete, 40,000 lb. yield steel and column loads of 400^k dead load, 80^k partion, 120^k live load with a moment of 210 foot kips. Let us check the reinforcing required. These conditions could be similar to those occurring in a ten or twelve story building with 5% or 6% base shear. On Page 176 of ACI's Publication SP-3, we find the following interaction diagram. With 600^k compression and 210 foot kips moment, 1% steel (Point A) is required or 5.76 square inches. Since this is the greatest load on the column, it is often assumed that this is the critical condition. However, if the same moment is retained but the live load and partition loads are reduced to nothing, a check of the same interaction curve indicates that 2.2% steel is required (Point B). Now if a vertical acceleration of 33% is applied in a manner to reduce the load, the net load on the column is 300^k and 3% reinforcing is required (Point C). Usually, vertical earthquake accelerations are neglected because they are generally not critical, but here is a case where they can be very critical - and this case is invariably overlooked. The actual vertical load on a column is almost invariably overestimated in the interests of safety. However, when bending moments are involved, the critical condition may be the condition with less than the design vertical load on the column. If overturning is a factor, there may even be a tension combined with the bending stress.

It should be emphasized that the reduction in vertical load can arise from many sources - absence of live load, smaller dead load than calculated, vertical earthquake accelerations, overturning forces, torsional effects, etc. - many of which are ordinarily neglected in calculations. We must remember that tension cracks were observed in the columns of tall structures in Caracas after the earthquake, indicating tension in the columns at the same time they had to resist major bending moments.

This problem of reduced vertical load is not unique to concrete construction. In steel frames we all have observed welded column splices similar to that shown in Figure 11. For maximum compression stresses there can certainly be no complaint on this detail. But what happens if there is major bending and a reduction of vertical load? Where is the much publicized ductility in this connection when our admittedly nominal code lateral forces are exceeded by a factor of several times and one flange or an edge of a flange goes into tension?

ıat about the stress concentration factor at the root of the weld?

Reviewing column design criteria in both concrete and steel, and :obably in the design of shear walls, it may be advisable for our ıilding codes to also specify a vertical earthquake acceleration, or : least a major reduction of compressive load combined with bending, : individual engineers do not take the initiative to investigate the :itical conditions in their own designs.

Recently our office had occasion to review the design strength of ırious concrete columns as related to various recent standards. ertain columns subjected to vertical compression and biaxial bending ıe to both vertical and lateral loads were reviewed and the following cample shows one condition that was found in an actual structure.

In Figure 12, a certain column 11.8-inches x 23.6-inches with 2 - #7 50,000 lb, yield stress bars and 4260 lb. concrete had a esign vertical load of 394 kips with vertical load bending of 78 ft. ips in the strong direction. Lateral bending moments were 38.3 ft. ips and 58 ft. kips as shown. By the criteria we are used to, the olumn looked light, so it was checked by the present San Francisco uilding Code which has the provisions of ACI 318-56, also used in the 964 UBC. Under these standards, it was 54% overstressed under vertical oad conditions and 55% under lateral load conditions using the code llowed 33% stress increase for earthquake loads.

In reviewing the same column for ACI 318-63 criteria, working stress esign, it was found that it was used to 96% of capacity under vertical oad and 2% overstressed under lateral load. This indicates a very cometent design. This code criteria is accepted for use in the 1967 dition of the Uniform Building Code, is accepted by the Structural ngineers Association of California, but has not been accepted as yet n San Francisco.

Under the ACI 318-63 provisions for ultimate stress design the olumn is used to only 80% of capacity under vertical load and 93% of apacity under lateral load design if Boris Bresler's method of bixial bending calculations are used and is stressed to 78% of capacity f Jacobsen's method of calculation is used. In other words, under the atest criteria this column was only used to 80% of capacity, the designer as quite conservative and could have used an even smaller column to carry he specified loads. This criteria has been adopted in the 1967 Uniform uilding Code, it is recommended as the preferred design method by the tructural Engineers Association of California in their 1967 Lateral orce Commentary (Page 65) and is recommended for adoption in the forthoming revisions to the San Francisco Building Code.

Now, when a major earthquake hits a building, it is affecting a iven structure with a built-in amount of structural steel, concrete, einforcement and consequent strength. The earthquake doesn't know hat method the designer used to arrive at that strength. Are we strucural engineers willing to say that in the last three years we have learned so much about earthquakes and now know so much about the structural performance of our designs that we can reduce our column strengths almost in half?

How do the analytical studies by many dynamic research workers on the required ductility ratios relate to the new column stresses? If certain ductility requirements were deduced from studies made in 1963 or were based on members sized under the old code, can the same ductility requirements be used on members only about half as strong?

This reduction-in-size concept is not limited to concrete. In structural steel it's called plastic design and by its use we can now design continuous beams for $\frac{Wl}{16}$ in place of $\frac{Wl}{12}$ - a considerable reduction. With composite design, we can use even smaller members. Where loads are known, these more logical methods of design might be quite appropriate, but to date, who has been able to tell us the magnitude of the forces our buildings are subjected to in a major earthquake? Certainly a review of the failures in Caracas or in Japan give us little comfort in the adequacy of our assumptions.

Every building code in the world that regulates earthquake resistant design contains a brief statement that forces must be distributed to various resisting elements in proportion to their relative rigidities. To the best of my knowledge no code in the world - at least none that has been written or translated into English - defines just what we mean by relative rigidities. Why should it be defined? Every engineer learned how to calculate rigidities at least by his senior year in college, and all of us know all about it. Or, do we?

Looking at Figure 13, let us consider the case of two resisting elements in a tall building - one a slender shear wall as shown on the left, and the other a structural frame as shown on the right. For this paper, we need not consider the complexities of shear wall-frame interaction. For the fundamental mode of vibration, and for the proportions of frame members usually found in practical structures, a load at a lower floor such as at "A" would go mostly into the shear wall. At this location the shear wall is much stiffer than the frame since shear effects are of the greatest importance. When considering foundation yielding, shear, column and girder deformations in both bending and axial loads, it will be found that for loads at "B", the frame is usually much stiffer than the slender shear wall. All loads, "A", "B", "C", and "D", must be resisted at the foundation, and all rigidities calculated accordingly. From any given frame dimensions and the physical constants of the structural materials and the foundations, a fairly accurate calculation can be made for the relative rigidity of the shear wall as compared to frame. It will vary from floor to floor and the loads at various floors will interact in a highly redundant and complex manner, but with modern computers, this is a minor detail, very definite calculations can be made with considerable assurance as to their reliability. However, it was assumed that the structure will vibrate in the fundamental mode. When we examine the higher modes of vibration, we find that the <u>relative</u> rigidities completely change, as shown in Figure 14. Loads "C" and "D" now do not span to the ground as they formerly did, but tend to partially cancel each other, and the shear wall stiffness greatly increases as compared to the frame. This means that the shear wall is taking a much larger proportion of the total load than was assumed for the calculations based on the fundamental load. It is not necessary to review the higher modes of vibration, for they would only further illustrate the same effect.

The importance of this observation concerns the fact that even if the total lateral load on the structure is known, the distribution of that load to the varying resisting elements is unknown unless they are very similar to each other. The structural engineer's final design of member sizes and strengths will depend on the loads and the distribution of loads to the various elements.

There is no known way in which the effects of the varying rigidities can be incorporated in static design calculations unless the proportion of load that can be assigned to each of the modes of vibration is known. The only mathematical solution requires the performance of a dynamic analysis. For a valid dynamic analysis, a known ground motion is necessary. The type of ground motion will greatly influence the relative rigidity of the various resisting elements. If the response spectrum shows that long period motions predominate, relative rigidies based on the fundamental mode may be appropriate but if the ground motion is rich in short period vibrations, the basic relative rigidities may vary greatly from that assumed. Here the lack of ground motion records again is critical. There has not been one, repeat NOT ONE, strong motion record obtained in any major city in the world. The best record to date is that of the moderately strong 7.1 Richter earthquake obtained in 1940 near the little town of El Centro, California, with a maximum response near the quarter second range. What validity does this record have in downtown San Francisco or Seattle or Anchorage where the dominant period is probably in the full second range? What validity for even the basic determination of the relative rigidities of the resisting elements, to say nothing of the response of a structure as a whole?

In order to arrive at some basis for the design of our structures to resist earthquakes, the engineering profession is forced to adopt one of two approaches or some combination of them. As one approach, we can take an assumed ground motion, run a dynamic analysis on the structure, use our engineering knowledge to assign the loads to their proper resisting elements, and then proportion the members and their connections to resist these loads. This can be done on either an elastic basis or on an elasto-plastic basis. This approach has certain difficulties. First, we do not have a ground motion record of a strong or great earthquake on the type of foundation material on which our big buildings are located. Secondly, while much research has been performed on the response and various parameters that govern response, all of it has been done on nice, regular, simplified elements. There has been little or no work done on models, either physical or mathematical, that resemble an actual building including its torsional motions, its variation in stiffness or weight throughout its vertical height, its discontinuous resisting elements and many other actual practical factors. Thirdly, there has been no work on the relative rigidities of differing elements under various dynamic loadings. And finally, there are great gaps in our knowledge relating to the physical performance of the structural elements under earthquake loadings. Such work that has been done on regular idealized models of buildings indicate that even with the moderate El Centro Earthquake motion, and under our present code prescribed forces, the full ductility ratio capacity of our structural materials is required - or may even be exceeded.

The other approach on which to base our designs is on the performance of our structures in the past - and this has been the basis for our code forces, because major buildings in San Francisco, up to nineteen stories in height, did survive the 1906 earthquake successfully. However, the type of buildings we are constructing now do not even remotely resemble the structures on which our codes are based. In place of the integral masonry walls of the past which cracked and absorbed energy, but still provided stability, we now have clean, bare frames with loose, non-structural curtain walls, partitions that are deliberately kept free of the structure to prevent damage, lightweight metal decks and a completely different type of structure. It is probably completely erroneous to extrapolate the successful performance of the old structures into a basis of design of our present, completely different structures - at least as the only approach.

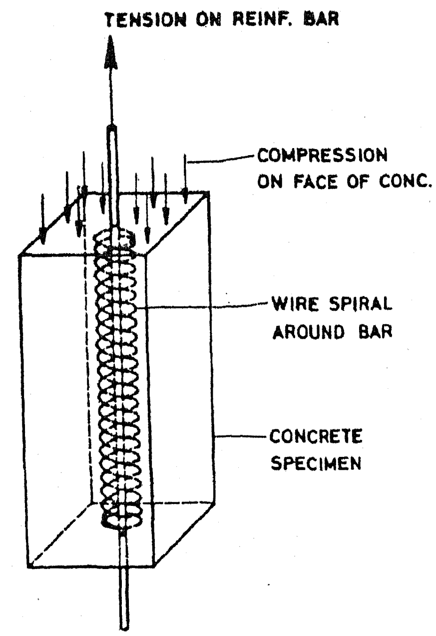
There are many other areas of uncertainty in our structural designs that could be covered but time does not permit more than a brief mention of even one.

All buildings are subjected to torsion, either designed or accidental and the building codes recognize this by requiring appropriate calculations and strengthening. How this is handled in actual design is subject to the same uncertainties of rigidity and strength mentioned earlier.

SUMMARY

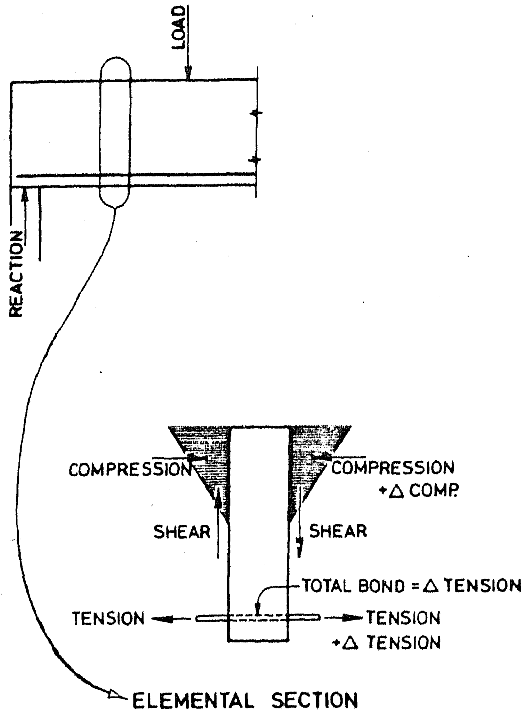
In summary, some of the limitations and uncertainties in structural earthquake resistant design can be classified as those due to (a) lack of fundamental knowledge; (b) knowledge that is available but not in the form where an average structural engineer can use it in an average building; (c) the limitation on available design time due to size of design fee or budget and, (d) available construction practices and techniques available in the area where the project will be built.

Some of the technical uncertainties concern the stresses and consequent strength of joints of both concrete and structural steel; the amount of vertical load variation in the design of columns with bending, especially due to the fact that vertical accelerations are neglected; relative rigidity of resisting elements, especially under higher modes of vibration; required load factors and ductility ratios for any given motion; and, fundamental to all, the lack of basic ground records for strong motions.



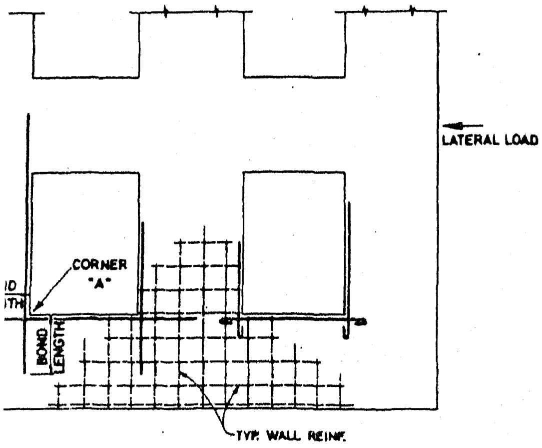
OUT TEST FOR BOND

re 1.



BEAM TEST FOR BOND

Figure 2.



RTION OF FRAME OR SHEAR WALL

·e 3.

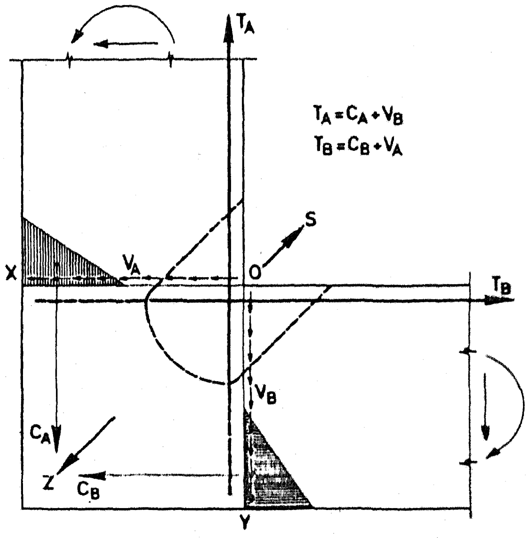
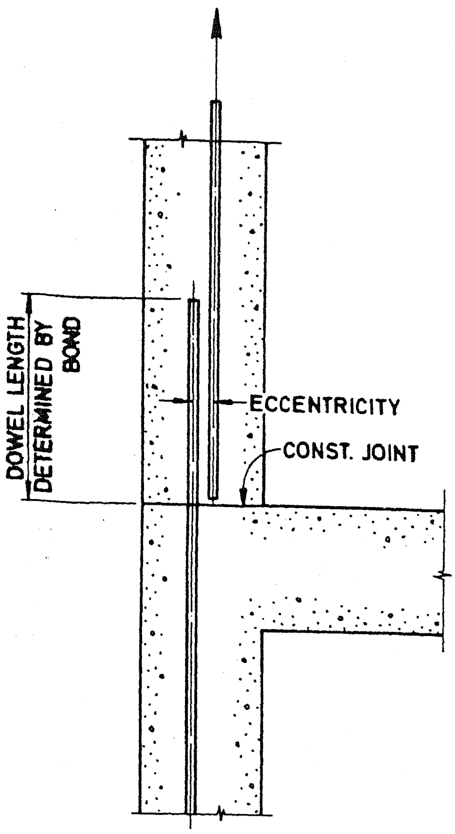


Figure 4.



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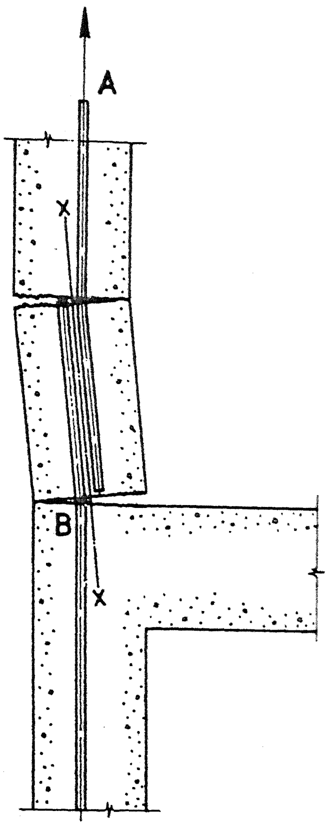


Figure 6.

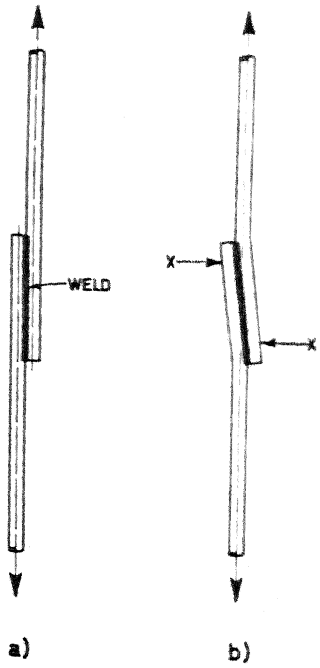
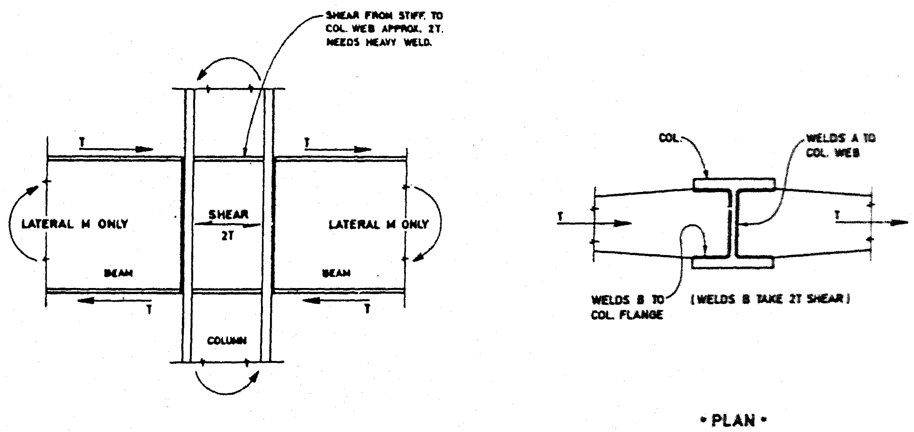


Figure 7.



BEAM CONNECTED TO FLANGES

PLATE FOR BEAM CONNECTED TO COL. WEB

Figure 8.

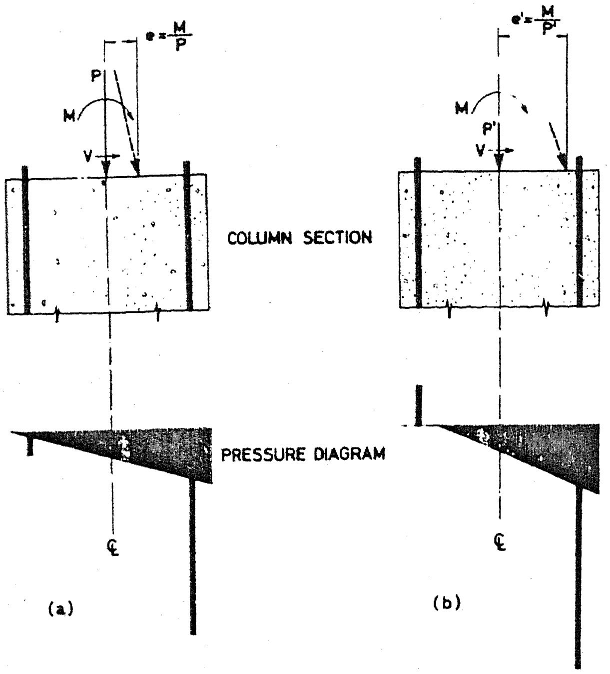
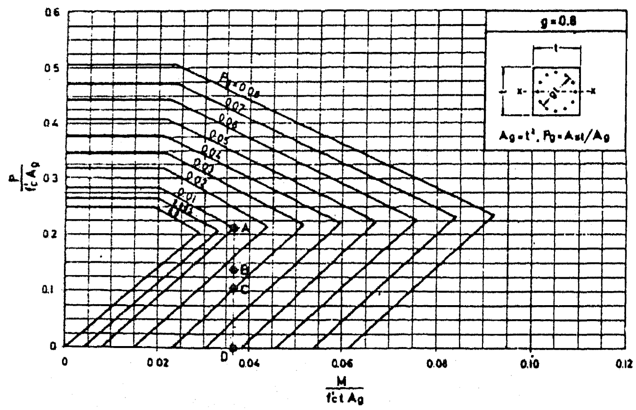


Figure 9.



ASSUME 24" × 24" SQ. COL. WITH SPIRAL

P →	DEAD LOAD	400^K
	PARTITIONS	80^K
	LIVE LOAD	120^K
		600^K

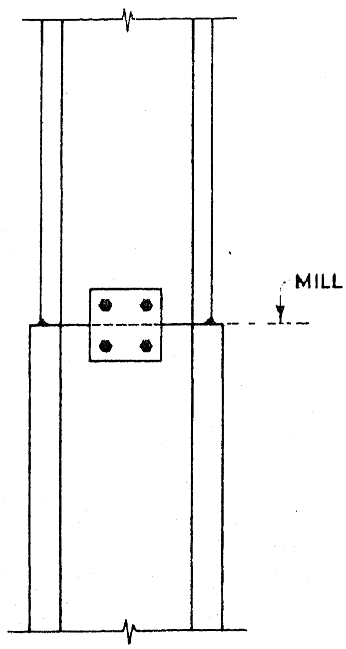
ASSUME M = 210 ft. kips

CONCRETE: $f'_c = 5{,}000\ \#/\square''$ STEEL $f_y = 40{,}000\ \#/\square''$ YIELD.

THEN $\frac{M}{f'_c t A_g} = \frac{210{,}000 \times 12}{5{,}000 \times 24 \times 576} = 0.0365$

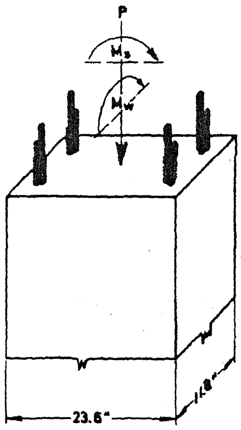
A)	$\frac{P}{f'_c A_g} = \frac{600{,}000}{5{,}000 \times 576} = 0.208$	1.0% STEEL	5.76 □"
B)	FOR $P = 400^K \rightarrow 0.139$	2.2%	12.70 □"
C)	$300^K \rightarrow 0.104$	3.0%	17.30 □"
D)	$0^K \rightarrow 0$	3.8%	22.00 □"

Figure 10.



COLUMN SPLICE

Figure 11.



12 #7 BARS - BUNDLED $p_g = 0.0258$
$f_y = 50{,}000$ p.s.i. $f'_c = 4{,}260$ p.s.i.

WORKING STRESS DESIGN

1/3 INCREASE IN STRESS ALLOWANCE

ULTIMATE STRENGTH DESIGN

LOAD FACTORS — 1.5 D + 1.8 L

— 1.25 (D + L + E)

SEAOC RECOMMENDS 1.4 (D + L + E)

LOADS

	VERTICAL	SEISMIC STRONG WAY	SEISMIC WEAK WAY
P	394^K	27^K	0
M_{STRONG}	$78'^K$	$38.3'^K$	0
M_{WEAK}	0	0	$58'^K$

RESULTS

	LOADED CAPACITY VERTICAL	LOADED CAPACITY LATERAL
1956 ACI WORKING STRESS METHOD	154.0%	155.0%
1963 ACI WORKING STRESS METHOD	96.4%	102.0%
1963 ACI ULTIMATE STRENGTH DESIGN,		
BRESLER METHOD	80.0%	93.0%
JAKOBSEN METHOD	80.0%	77.5%

Figure 12.

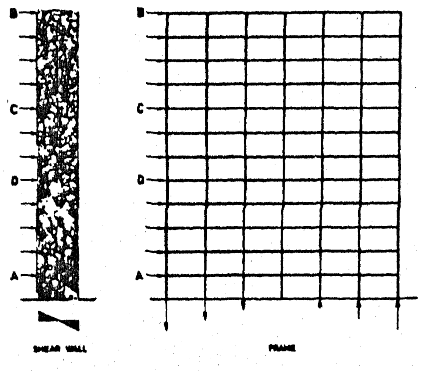


Figure 13.

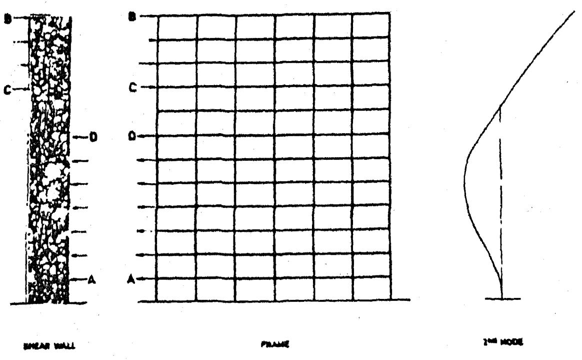


Figure 14.