# REINFORCED CONCRETE SEISMIC DESIGN

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## 1. Editorial Foreword

This paper is reproduced from the proceedings of a seminar on "Seismic Problems in Structural Engineering" arranged by the Departments of Civil Engineering and Extension Studies of the University of Canterbury, and held in Christchurch from May 13 to 16, 1968. Reinforced concrete, as customarily designed and detailed, and in contrast to structural steel, is essentially a brittle construction material. Brittleness can be a danger in regions prone to earthquakes. However, with due care in design and detailing, reinforced concrete structures can be made adequately ductile for good performance in earthquakes. This paper presents a rational design procedure to achieve ductility of reinforced concrete structures.

## 2. Introduction

This subject contains the key words, design, earthquakes and reinforced concrete. Because of lack of time the important subject of philosophy of design must be left in order to deal in a practical way with the problem of reinforced concrete as used for earthquake resistant design.

Since 1956, we have had three world conferences on earthquake engineering, and a valuable textbook, by Blume, Newmark and Corning (almost the only one) has been published on the design of reinforced concrete structures. In spite of its faults, it is a big step forward: using this and other references the well informed designer working in reinforced concrete can - provided he avoids certain unsuitable structural types - predict the general performance of a building in an earthquake with some confidence. Further, if he is prepared to take certain precautions, he can even say that in almost the worst possible earthquake the structure is protected against total collapse. This is in contrast to the bad old days when we designed for static lateral force from the Code, and used code stresses, and then forgot the rest of our earthquake problems.

There are three main developments which make this new confidence possible - firstly we now have some idea of what is the maximum possible earthquake; secondly we know how to compute (in regular structures at least) the maximum values of the forces in the structure provided the structure remains

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elastic; and thirdly the importance of the post-elastic performance of the structure has now been realised.

The object of this lecture is to demonstrate that the responsible designer in reinforced concrete must now direct his design effort from elastic analysis and working stress concepts and towards a thorough understanding of how his structure behaves when loaded beyond its elastic stage. A design method is presented which it is believed will ensure that the post-elastic performance of the structure is adequate notwithstanding our present incomplete understanding of this part of the subject.

### 3. Outline of Elastic Response Concepts

In order to develop a design method it is first necessary to review briefly the concepts of plastic and elasto-plastic response of a structure to the design earthquake.

This can be done conveniently by considering the single degree of freedom oscillator and extending the argument to multi-degree systems.

Fig.6.1 shows the well-known maximum "smoothed" acceleration response experienced by single degree of freedom oscillators of varying natural periods during the passage of the design earthquake. For comparison the "worst" or maximum possible

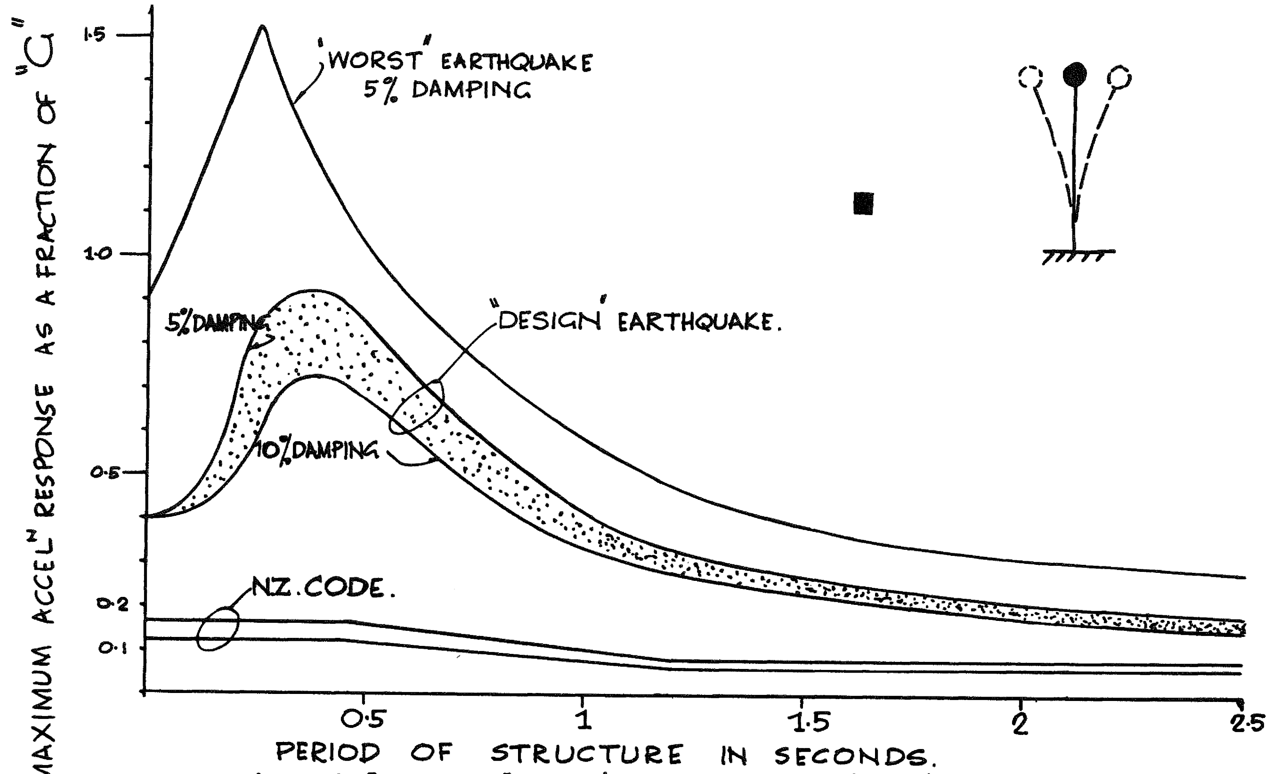


Fig. 6.1 Acceleration Response Spectra.

earthquakes and the New Zealand code requirements are also shown. It is clear that structures which are to remain elastic in the "design" earthquake (especially in the 0.25 to 0.75 period range) must have elastic strengths of the order of 5 to 10 times that required by the code. Yet, in fact, the stresses which are used with the New Zealand Code loads give a margin of only 1.25 to 1.5 before the elastic limits are reached. This discrepancy is usually explained in general terms by saying that the structure both absorbs and dissipates energy and does not behave elastically. It can however be expressed rationally in more precise terms as follows.

## 4. Outline of Elastic-plastic Response

Using again the one degree of freedom system and referring to Fig. 6.2.1. point b represents the maximum response to the earthquake which produces the worst response. Under those conditions the area obd underneath the curve is a measure of the stored potential energy in the structure. As it vibrates from its worst position b back across to zero position o, the energy is converted to kinetic energy, and then back to stored energy at position a. Supposing we now suddenly, by some magic process, instantly transform the stress/strain properties by introducing a plastic hinge so that (referring now to Fig. 6.2.2.) as the structure reaches the plastic hinge moment capacity at point e, instead of carrying on to its full elastic response at b as before it now proceeds along line e-f until brought to rest at f. The velocity energy which existed at o has now been transformed into stored energy as represented by the area oefg (Fig.6.2.2.) while the stresses in the structure have been limited by the formation of the plastic hinge. The amount of rotation required from the plastic hinge is measured by the deflection og and an upper limit for this can be set by the equal energy criterion that the area oefg must be equal to the area obd which measures the kinetic energy at point o. We thus have the nucleus of a design method (known as the reserve energy technique) but before developing it in more detail it is necessary to mention some factors which modify the basic theory.

Firstly in Fig. 6.2.2. of the total stored energy oefg at the position of maximum deflection, only the piece hfg is returned as velocity energy as the structure returns to dead centre. This is in contrast to the elastic system of Fig.6.2.1. where the full stored energy obd is returned as velocity energy on each cycle. The effect of this is that a structure which undergoes a number of elasto-plastic cycles during the passage of the earthquake does not build up as much energy as

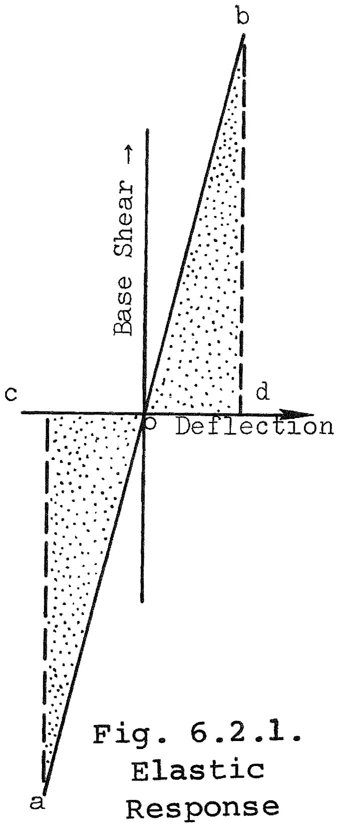



**Fig. 6.2.1.**
**Elastic Response**

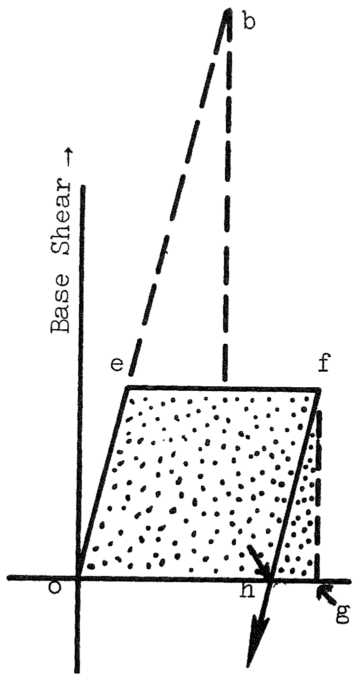



**Fig. 6.2.2.**
**Elasto-Plastic (Equivalent Deflection)**

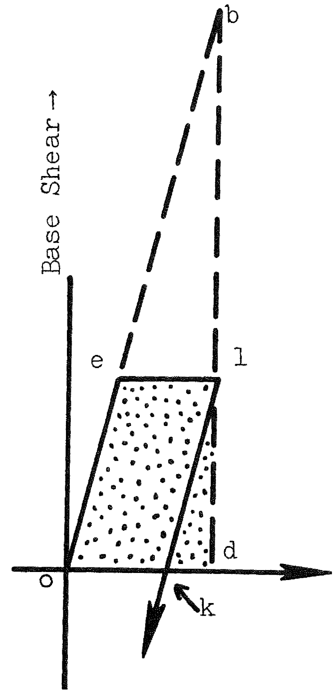
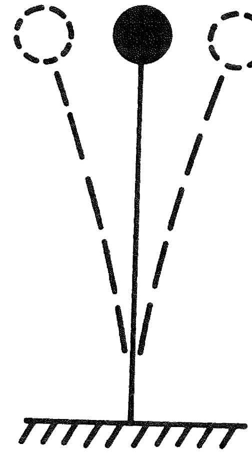



**Fig. 6.2.3.**
**Elasto-Plastic (Equivalent Energy)**



Single Degree of Freedom Resonator.

Fig. 6.2
Comparison of Elastic and Elasto-plastic Responses.

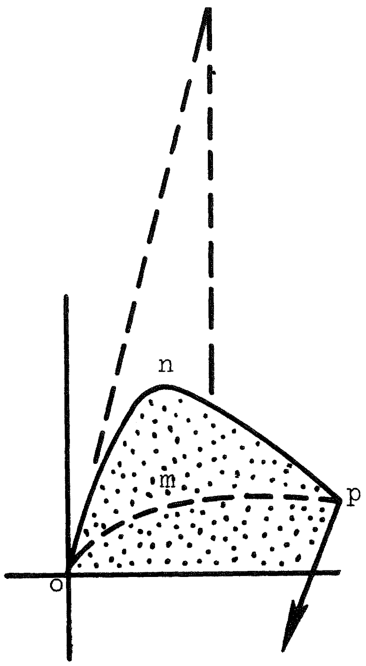



**Fig. 6.2.6.**
**"Softening" Reinforced Concrete**

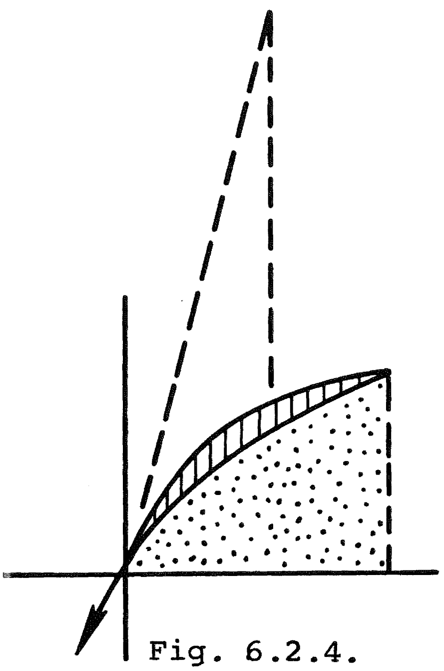


**Fig. 6.2.4.**
**Prestressed Concrete**

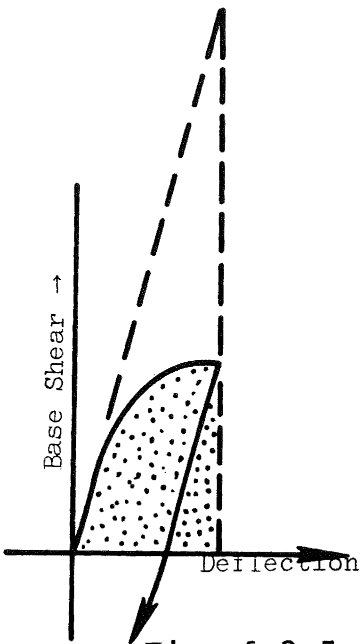



**Fig. 6.2.5.**
**Reinforced Concrete Frame**

a fully elastic one so the equivalent energy concept ( obd equal to oefg) for estimating the plastic hinge rotations tends to be excessively conservative in an elasto-plastic system.

Theoretical analyses of elasto-plastic systems based on real earthquake records have found that the maximum deflections experienced by an elasto-plastic system during the passage of an earthquake tend to be about equal to those of purely elastic systems - or referring to Fig. 6.2.3, the equivalent deflection concept suggests that the energy storage needs are merely the area oeld (od of Fig. 6.2.3. equals od of Fig. 6.2.1.), not oefg as in the equivalent energy concept (Fig. 6.2.2.).

These ideas, although they suggest large differences in the energy storage needs which we should allow for and hence in the plastic rotations called for at hinges, do not affect the principle of our basic design method, rather only the details of its application are affected. For example, referring to real structures for prestressed concrete which tends to have a load deflection curve like that of Fig. 6.2.4. the equal energy rule would be appropriate, whereas for a reinforced concrete frame (Fig. 6.2.5.) one would tend to give more weight to the equal deflection rule.

Certain types of concrete shear wall structures tend to "soften" on repeated loads moving along the curve omp in Figure 6.2.6. Here the use of the equal energy criterion with allowance for the "softening" should be considered.

## 5. Definition of Ductility and Reduction Factors

In Fig. 6.3.a. these two factors are defined in terms of the preceding discussion and by applying the geometry of the energy areas involved we can obtain a direct relationship between these factors both for the equal energy rule (Fig.6.3.b) and for the equal deflection rule (Fig. 6.3.c). The fact that for the equal deflection rule the ductility factor is numerically equal to the reduction factor has led to much confusion in the use of these terms. It is essential to remember that the reduction factor is a ratio of loads whereas the ductility factor is a ratio of deflections.

## 6. The Development of a Design Method

Our argument so far has demonstrated that because the purely elastic response of a structure is 5 to 10 times that required by the code (Fig. 6.1) designing at code levels can

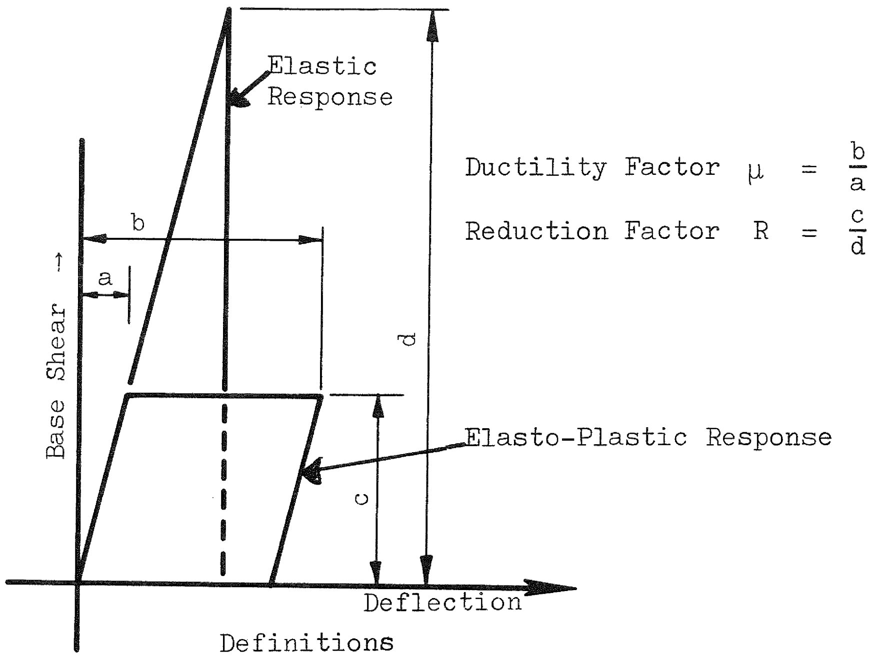
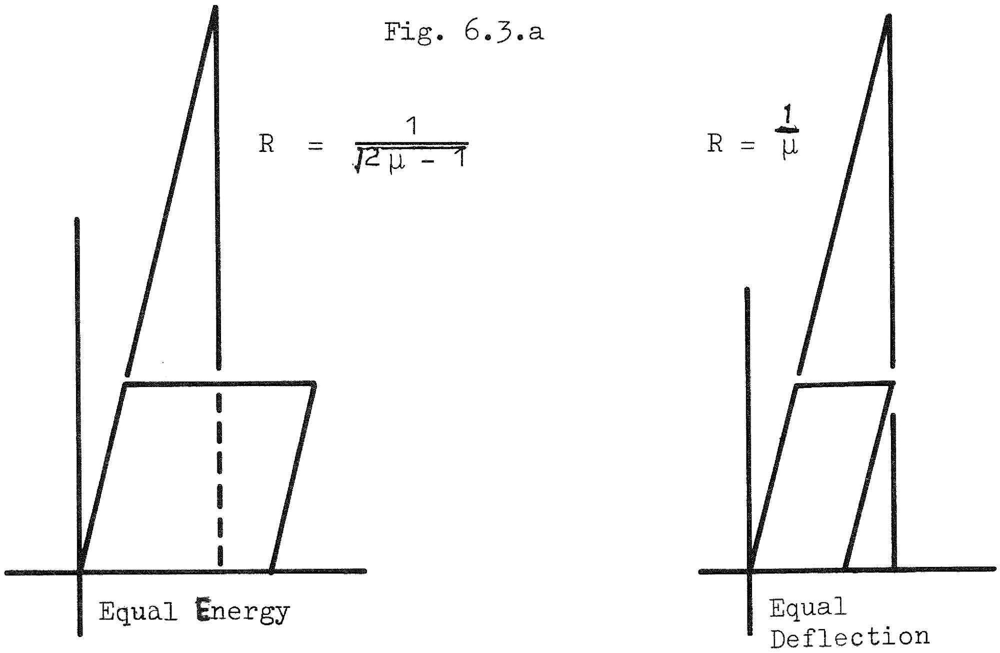



Fig. 6.3.a




Relationship Between "$\mu$" and "R".

Fig. 6.3.b Fig. 6.3.c

Fig. 6.3 Definitions and Relationship Between "$\mu$" and "R".

only be justified by ensuring adequate post elastic performance of the structure. Now in contrast to steel which as a ductile material will produce a ductile structure with only moderate care from the designer, reinforced concrete as normally detailed is an essentially brittle material. It is brittle in compression, in shear, in some forms of bending and even in tension if imperfectly spliced. One might say it is essentially glass-like in character, whereas for good earthquake performance we want a structure of lead-like character. Now it would be feasible to build a structure of glass with lead hinges which would perform satisfactorily in an earthquake and this is the mental image we should bear in mind in designing reinforced concrete for earthquakes - we are using our design skill to transform an essentially brittle or glass-like material to an essentially ductile or lead-like structure.

It should be clear that this object is not going to be achieved by haphazard methods - we must have a systematic approach.

RESERVE ENERGY

PHASE ONE - STRENGTH

Step 1. From preliminary design select tentative sizes.

Step 2. Estimate period and mode properties.

Step 3. Use response spectrum to obtain the elastic response.

Step 4. Compute the required reduction factor.

PHASE TWO - DUCTILITY

Step 1. From reduction factor compute ductility factor using

$$R = \frac{1}{\mu} \quad \text{or} \quad R = \frac{1}{\sqrt{2\mu - 1}}$$

Step 2. Compute ductile deflection as $\mu$ x elastic deflection.

Step 3. Find position of plastic hinges.

Step 4. Compute hinge rotations.

Step 5. Design hinges.

Step 6. Design structure between hinges.

Step 7. Check stability.

Step 8. Check secondary damage.

Fig. 6.4 Reserve Energy Steps - Phase I and Phase II.

The design method set out in Fig. 6.4 is therefore offered as a rational process which if applied with care will in spite of the gaps in our knowledge give us a sound structure. It is not limited to frames but can be applied to shear walls, pre-cast structures, prestressed structures or any combination of these types.

It is now proposed to go through this design method step by step - familiar operations will be touched on only lightly and more time given to discussion of the vital steps of phase two will give the reinforced concrete structure its vital ductility.

Phase I - Steps 1 to 4

Phase I, Steps 1,2,3 are part of the familiar elastic design process. It will be noted that Phase I is concerned with strength: Phase II with ductility. The amount of emphasis laid on one phase or the other depends on the value we select for the reduction factor (Phase I, Step 4). Usually, however, if we are working to the code this will have values in the range 2 to 6 and Phase II becomes of vital importance.

Phase II - Step 1 (Fig. 6.4)

Step 1 (Phase II) follows from the definitions of paragraph ("Definition of Ductility and Reduction Factors") above.

Phase II - Steps 2,3 and 4 (Fig. 6.4)

While Steps 2,3 and 4 are elementary for the single degree of freedom systems examined in the paragraph above on "Outline of Elasto-plastic Response", in extending the argument to multi-degree of freedom systems we find immediately that there is no generalised solution for Steps 2,3 and 4 short of a full scale elasto-plastic analysis in which the structure is examined at short time intervals through the passage of the earthquake. At the present state of the art this technique is available on a practical basis for frame type structures only and is really best applied as a checking device. Nevertheless, the finding of the plastic hinge positions is an essential step in converting our brittle glass-like concrete structure to a ductile lead-like one.

The only remaining course then is to adapt our structural layout so that we can reliably predict the positions where hinges will form notwithstanding the difficulties inherent in multi-storey structures. Experienced structural designers will not have great difficulty in finding suitable structural layouts for this purpose and three examples are given as illustrations.

Example 1 (Fig. 6.5 )

If the beams of a frame structure are made very much stiffer than the columns as in Fig. 6.5, an analysis soon shows that the only prudent design course is to allow for all the plastic hinges to be formed across the columns of one storey as in Fig. 6.5.c.

This is because:

(i) It is impracticable in detailing to accurately match the column capacities storey by storey to an elastic response to a given earthquake.

(ii) Even if this were done for one earthquake type it would not suit another.

(iii) It is not practicable to design column hinges with a strong rising or strain-hardening trend in the moment-rotation curve rather than the normal horizontal trend as in Fig. 6.5.a. If this could be done additional hinges would be forced to occur in stories above and below the hinging storey as rotations develop.

Thus, if a structure with a powerful beam is selected, we can design for hinges across a selected storey only and Steps 2,3, and 4 of Phase II (Fig. 6.4) follow without difficulty.

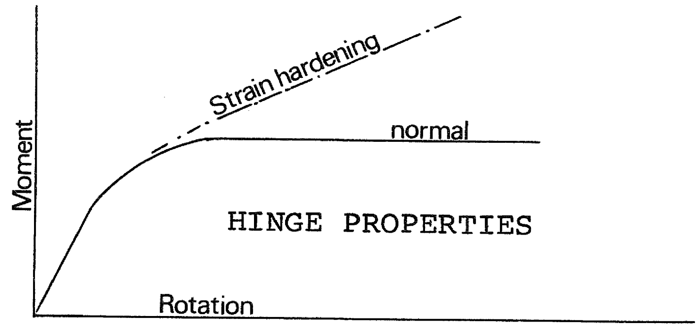



Fig. 6.5.a

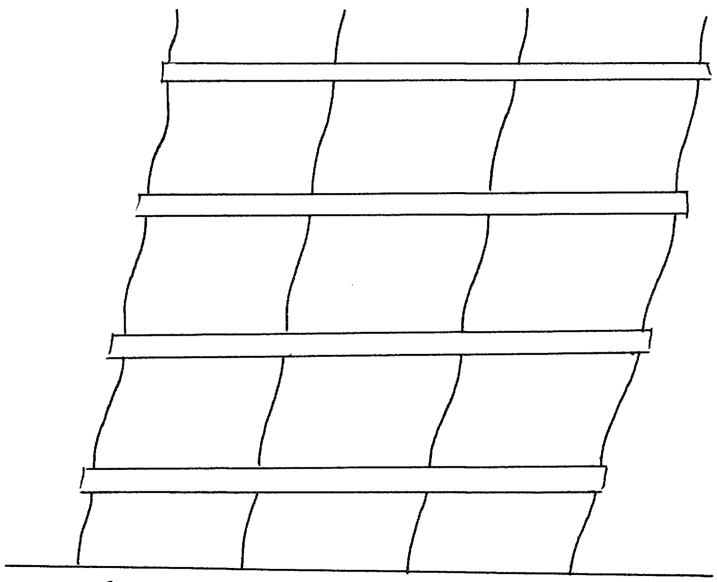


Fig. 6.5.b Elastic Deflection

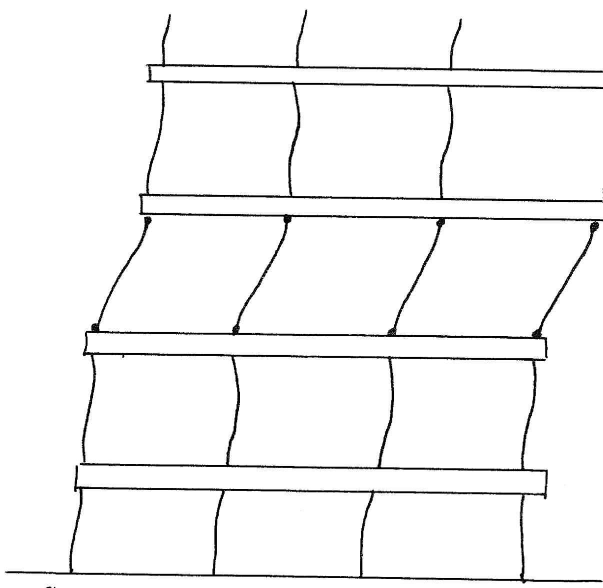


Fig. 6.5.c Formation of Hinges

Fig. 6.5 Finding Hinges in Multi Mass Structures.

A word of warning is necessary, however - this type is unsuitable for buildings more than a very few stories in height because:

(i) The concentration of energy absorption in a few members demands very high rotations at the hinges - more in fact than column hinges in reinforced concrete can be relied upon to give.

(ii) These high rotations in primary compression members produce an instability risk for the structure as a whole.

(iii) In tall structures the column strengths naturally dominate the beam strengths and to attempt to reverse this trend is nearly always uneconomic.

(iv) The possibility of permanent deflections and the difficulties of structural repair may both be greater after the earthquake than for any other types.

Example 2 (Fig. 6.6)

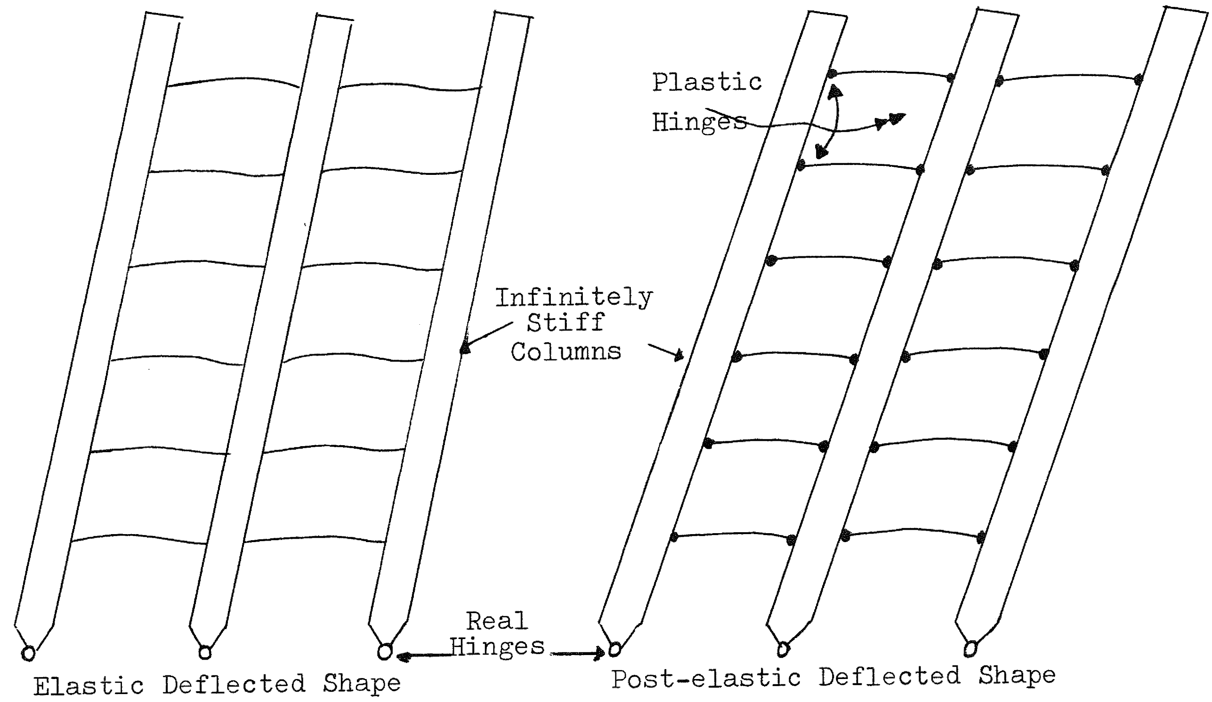



Fig. 6.6 Finding Hinges in Multi Mass Structures.

At the other extreme to Example 1, if we make the columns of a frame structure infinitely stiff then the plastic hinges are forced to occur in every beam end simultaneously as in Fig. 6.6. Again the Steps 2,3,4 of Phase II become straightforward for the set of assumptions. The advantage of this type of hinge formation is, in contrast to Example 1, that because of the number of hinges the rotation required from each is small, and this means -

(a) Small total deflection at maximum plastic response;

(b) Small risk of instability;

(c) Small permanent deflections;

(d) Excellent resistance to the ultimate earthquake because beam hinge rotations can be made very large;

(e) Structural repairs easier after earthquake because beam damage likely to be slight and each beam (unlike a column) is not supporting a large volume of structure.

This is clearly an excellent structural type but the hitch is that infinitely stiff columns are inconvenient.

If as a practical step we make the columns just stronger than the beams (say of factor of 1.25) at each and every joint, it might be supposed that we have achieved the same effect. (This is convenient because columns tend this way due to code load factoring methods. Note that the current SEAOC code also has this as a requirement.)

There is a danger here though that our experience of elasto-plastic design is insufficient to justify such a short cut as a reliable solution for important structures. Might not an earthquake which, for example, stimulated the building's second mode, cause column hinges?

We must conclude that because of the glass-like nature of our material, major structures designed on this basis should have a full elasto-plastic analysis as a final check - for less important structures for which such an analysis is not justified, yet for which we must nevertheless still ensure our lead-like - not glass-like performance, it is necessary to seek other structural types; e.g. that of Example 3.

If we introduce into a frame structure a strong vertical element of great stiffness compared with the generality of the columns (see Fig. 6.7), we have all the advantages of the Example 2 type without the disadvantages.

It is true that carrying out the Phase II Steps 2,3,4 is made a little more difficult than the infinitely stiff column case because the beam hinges tend to form first at the top of the structure and progress downwards: this is not a great difficulty however especially if there is only minor base restraint at the base of the stiff element. The possibility of hinges forming in the external columns can be completely discarded because the stiff element can be easily designed to elastically absorb all second or higher mode shears (Fig. 6.7) which could induce such hinges.

Example 3 - Fig. 6.7

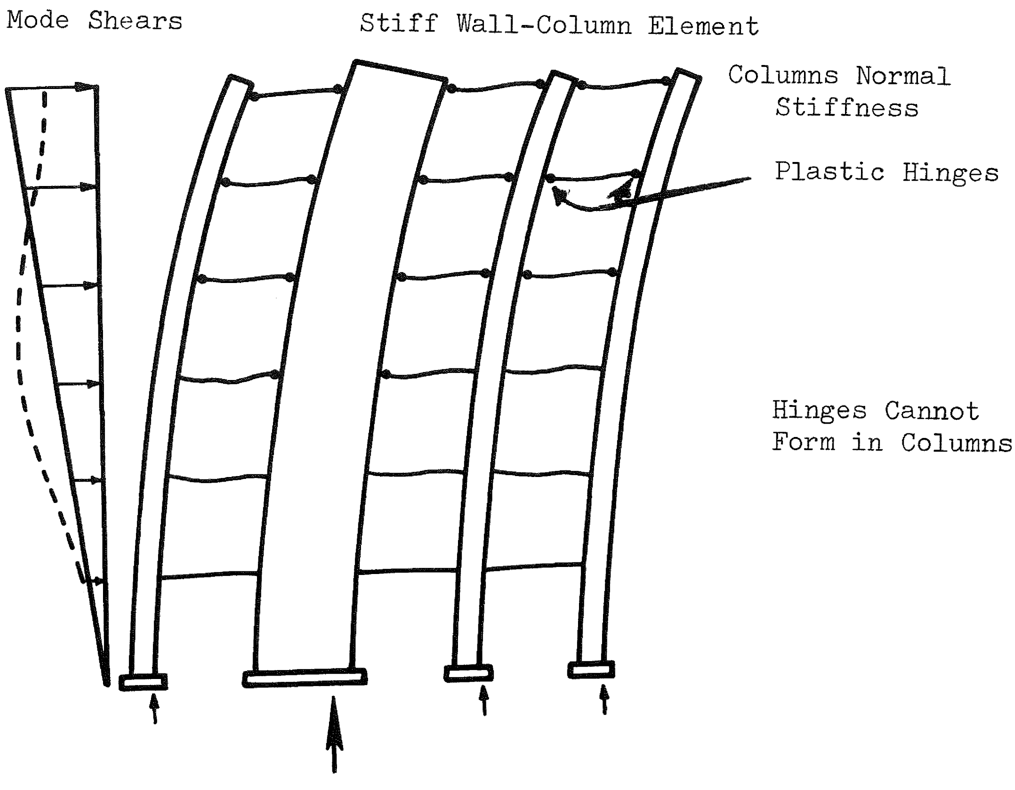



Fig. 6.7
Structural Type to
Force Beam Hinges.

We can conclude therefore that the problem of imposing on the structure a deflection sufficient to absorb the required energy (Step 2), of finding where the hinges will form and what these rotations are (Steps 3 and 4) can be met even for multi-storey structures provided we adjust the structural type to force the hinges to occur where we want them.

Design of Plastic Hinges (Fig. 6.8)

Plastic rotation (Concrete governs) =

$$\frac{(\text{Max. safe concrete strain} - \text{Elastic concrete strain}) \times \text{Plastic length}}{\text{Distance of neutral axis from compr}^{n}\text{ fibre}}$$

Fig. 6.8.a

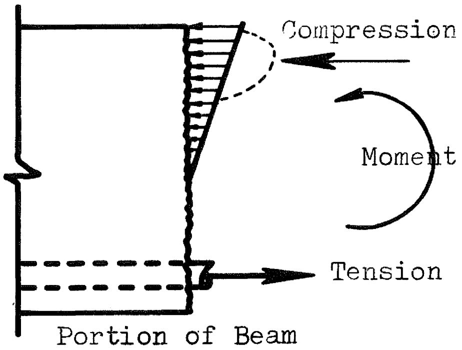



Fig. 6.8.b

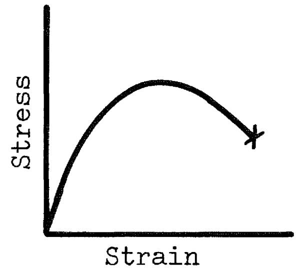



Fig. 6.8.c

$$\text{Plastic Rotation (Steel governs)} = \frac{\text{Max}^{m} \text{ safe steel strain - Elastic steel strain}}{\text{Distance of neutral axis from tension steel}} \times \text{Plastic length}$$

Fig. 6.8.d

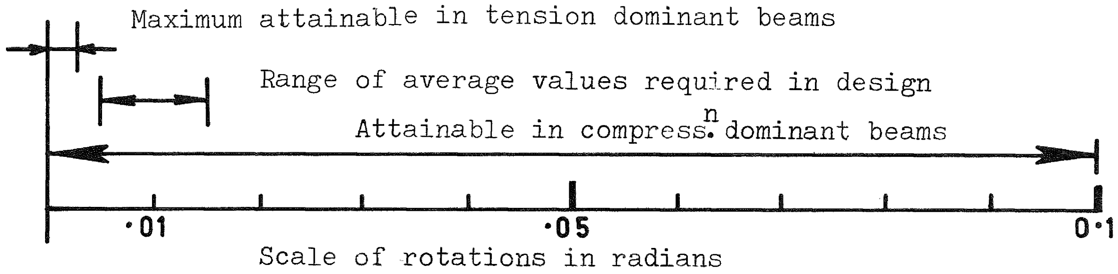



Fig. 6.8.e

Fig. 6.8 Rotations at Plastic Hinges

Having ascertained those positions in our structure where plastic hinges will be required to form and having also estimated the total rotation required from the hinges, we can now proceed to Step 4 Phase II - the design of the hinge itself.

The basic criterion is that the hinge used should be capable of sustaining the imposed rotations through several reversals without loss of structural integrity. This means in a beam hinge without loss of shear capacity and in a column hinge without loss of either shear capacity or axial load capacity.

To decide how to achieve this aim we must review the available hinge design data - to keep the problem simple this discussion will be limited to beam hinges.

In Fig. 6.8.a is given the now well known equation for one cycle plastic rotation in a beam hinge. Technical discussion of this equation has centred around the supposed need to obtain high safe concrete strains in order to obtain high rotations. For visualising the hinge performance however, it is more convenient to think in terms of the internal forces in the beam at the hinge as set out in Fig. 6.8.b.

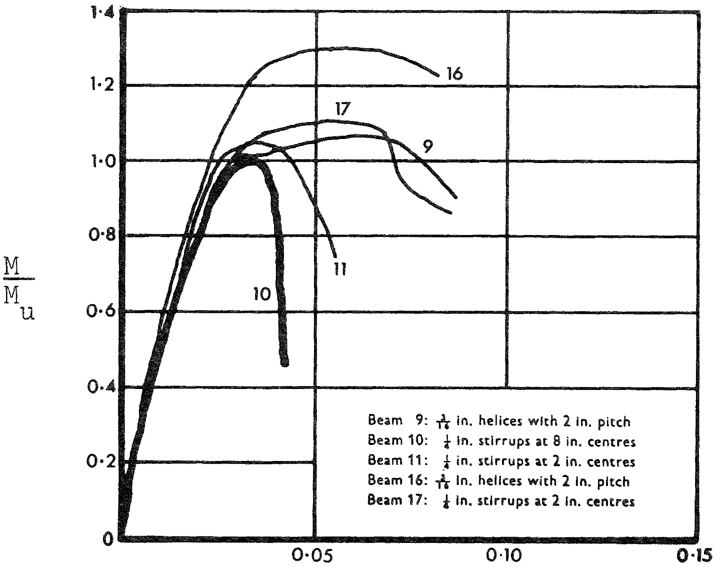
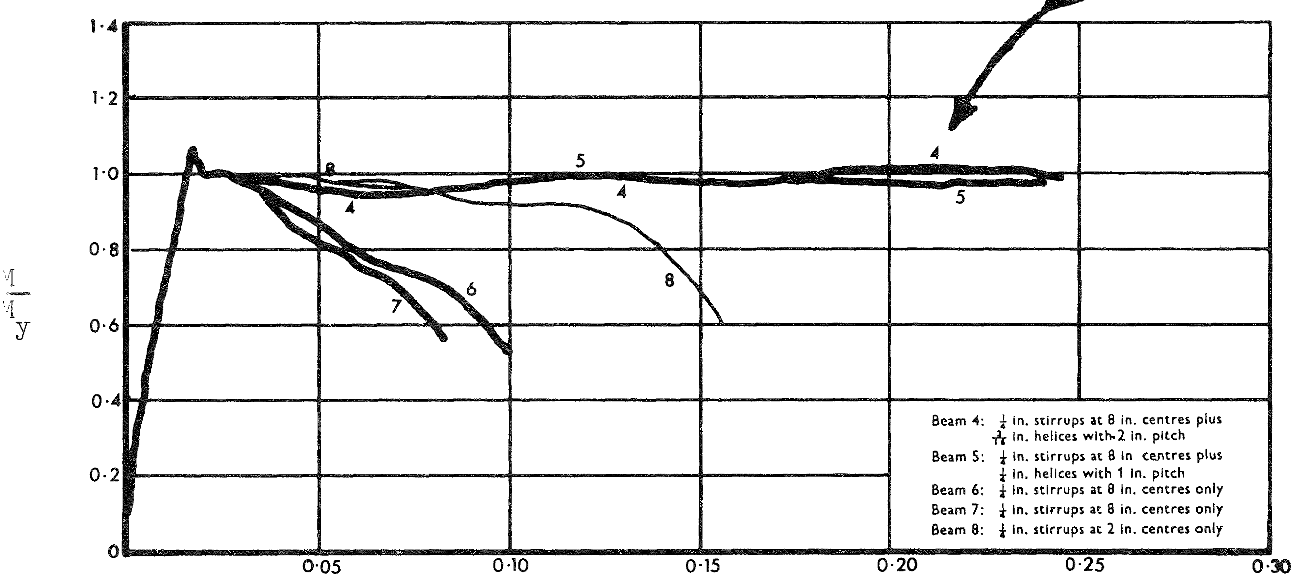



Fig. 6.8.f

Test Results on Beam Hinges.

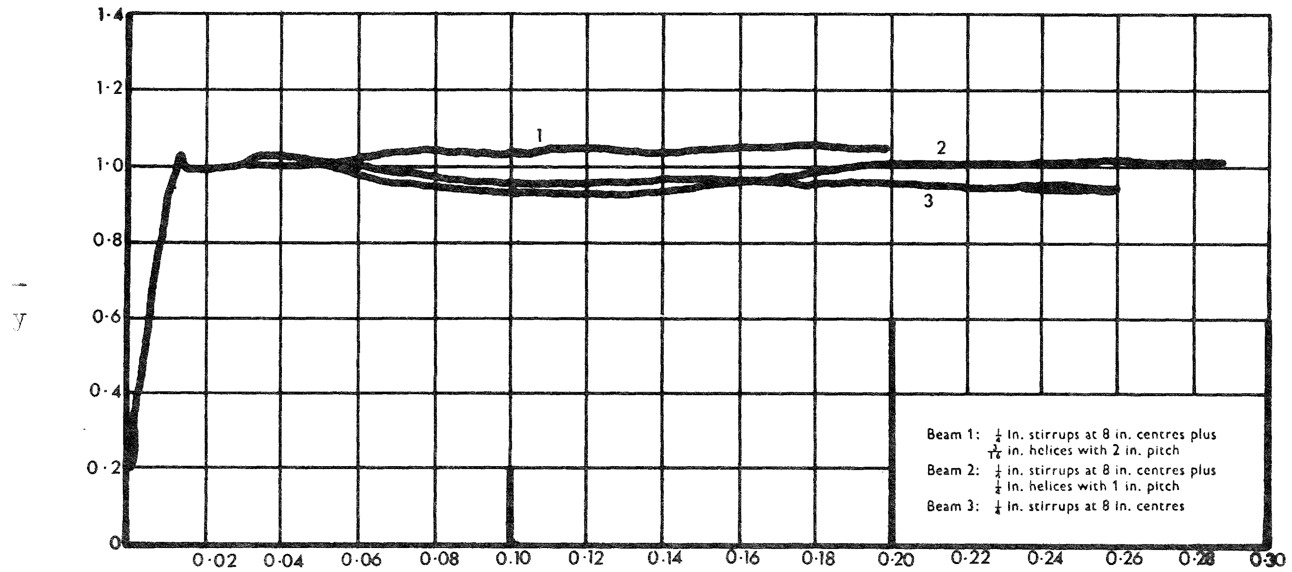
Total Rotation Between Beam Supports - rad.

TENSION ZONE DOMINANT




Total Rotation Between Support Points - rad.

INTERMEDIATE CASE




Total Rotation Between Support Points - rad.

COMPRESSION ZONE DOMINANT

If for example we have a beam in which the tension capacity greatly exceeds the compression capacity (called over-reinforced or tension zone dominant), Fig. 6.8.a applies and rotation is limited by the concrete strain. Moreover rotation at the hinge tends to occur about the tension zone as a fulcrum (low neutral axis) so to obtain rotation the whole of the concrete stress block must be strained. Because of the falling trend of the concrete stress-strain curve (Fig.6.8.c), concrete "strain hardening" at the hinge does not occur so the hinge cannot spread along the beam and total rotation is limited by a short plastic length also.

If, on the other hand, the compression zone of the beam is somehow made very much stronger than the tension zone, e.g. by providing both compression reinforcing and binding on the compression side, (called a compression zone dominant beam) then the equation of Fig. 6.8.d rather than that of Fig. 6.8.a applies and the rotation available is limited only by the safe strain available from the tension steel. To give some sense of proportion to these opposing concepts Fig. 6.8.e drawn to scale shows the tremendous difference in rotational capacity available if calculations are made from equations Fig. 6.8.a or Fig. 6.8.d Fig. 6.8.f shows the results of some actual tests made on full size beams.

Referring again to the equation of Fig. 6.8.a we see that we have been so far discussing only the first term of the right-hand side but the total rotation available is also influenced by the second term, i.e. the plastic length. Fig. 6.9.a shows how the plastic curvature is distributed in the plastic hinge of a beam and in Fig. 6.9.b an empirical (Professor Baker) expression is plotted for obtaining the plastic length available in a given beam. It is seen that when the beam half length over depth ratio is less than two, the plastic length is reduced to half the beam depth (Fig. 6.9.c) and this is suggested as a minimum condition especially when combined with the use of high tensile steels.

All of these rules so far for calculating plastic hinge rotations are for <u>one cycle</u> loadings only.

If now we look at specimens designed to these rules after the specimens have received a one cycle test, an immediate doubt is raised about the number of further cycles such specimens will stand while still retaining their shear capacity. Figs. 6.10 and 6.11 give examples which are, admittedly, very severe ones since the rotations are imposed at or above the extreme limits which could possibly be needed in design.

$$\text{Plastic rotation} = \frac{\text{Strain difference}}{\text{Depth to N.A.}} \times \text{Plastic length}$$

$$\text{Plastic length} = K_1\ K_2\ K_3\ \left(\frac{z}{d}\right)^{\frac{1}{4}}\ d$$

$$\text{Or in beams} = 0.5\ d\ \left(\frac{z}{d}\right)^{\frac{1}{4}}$$

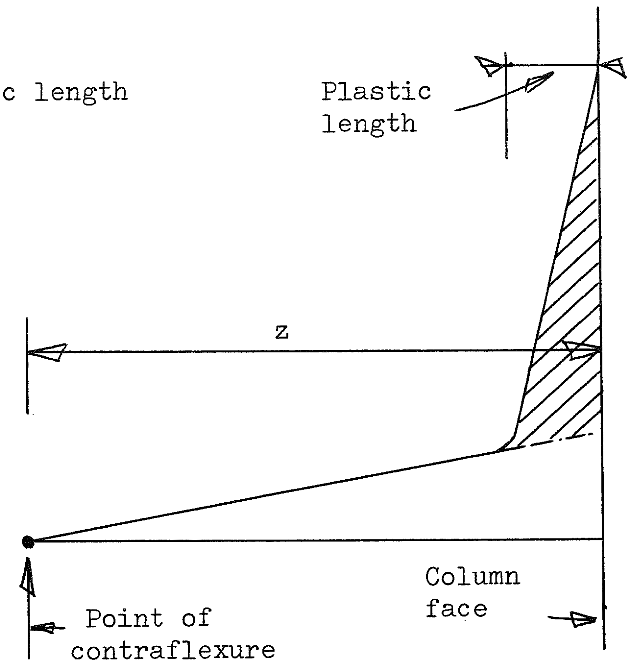



Fig. 6.9.a Curvature Distribution in Beams.

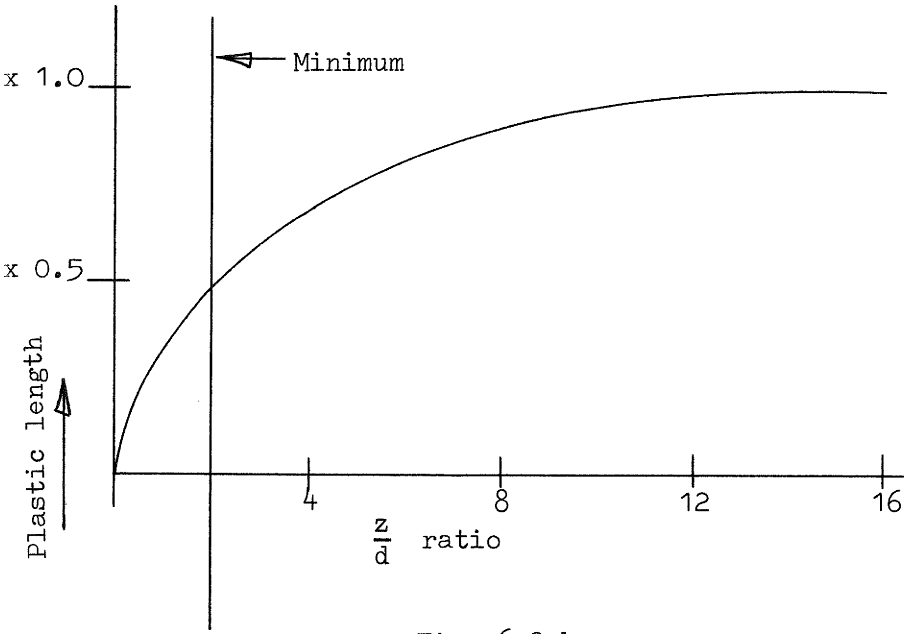



Fig. 6.9.b

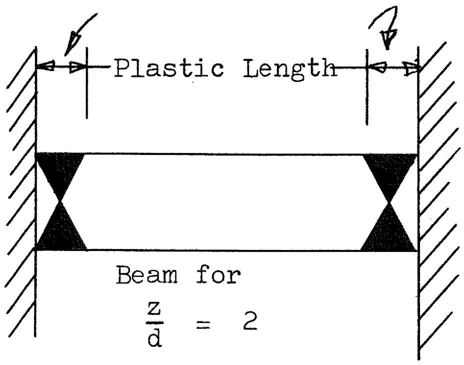



Fig. 6.9.c

Fig. 6.9 Discussion of Plastic Length in Beams.

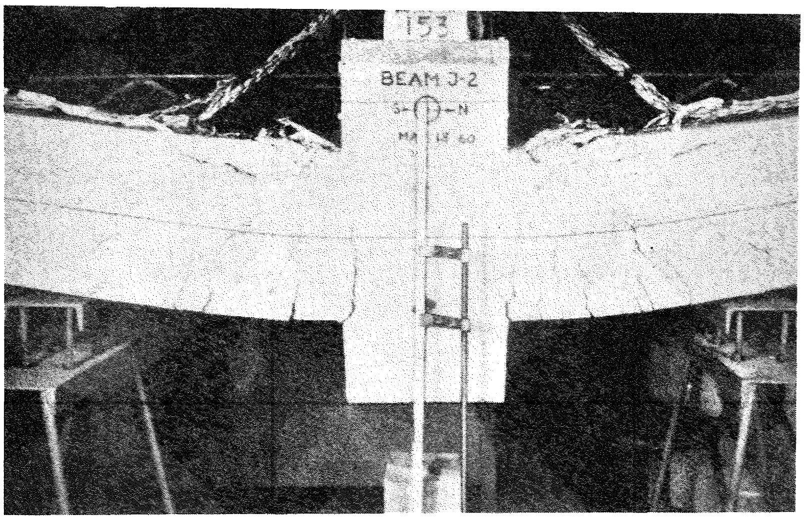



Fig. 6.10 View of Beam Hinge at 0.12 Radians Plastic Rotation Approximately.

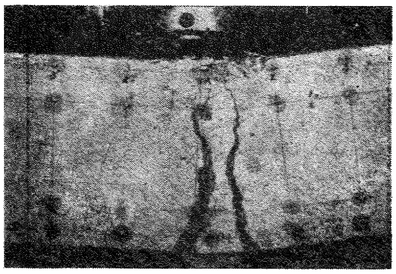


Fig. 6.11 Plastic Hinge.

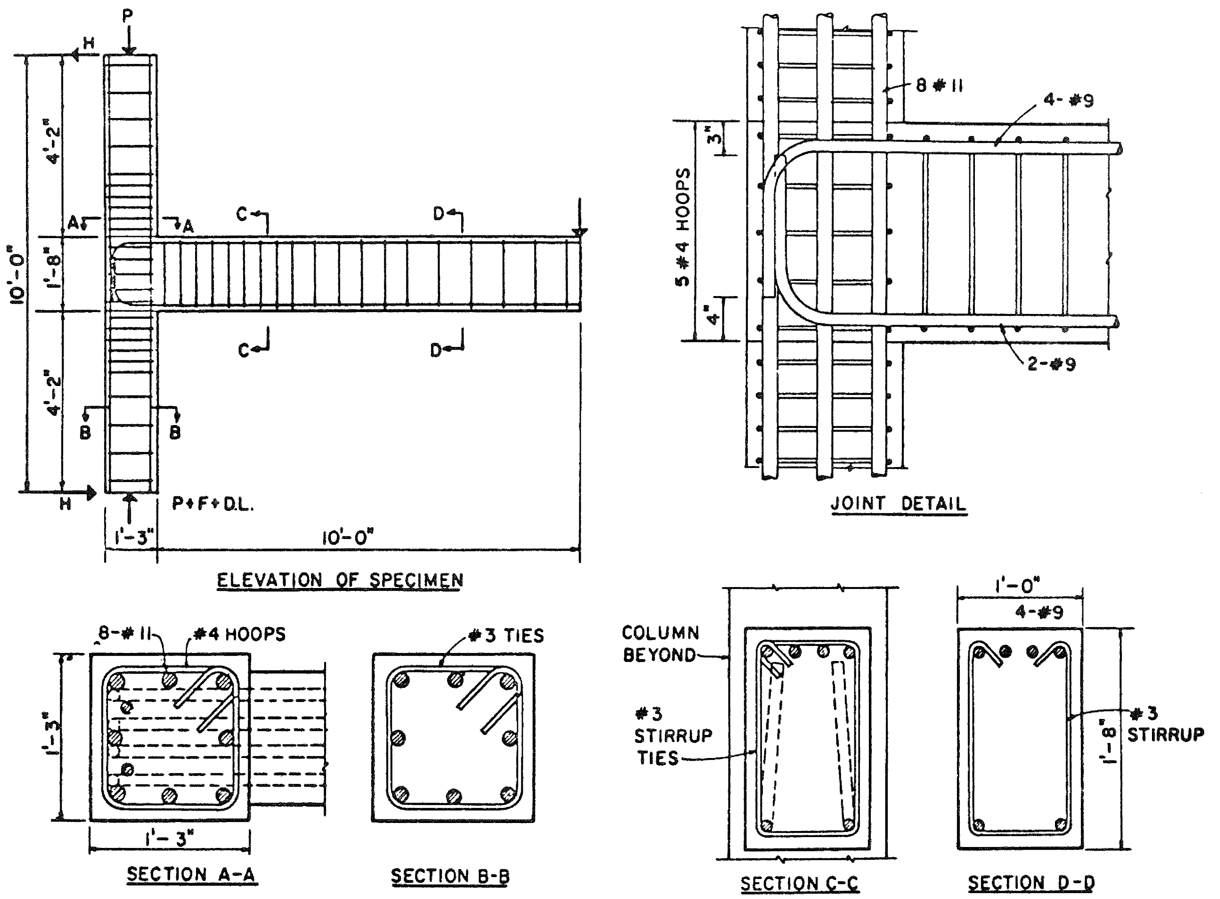



Fig. 6.12 Portland Cement Association Test Piece Details.

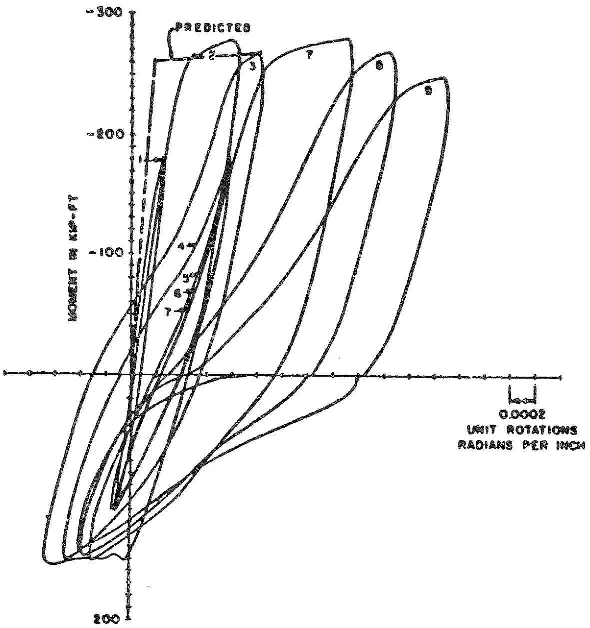



Fig. 6.13 Portland Cement Association Test Results.

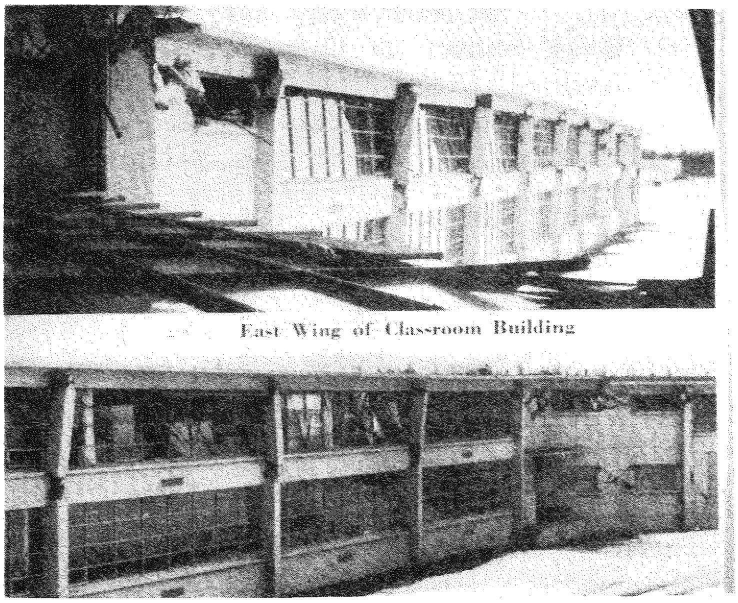



Fig. 6.15 Example of Beam-Hinge Failure from Actual Earthquake.

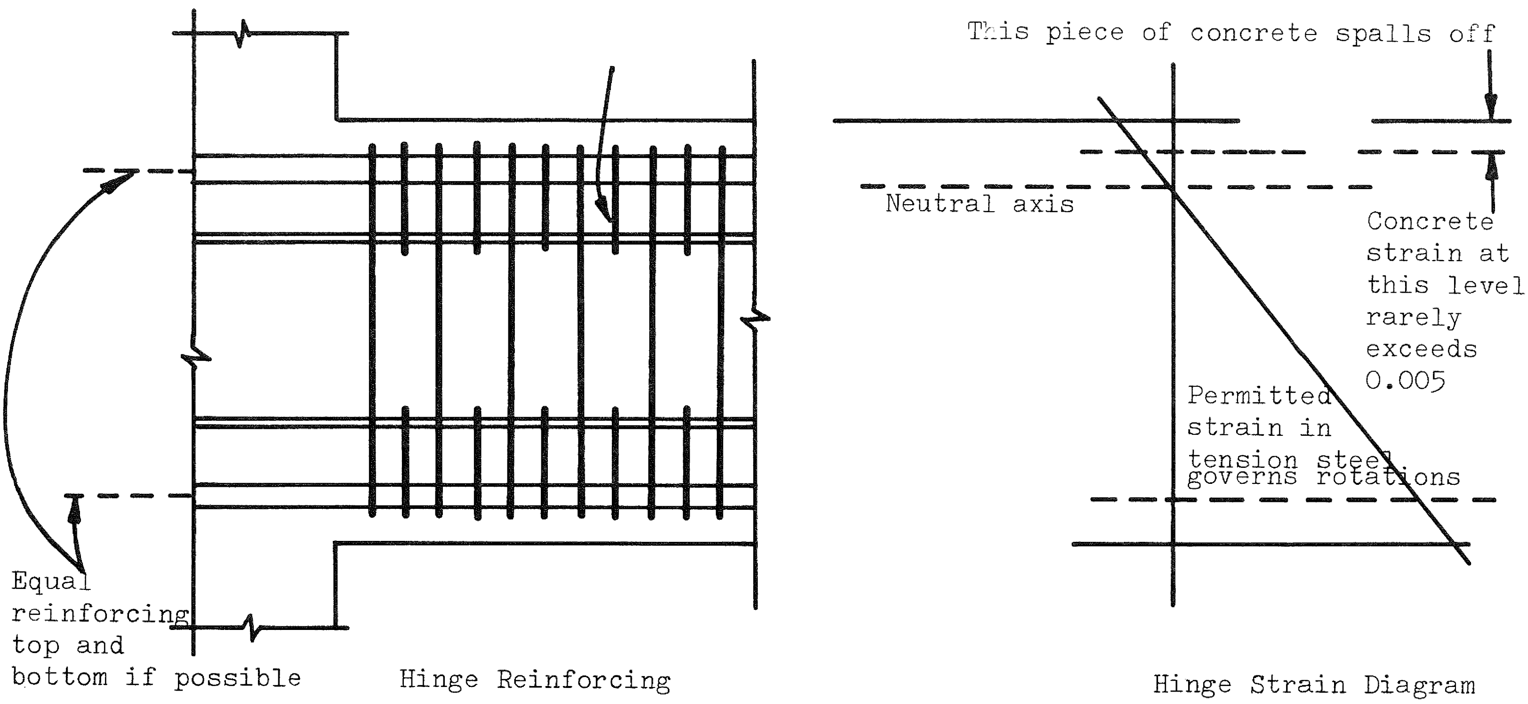



Fig. 6.14

Of the very few tests using repeated cyclic loading the best known is the series carried out by the Portland Cement Association (U.S.A.) on specimens of the type shown in Fig. 6.12 with results as shown in Fig. 6.13. This is not quite the arduous test the authors would have us believe since -

(i) The plastic rotation imposed during each cycle was in the range 0.005 to 0.01 radians which is modest;

(ii) The span depth ratio of the beam is large which both makes it easier to get the required rotation (refer Fig. 6.9) and also means that the shear on the hinge section was light (only about 100 p.s.i. at full moment capacity).

All this leads us to conclude that in the design of hinges in important structures for the dual aim of hinge integrity and high rotations we should be quite cautious in our approach. Further, because the volume of material even in an over-designed hinge is quite small in relation to the volume in the total structure, the extra cost of caution is very low.

Suggested design principles are therefore:

(i) Only fully compression dominant hinges are satisfactory; this may be achieved by providing equal steel top and bottom and by also providing binding in the compression zone.

(ii) The full shear through the hinge should be provided for with steel reinforcing without reliance on the concrete.

This is illustrated in Fig. 6.14 and it should be noted that the very high neutral axis which goes with compression dominant beams means that only a very small volume of concrete must be restrained by binding so that the volume of binding steel is also very small. The loss of concrete cover must be expected in these hinge designs and this can be allowed for in computing the ultimate moment if desired.

All of the above discussion on beam hinges is theoretical but many examples of failed beam hinges can be found from pictures of actual earthquake damage. Figs. 6.15 and Fig. 6.16 are typical of poorly designed (or not designed) beam hinges from the Alaskan earthquake. Note that although these hinges are in vertical or column members, the level of vertical load is so small (less than 150 p.s.i.) that they must be classified as beam hinges.

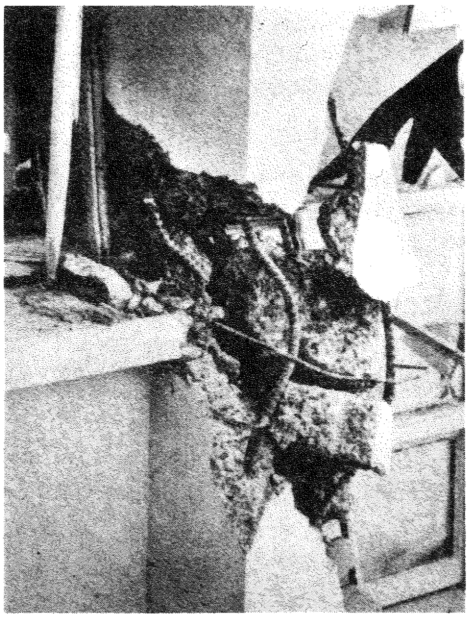


Fig. 6.16 Detail from Fig.6.15.

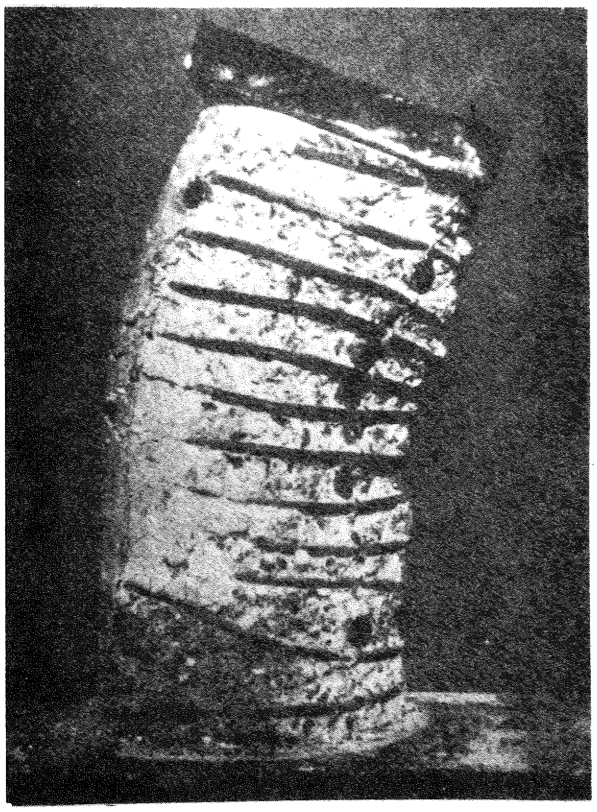


Fig. 6.17 Laboratory Example of a Column Hinge.

Fig. 6.18 Example from Alaskan Earthquake of a Column Hinge.

Fig. 6.19 Example of a Failed Column Hinge.

Fig. 6.20 Example of a Failed Column Hinge.

So much for the design of heam hinges - space does not allow a full discussion of column hinges (defined as hinges in members carrying high vertical stresses) but we can observe in passing -

(i) We should avoid introducing column hinges as far as possible in multi-storey structures;

(ii) In column members of moderate dimensions they can be designed to perform adequately as is indicated in Figs. 6.17 and 6.18;

(iii) In examining records of earthquake damage to framed structures one finds that column hinge failures are among the commonest damage these structures experience (Figs. 6.19 and 6.20).

Design of Structure Between Hinges - Step 6, Fig. 6.4

Having located the position of the plastic hinges and designed the hinges themselves we are now in a position to design the structure between the hinges (Phase II Step 6, Fig. 6.4).

In carrying out Step 6 some important facts must be borne in mind. These are:

(i) Once the positions of the hinges have been fixed and the magnitudes of their ultimate moment capacities fixed, the structure becomes statically determinate between hinges.

(ii) Further, once all hinges have formed they act as a system of fuses, such that no matter how much greater the earthquake may become the internal forces in the structure cannot increase: instead the additional earthquake energy is disposed of by further rotations at the hinges; i.e. by bigger structure deflections.

(iii) Thus, if we apply a suitable load factor (whose magnitude must vary with the accuracy of our analysis) to the hinge moments we can then in designing the elements between the hinges guarantee that their strength cannot be exceeded even in the greatest earthquake. This gives us a certain method for converting even brittle materials into a structure which is ductile overall (glass-like material, lead-like performance).

In carrying out Step 6 also, experience has shown the need for a systematic first principle approach in which the forces in each element of the structure are checked through by a logical

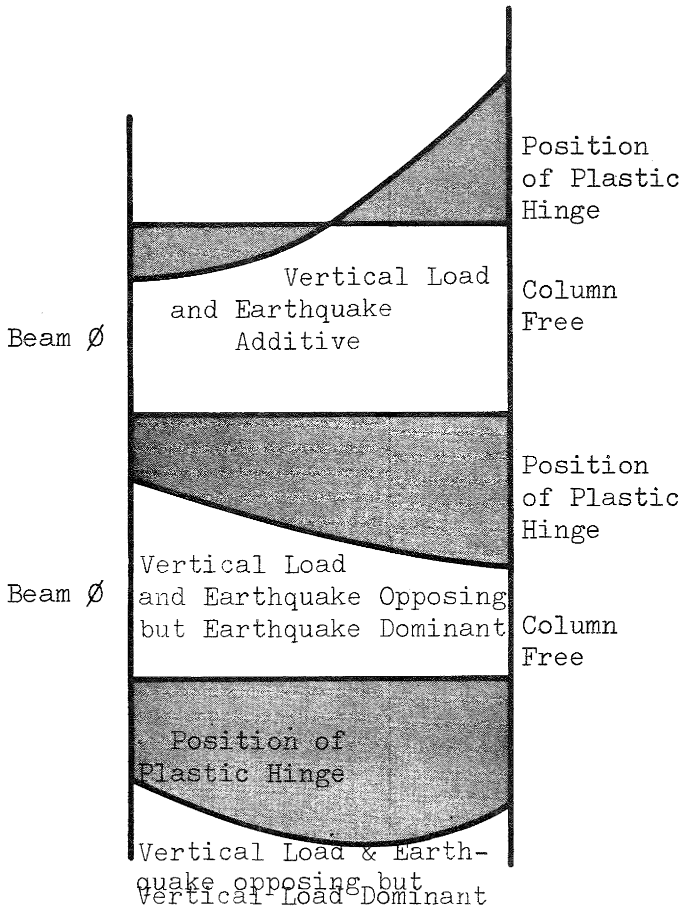



Fig. 6.21 Bending Moment Diagrams Showing that Plastic Hinges are not Necessarily at Beam Ends.

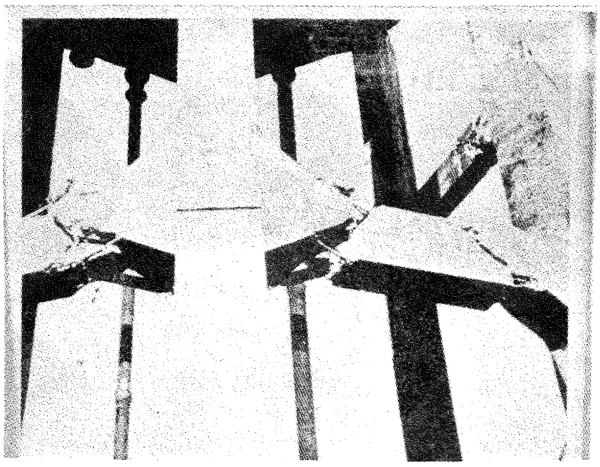


Fig. 6.22.a Shear Failure Detail from Fig. 6.22.

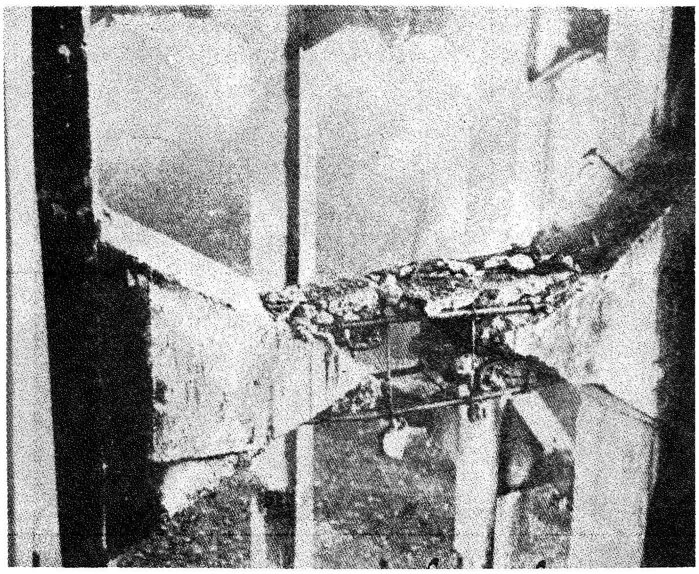


Fig. 6.22.b Shear Failure Detail from Fig. 6.22.

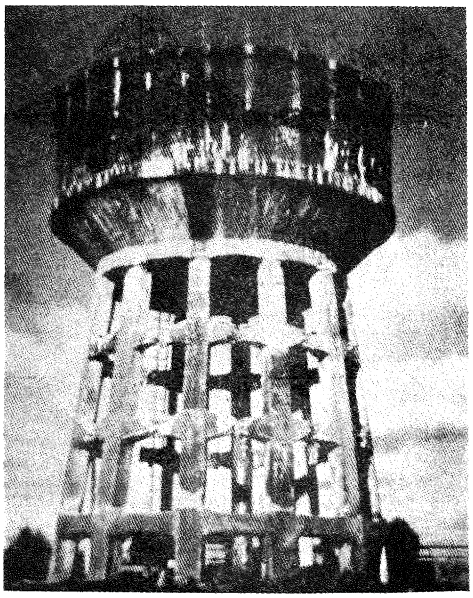


Fig. 6.22 Water Tank After Earthquake.

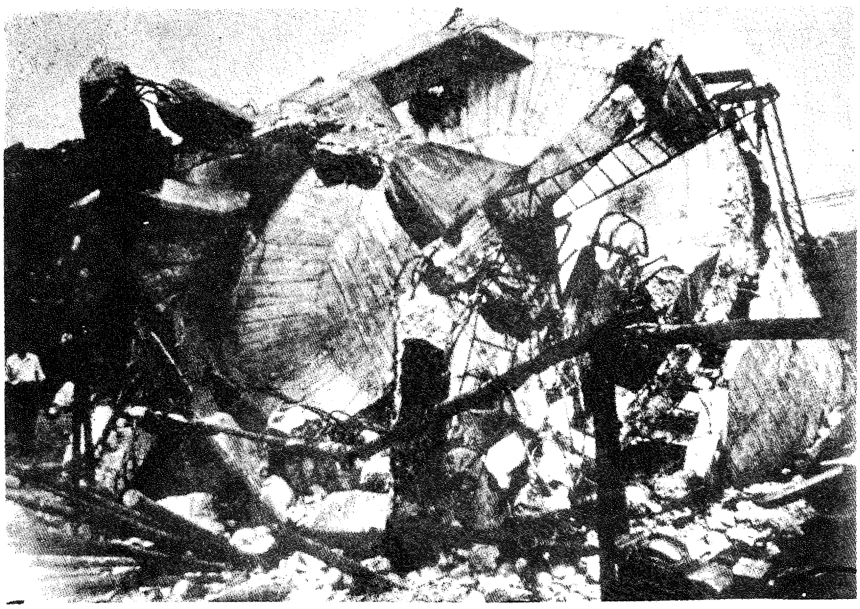
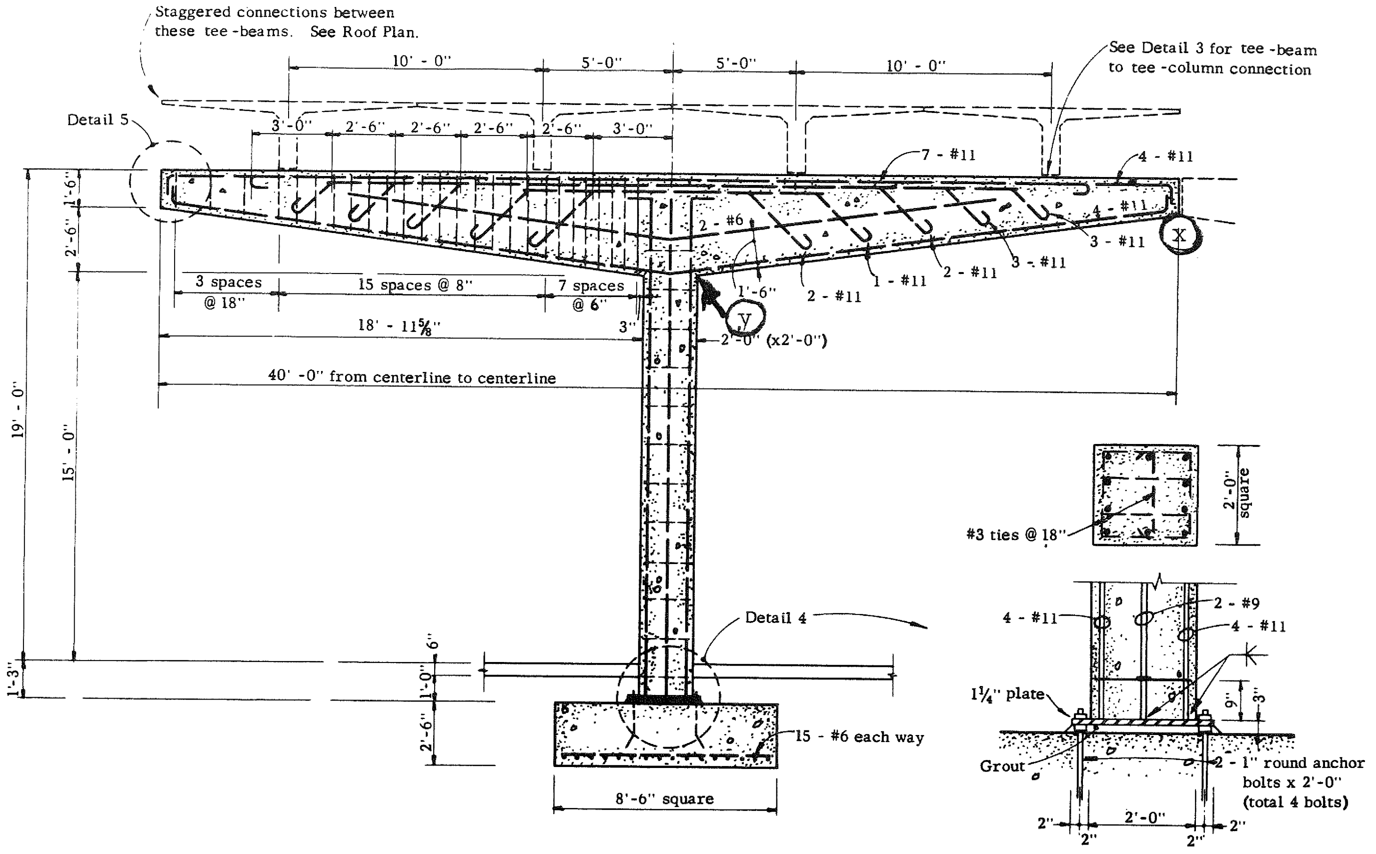


Fig. 6.22.c Collapsed Tank.



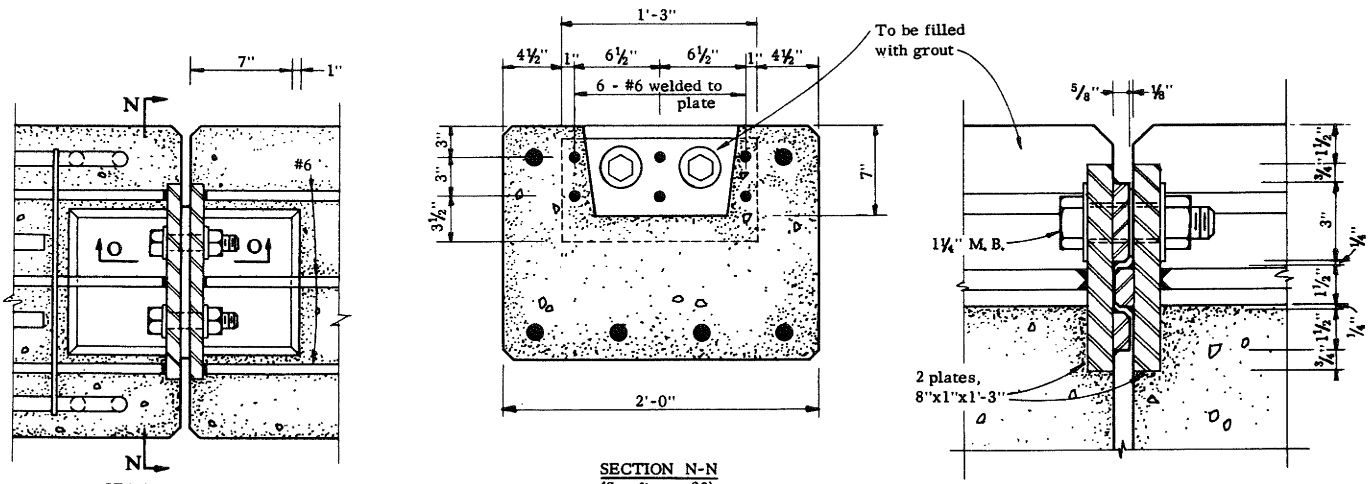

SECTION A-A

TYPICAL TEE COLUMN

DETAIL 4

ANCHORAGE OF TEE COLUMN TO FOUNDATION

Fig. 6.23.a




SECTION M-M
PLAN

SECTION N-N
(See figure 93)

SECTION O-O

DETAIL 5

CONNECTION BETWEEN TEE COLUMN ARMS

Fig. 6.23.b

Fig. 6.23 Example of Failure Between Hinges from Actual Earthquake - Connections of Pre-cast Members.

process. If this is not done - if a weak brittle link is not found - the possibility of collapse in the full scale load test of the earthquake exists.

Recourse to rules rather than this systematic first principle approach is possible in some structures, for example regular frames, and both the book by Blume, Newmark and Corning and the SEAOC code give rules for designing the shear capacity of beams which are based on the assumption that a hinge will form at each end of the beam. This is true in most cases but where the vertical load moments are large as in a long span beam there is a tendency for the hinges to move out from the ends of the beam, with a consequent increase in beam shear between the hinges (Fig. 6.21).

The need for this systematic first principle approach to the design of the structure between the hinges is best illustrated by examples and as with the plastic hinges themselves there is no difficulty in finding examples from actual earthquakes.

Fig. 6.22 shows clearly that the designer in making the beam to column connections over-strong (instead of designing for a plastic hinge at the beam ends) forced a shear failure at the centre of nearly every beam which actually led to the collapse of one tank (Fig. 6.22.c). With plastic hinges properly detailed at the beam ends, and adequate shear reinforcing between, these tanks could have been relied on to remain standing although perhaps with permanent deflection.

Fig. 6.23a shows a precast structure from the Alaskan earthquake - when the Tee members of Fig. 6.23.a were connected by the bolted joint of Fig. 6.23.b at the Tee ends (x in Fig. 6.23.a) they formed a series of 3-pin arches. Properly detailed these would have formed plastic hinges at point y on Fig. 6.23.a in a major shaking. Instead, before these hinges could develop, the connection x sheared its welds as in Fig. 6.24.b with damage as in Fig. 6.24.a or collapse of the Tees in some cases. Attention to Phase II Step 6 would have prevented this. It is possibly not an exaggeration to say that the majority of precast structures designed in New Zealand to our present codes would perform in this brittle way.

Again illustrative of the need for the systematic first principle approach in Step 6 is a new structure (Fig. 6.25) in which the precast columns are designed to form plastic hinges at a and b. This causes exceptionally high shears to be transferred from the precast column to the steel rafter at c (Fig. 6.25). Fig. 6.26 illustrates the special precautions necessary at c when the Step 6 approach is followed through.

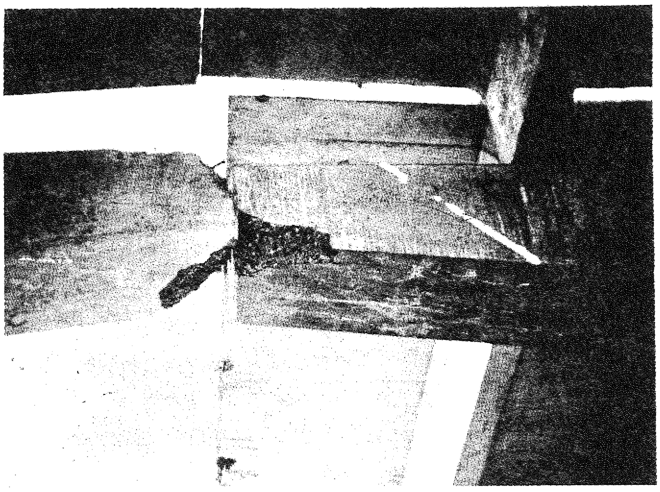


Fig. 6.24a

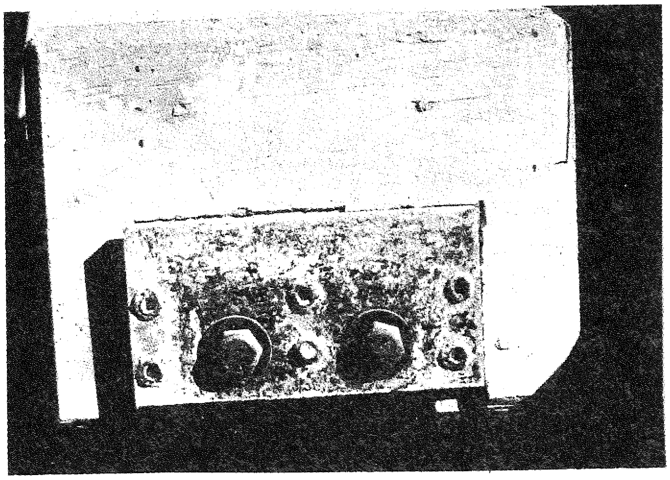


Fig. 6.24.b

Fig. 6.24 Examples of failures between hinges of actual earthquake: Connections of pre-cast members.

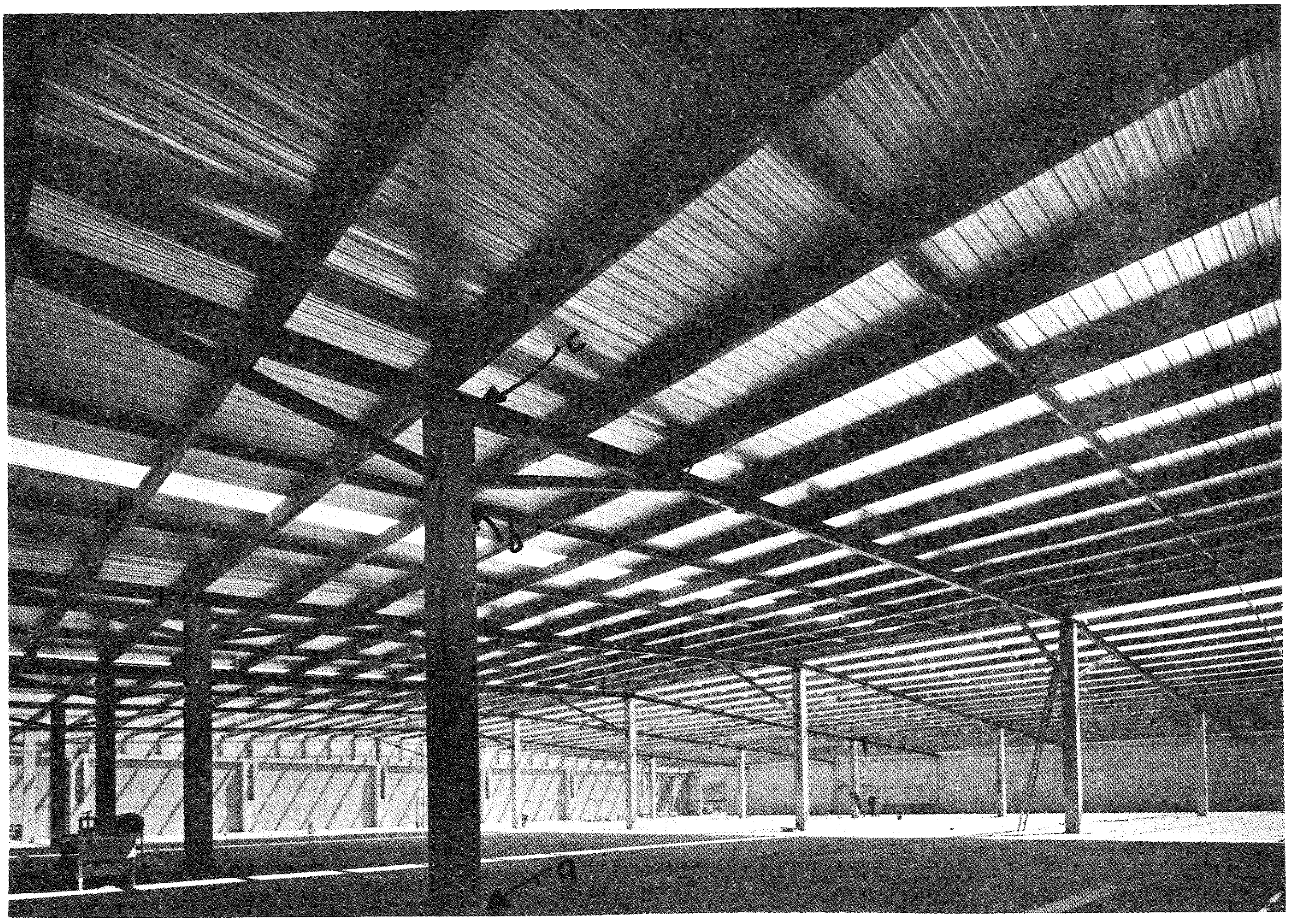


Fig. 6.25 Example of type of detailing needed to avoid failure between hinges.

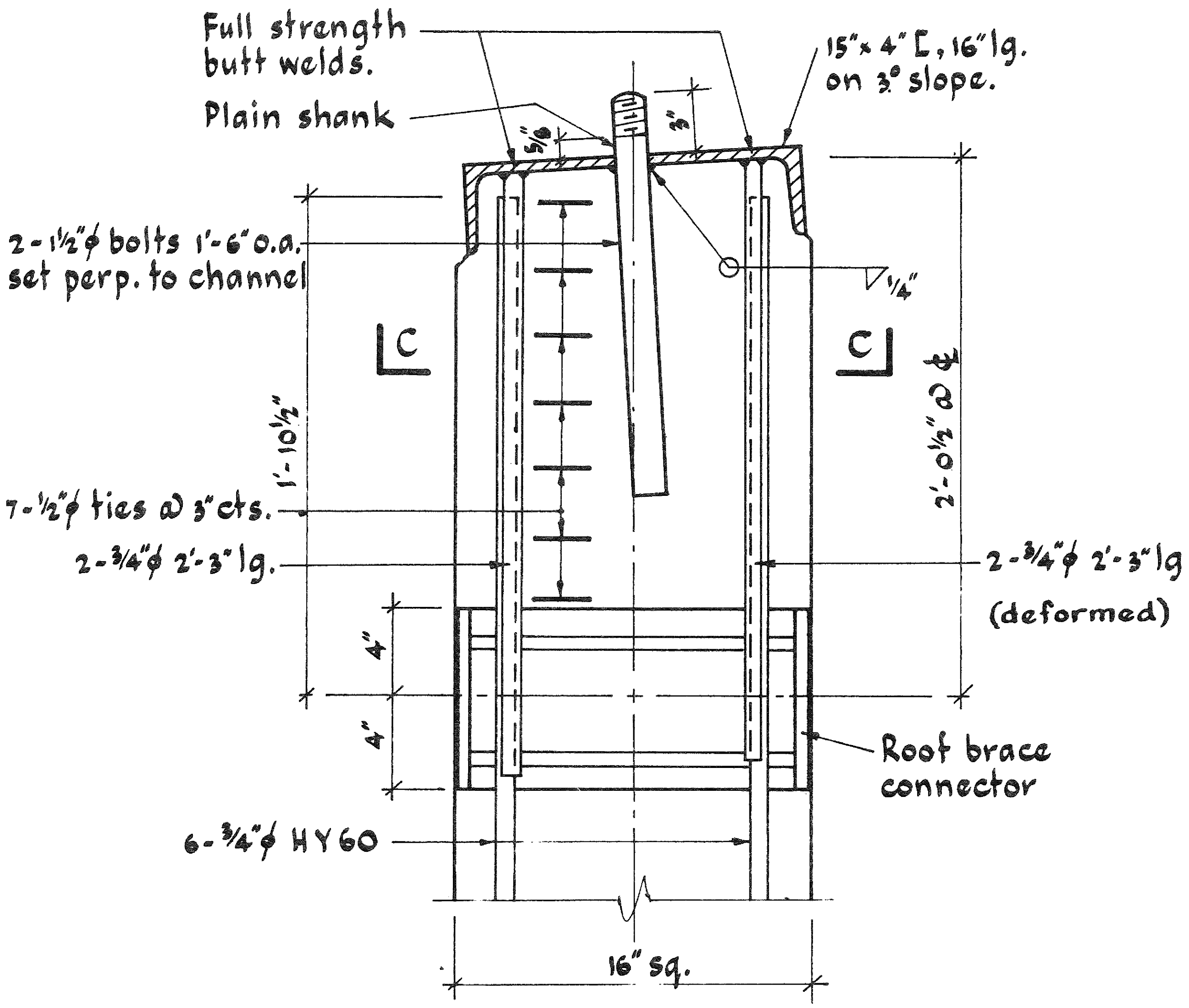



Fig. 6.26 Detail on Fig. 6.25.

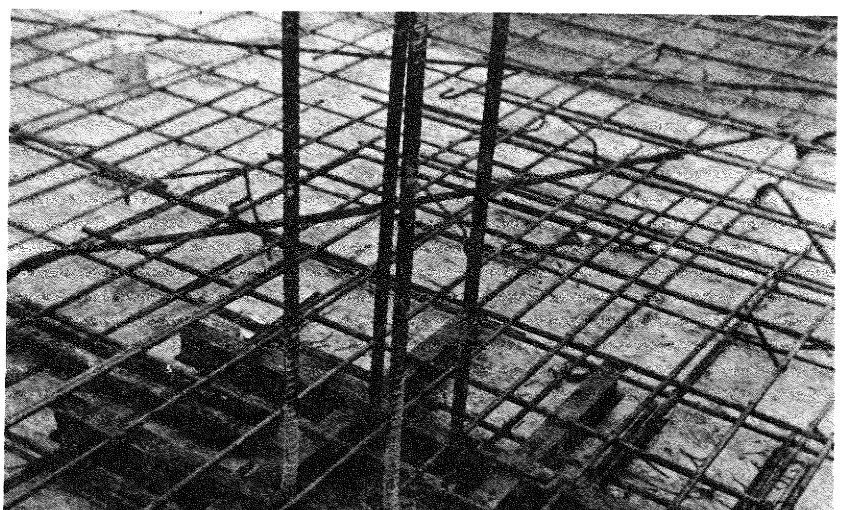


Fig. 6.29

Example of detailing needed to avoid failure between hinges.

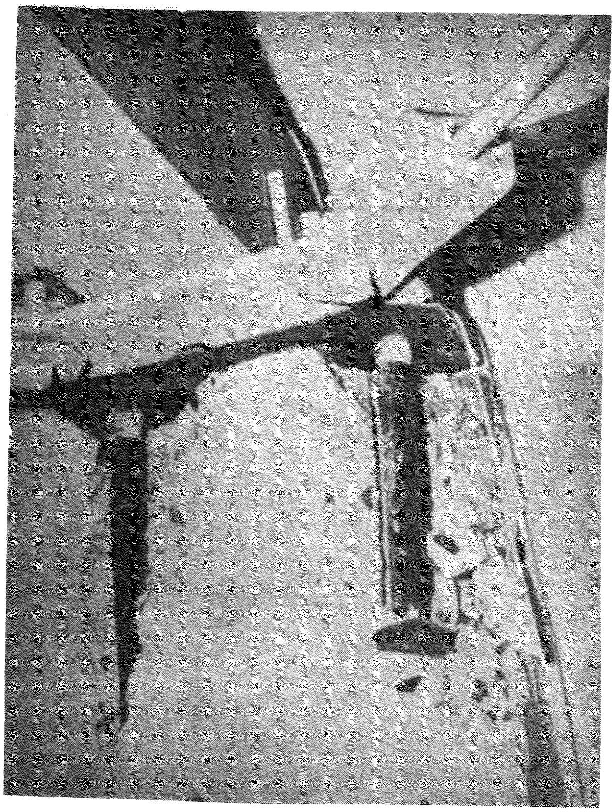


Fig. 6.27 Earthquake Damage at Roof to Concrete Column Junction

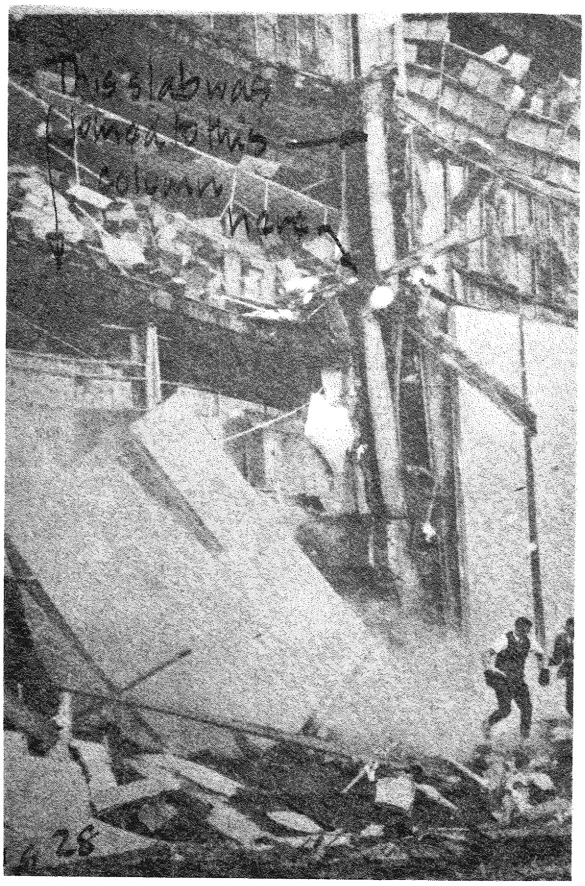



Fig. 6.28 Flat Slab Earthquake Behaviour.

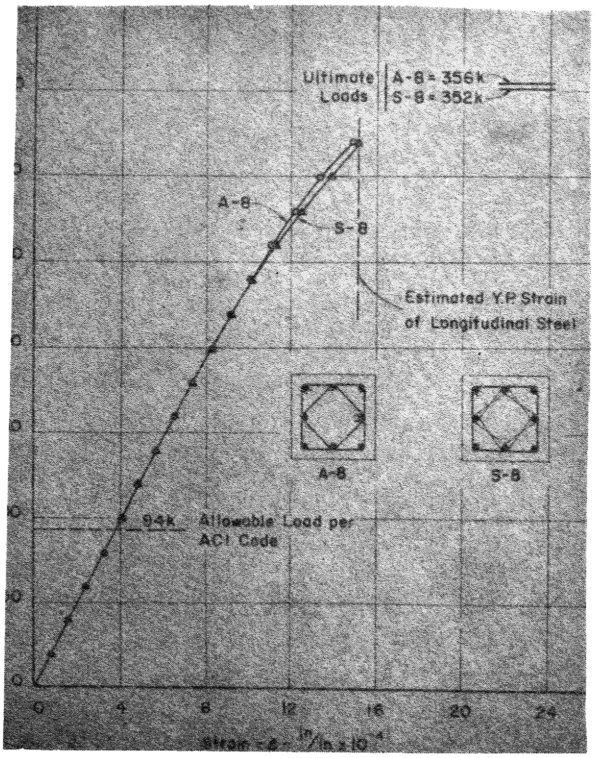



Fig. 6.30 Tied Column Load Test.

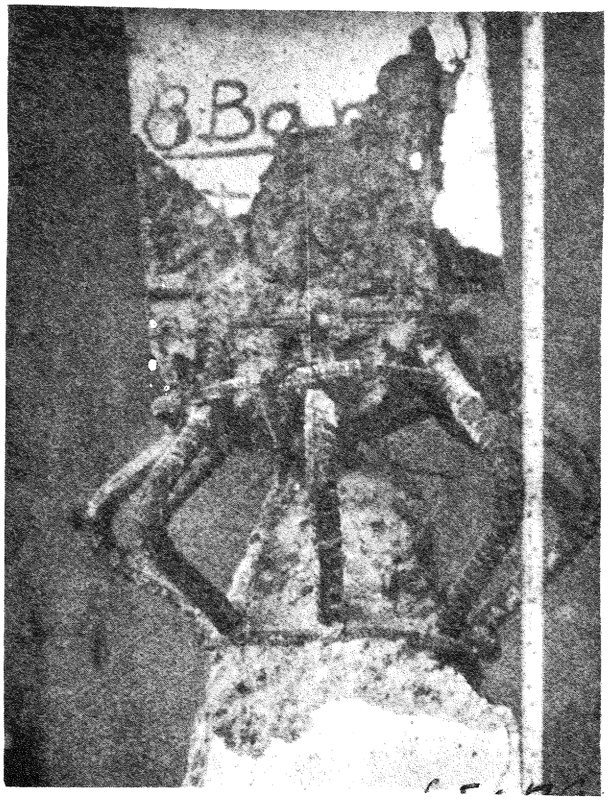


Fig. 6.31 Column of Fig. 6.29 After Testing.

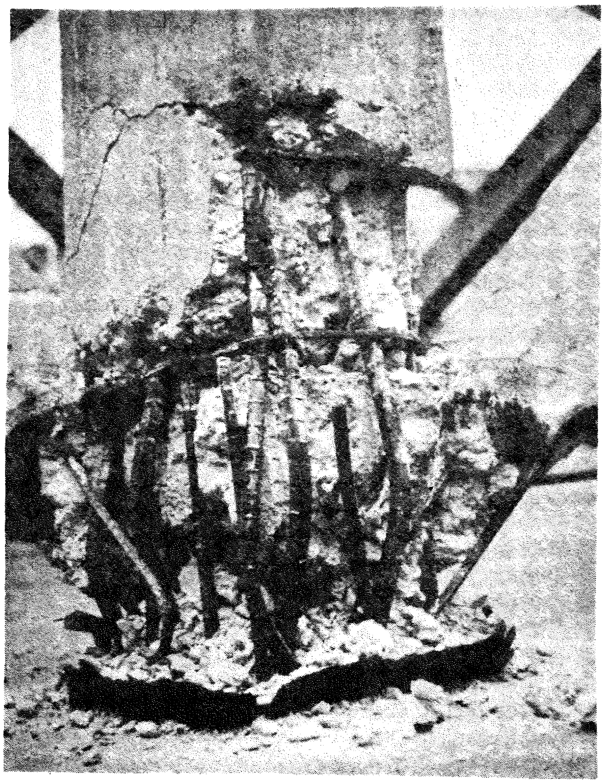


Fig. 6.32 Tied Column Earthquake Failure in Pure Compression.

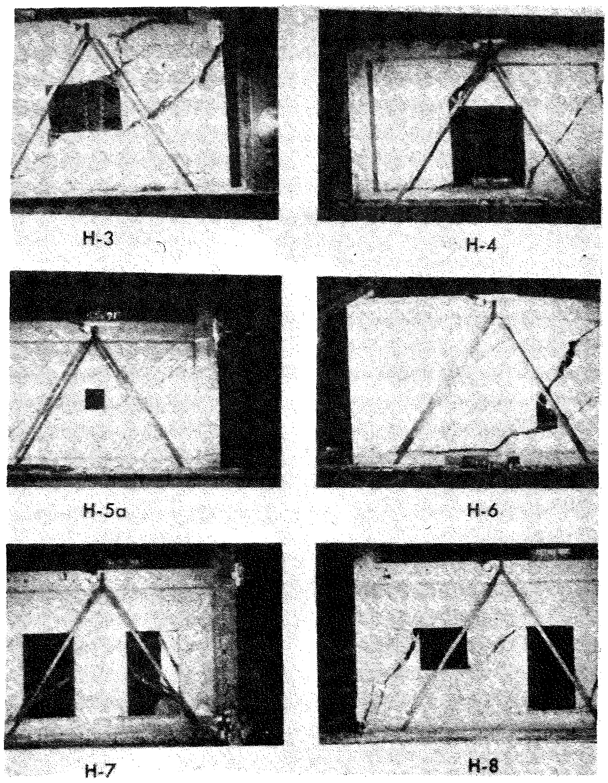
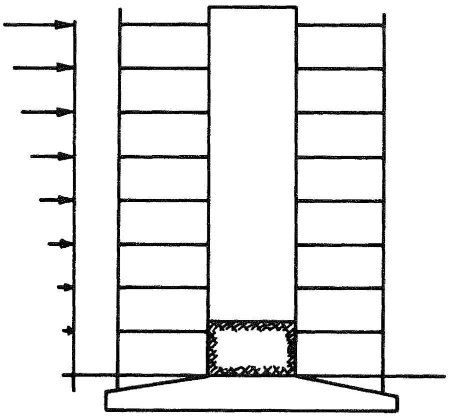



Fig. 6.34 Laboratory Tested Walls After Testing.



Designed to form a hinge in shaded zone.

Fig. 6.33 The Vertical Beam (Not a Shear Wall).

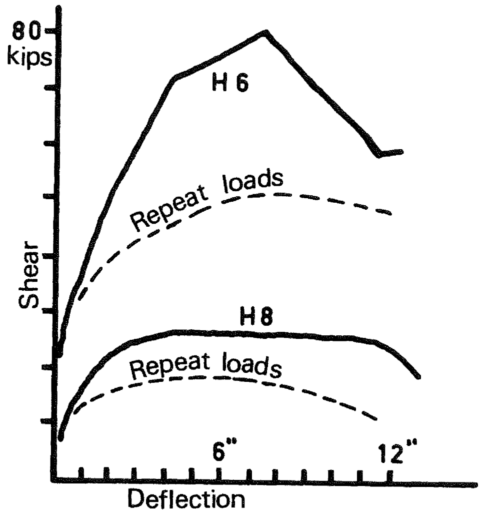



Fig. 6.35 Shear Wall Ductility.

Fig. 6.27 shows a similar earthquake damaged detail where such precautions were not taken. Fig. 6.28, again from the Alaskan earthquake, shows the failure of a flat slab to column connection. Again, attention to Phase II Step 6 would have ensured the development of hinges in column or slab before failure between the hinges, i.e. at the connection. Fig. 6.29 shows a steel member developed to prevent this type of connection failure where earthquakes impose high moments on these connections.

As a final example, the total lack of ductility of tied columns in pure compression has to be watched in Step 6 Phase II calculations. Figs. 6.30 and 6.31 demonstrate this form of brittleness by laboratory testing and Fig. 6.32 shows the compression leg of a braced tower which has failed in this way after an earthquake. Here the equivalent of the plastic hinge fuse should have been introduced by ensuring tension yield in the tower diagonals or tension legs before this compression failure occurred.

Check Stability and Secondary Damage - Steps 7 and 8, Phase II

Stability is not usually a problem in reinforced concrete structures because of the substantial sizes of the members. Stability should be watched however where the energy absorption is concentrated in a few members. For example, in multi-storey structures with column hinges or in single storey structures which act as vertical cantilevers - i.e. those which have only one hinge.

The problem of secondary or non-structural damage is important but as it is a large problem and is not peculiar to our subject of reinforced concrete it cannot be given space here.

## 7. The Shear Wall

We have now developed our design method in detail concentrating our attention on Phase II where our brittle material is converted to a ductile structure. The discussion has centred around structures which form plastic hinges and this has meant the omission of that important structural type the shear wall which, by definition, does not form plastic hinges. A shear wall we must define as a wall in which eventual failure is due to the shear capacity of the wall being exceeded and failure occurring in shear not bending. For example, the structure of Fig. 6.33 is not a shear wall although in common usage it is often called one. Rather it should be called a vertical beam since it can be designed to form a plastic hinge at its base before any shear failure occurs; i.e. the Phase II technique outlined above applies. In contrast Figs. 6.34 and 6.35 show the results of some of the very few tests done on concrete shear walls proper.

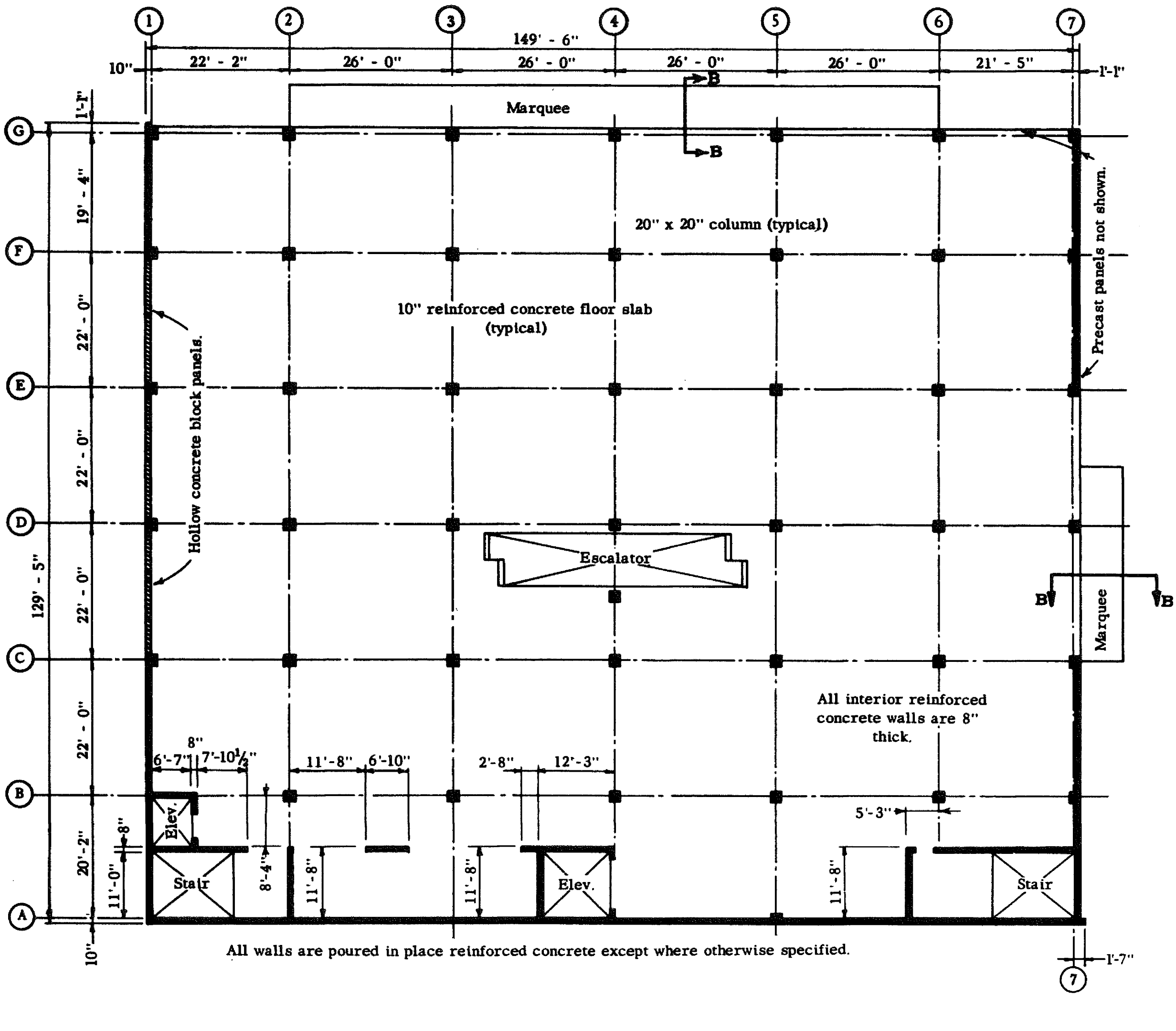
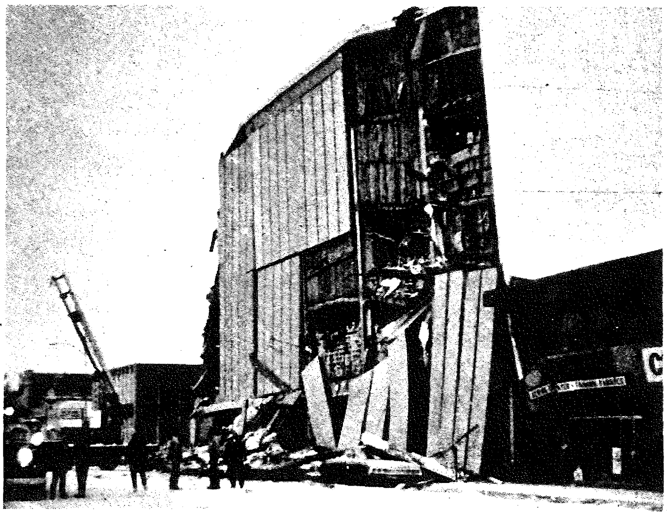
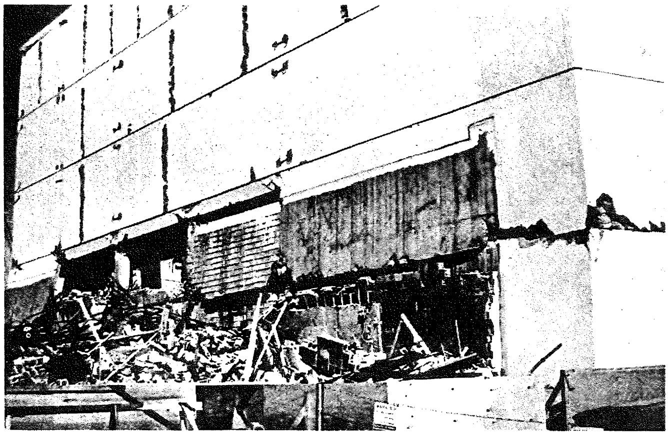


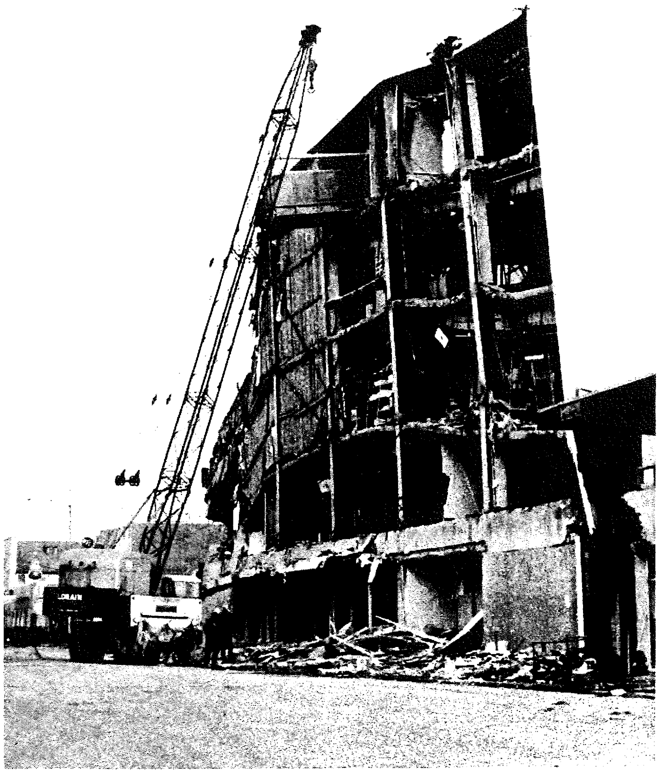

Fig. 6.36 Layour of Penney Building Shear Walls.



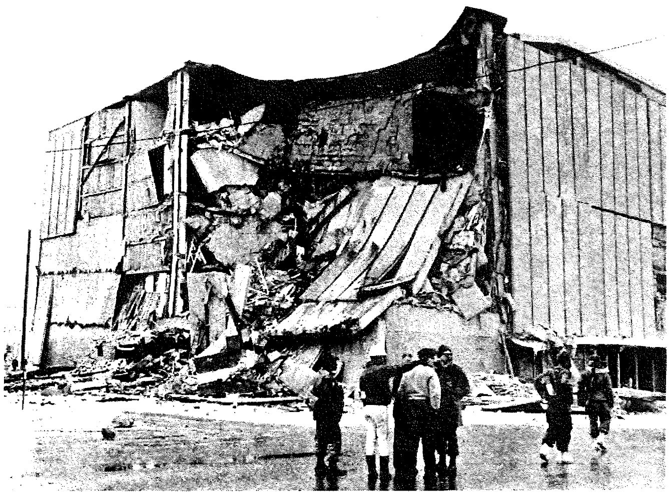
North elevation of Penney Building before a significant amount of debris had been removed.



West elevation of Penney Building during demolition. Adjoining building has been removed during demolition. Note that the west wall has shifted off the wall below, at the second-floor level.

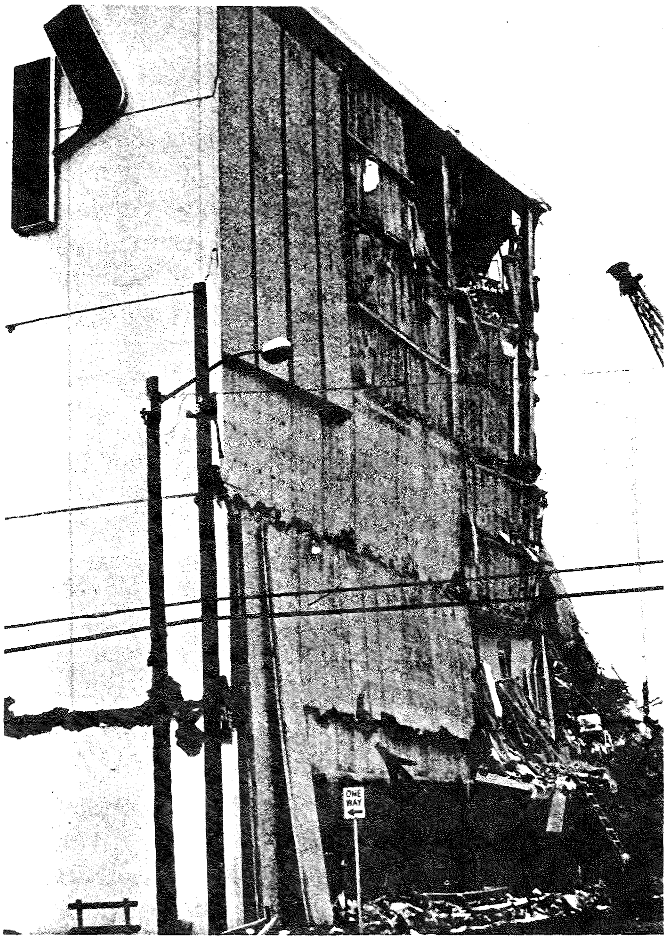


North elevation of Penney Building after much of the debris had been removed. Note that no frame existed above the second floor.

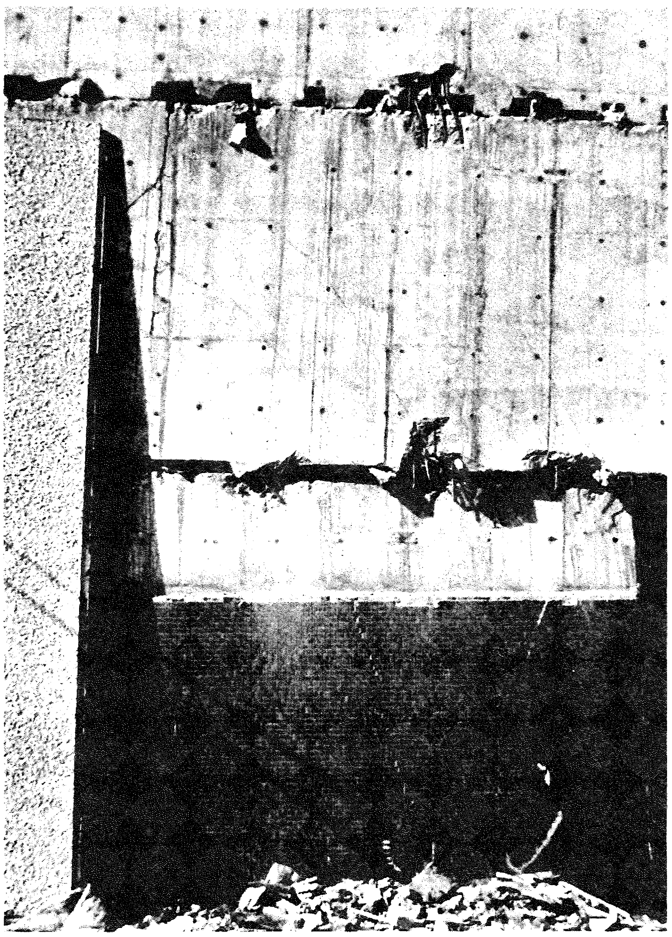


Northeast corner of Penney Building.

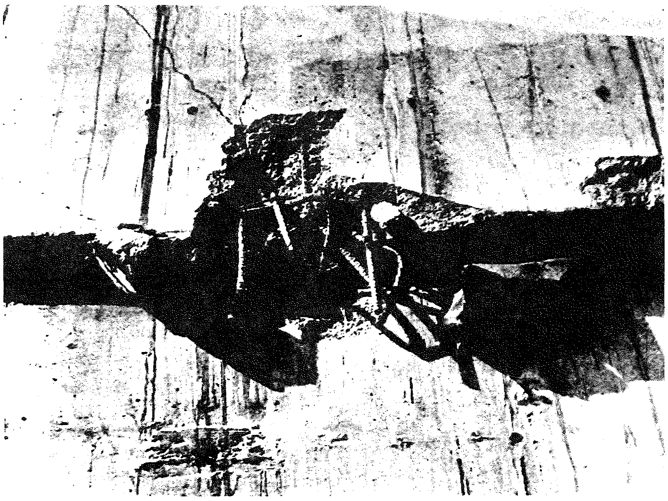
Fig. 6.37 (Part I) Performance of Penney Building Shear Walls.



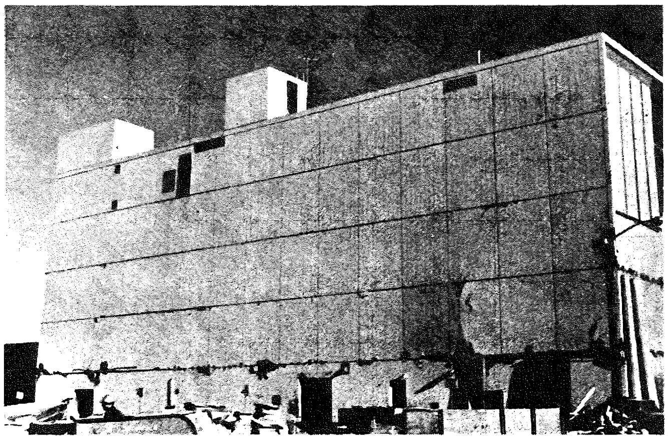
South elevation (at left) ; east elevation (at right).
Fallen precast panels have been partially removed.



Failure of column B–7 shown in figure below is also shown in center right. Fallen precast panels have been partially removed. Pockets at the third-floor level were for anchorages for precast panel facing. Pockets at the second floor have closed due to wall shifting and dropping.



Failure of column B–7 at second-floor line. Penney Building.



South elevation of Penney Building.

Fig. 6.37 (Part II) Performance of Penney Building Shear Walls.

Although the considerable area under the curves of Fig. 6.35 suggest the energy absorbing properties of these walls are good, great caution should be used when introducing them into important structures, especially high ones, for the following reasons:

(i) The tests are one cycle and one direction - other tests show that for more cycles the Fig. 6.35 curves follow a much lower line as shown dotted.

(ii) These walls are almost invariably used as bearing walls as well as shear walls - if in Fig. 6.34 one visualises the results of reversing the load and applying more than one cycle, what will happen to the bearing capacity?

(iii) The cost of structural repairs to these walls after shaking is likely to be close to that of rebuilding.

These thoughts tend to be confirmed by the few examples of shear wall performance in earthquakes; e.g. the Penney Building in the Alaskan earthquake, Fig. 6.36 and Fig. 6.37.

It must be concluded that the present state of our knowledge of the ductility of true shear walls is so sketchy that if used in other than low buildings they should not be relied on for ductile performance. This means that at Step 4 Phase I (Fig. 6.4) we should select a reduction factor which is close to one. (This is in contrast to our present code which implicitly allows high reduction factors for shear walls). Further we should consider where shear walls are also used as bearing walls in major structures the possibility of introducing other vertical elements (such as isolated columns designed at limit conditions) which will take over the important function of holding the floors apart if the shear wall bearing capacity is destroyed.

## 8. Summary

The essential concepts which this lecture has tried to emphasise may be summarised as follows:

(1) We know the stresses in structures which remain purely elastic under substantial earthquakes are greater than those nominated in our codes by a factor between 0 and 10.

(2) We know also that if we choose to make the elastic strength of our structures match those strengths required by our codes, rather than those required by the elastic response spectrum that satisfactory performance in a severe earthquake can be obtained by designing a ductile structure.

(3) Reinforced concrete is a material which is essentially brittle or glass-like in its properties.

(4) Our problem then is to design ductile structures from a brittle material.

(5) It is emphasised that this cannot be done reliably by haphazard methods or arbitrary rules. Instead we require a logical step by step process which engages the mind of the designer throughout and which when completed allows us to be assured that we have everywhere converted our brittle material to a ductile structure.

Acknowledgements: Figures copied in Mr Holling's paper.

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Base G.D. & Read J.B. "The Effectiveness of Helical Binding in the Compression Zone of Beams," Cement and Concrete Association, London - Figs. 6.8f, 6.11.

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de Cassio R.D. & Rosenbleuth E. "Reinforced Concrete Failures during Earthquakes", American Concrete Institute, Journal November 1961 - Figs. 6.19, 6.20, 6.22.

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Steinbrugge K. et al, "Damage to Buildings and Structures" San Francisco Earthquakes of March 1957, Special Report 57, California Division of Mines - Fig. 6.27.

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"The Prince William Sound Alaska Earthquake of 1964 and After-shocks", Vol.2, Part A, U.S.C.G.S. U.S. Dept. of Commerce, Washington D.C. Figs. 6.15, 6.16, 6.23, 6.24, and 6.24b, 6.28, 6.36, 6.37.