LESSONS FROM SOME RECENT EARTHQUAKES IN LATIN AMERICA

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Synopsis

Several moderate and intense earthquakes that have ocurred in the last few years, affected a great number of modern buildings in some important Latin American cities. Design criteria, quality of materials, and workmanship vary widely. Although some of the architectural features are peculiar to some regions, most lessons derived from these earthquakes are applicable to constructions located anywhere in the world. The paper deals with problems such as ground motion characteristics, foundation behavior, overturning moment, and brittle failure of structures.

Introduction

Knowledge derived from the study of the effects of earthquakes on structures serves to improve design criteria, to evaluate them, and to call attention on faulty habits and systematic defects. Significant information concerns ground motion characteristics, structural response, and their interaction. The latest observations have confirmed some previous ideas. Besides, they have revealed some facts that had passed unnoticed and others that have not been assimilated within design practice. These are the observations with which this paper is mostly concerned. The shocks (and their respective properties) studied in this paper are the following.

Date	Magnitude	Place affected	Focal distance	MM intensity
11 May, 1962	6.7	Acapulco	70	8
19 May, 1962	6.5	Acapulco	100	9
6 July, 1964	6.5	Acapulco	190	6
		Mexico City	250	6
3 May, 1965	6.2	San Salvador	12	7, 8
9 December, 1965	6.8	Acapulco	60	6
		Mexico City	330	6
17 October, 1966	7.6	Lima	210	7
29 July, 1967	6.5	Caracas	80	6, 7

1. Response spectra on soft soil

Prediction of response spectra on soft soil is based on the previous knowledge of the properties of the layered medium, such as thickness (H), rigidity modulus (G), shear wave velocity (v_s), and internal viscous demping (η), of each stratum. This permits the computation of a curve of period versus amplification factor. Multiplication of this factor by the corresponding undamped spectral ordinate for the free surface of rock (as if there were no soil layers on it) gives the spectral ordinate on soft soil.

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Herrera and Rosenblueth's model[1] is used for computing amplification factors. It assumes one-dimensional ground motion, linear behavior of soil, and frequency-independent wave attenuation; the motion arriving at the rock soil interface is assumed to be a Gaussian stationary stochastic process.

Previous attempts to predict spectra on soft soil were based on the expected spectra on rock, as computed by using empirical correlations in terms of earthquake magnitude and instrumental focal distance[2]. It was then concluded that measured and predicted spectra had the same shape, but that their absolute values differed by a factor of about 2. Lack of agreement was ascribed to uncertainties in the predicted spectra on rock.

In this study, actual spectra on rock for the earthquakes of July 1964 (fig 1a) in Mexico City are multiplied by the magnification factors in order to obtain the predicted spectra on soft soil. Fig 1b shows the properties of the layers of soft material for a zone in Mexico City for which spectra were obtained from strong motion records. These parameters were estimated by combining geophysical results[3] with laboratory tests[4]. This was a necessary step, since amplification factors are very sensitive to variations in properties and thicknesses of the layers, mainly near the dominant period of the spectrum. Thus, a change in the thickness of the top layer from 15 to 14 m, of its rigidity modulus from 90 to 120 kg/m^2, and of its shear wave velocity from 27 to 30 m/sec, shifted the dominant period from 2.45 to 2.15 sec.

Figs 1c, d compare actual and predicted spectra for two different components of the July 1964, Mexico City earthquake. Spectra on soft soil corresponding to two sites separated 1 km are used. There is satisfactory agreement between actual and predicted spectra, although some difference exists between the actual spectra computed from records obtained only 1 km apart, where the overall soil conditions seem to be the same. This discrepancy may be attributed to differences in preconsolidation conditions.

Satisfactory behavior of the mathematical model in the case under study makes it feasible the simulation of earthquakes on soft soil, provided it could be idealized as behaving linearly throughout the motion.

At distances of about 50 m from each of the sites to which the spectra of figs 1c, d correspond, simultaneous records were obtained on the foundations of two buildings, 22 and 13 stories-high. First and second natural periods for the first building were 2.06, 0.57 sec in the NS direction, and 1.90, 0.36 sec at right angles to it. Periods of the second building were 1.20 and 0.44 sec in the EW direction.

All the undamped spectra on the foundations have practically the same peaks as those on the ground, but shift about 0.4 sec towards the long period range. In one case, peaks are reduced to about 70 per cent for the intermediate period range, and to much less for periods shorter than 0.5 sec, while there is no significant change in ordinates for periods longer than 3.0 sec. In the other building, peak ordinates in the intermediate range are increased, while the variations in the extreme ranges agree with those observed in the first building.

Differences become negligible at damping ratios of about 0.2.

2. Correlation between significant earthquake parameters

The Acapulco (9 Dec 1965) and the San Salvador earthquakes were characterized by ground accelerations and velocities of unusual amplitude, much greater than what would be consistent with the observed intensities, according to accepted correlations[5]. Maximum ground acceleration and velocity in Acapulco, obtained from an accelerogram, are 270 cm/sec^2 and 17 cm/sec respectively[6]. The duration of the intense phase was about 8 sec; although the motion cannot be described as consisting of one single shock, the greatest pulse was about twice larger than any other. MM intensity at the instrument site was estimated as 6, whereas that computed using the intensity-velocity correlation of ref 5 is 7.9.

In San Salvador, only a seismoscope record was available. It shows that at the seismological station, on firm rock, the motion was of the single pulse type.[7] Maximum ground velocity was estimated as 14 cm/sec, wherefrom the computed intensity turns out to be 7.6; the observed intensity was smaller than 7. In a region of about 25 km^2, where soil conditions comprised from loose sand to artificial fills, the intensity fluctuated between 8 and 8.5. The form of cracking of structures located within that area indicates that there, the ground motion also consisted of a single strong shock. In a factory located in the same area, some pieces of machinery underwent horizontal displacements, all in the same direction. The most rigid one moved about 5 cm on a concrete floor, whose friction coefficient with the machine was estimated as 0.2. This information, coupled with several alternate reasonable assumptions about the shape and duration of the single pulse of which the ground motion consisted, led to the conclusion that ground acceleration must have exceeded 0.5 g, and that the maximum velocity was about 60 cm/sec, from which an MM intensity above 9.5 would be derived. Again, this is significantly greater than that observed.

Response spectra were computed for both components of the Acapulco earthquake[6]. The fact that structural damage was negligible seems to be consistent with the low values of spectral ordinates for a wide range of periods, despite the high value reached by the maximum ground acceleration. Compared to most acceleration spectra of earthquakes recorded on hard ground, the one of Acapulco has a much more pronounced variation passing from the short to the long period ranges. This is seen in fig 3, which shows ground motion records and acceleration spectra for both horizontal components of the El Centro shock of 1940[8] and for those of the Acapulco earthquake. Both had about the same magnitude and focal distance and were recorded on hard ground. Maximum undamped spectral ordinates are nearly the same for both earthquakes (3 g). These maxima correspond to very short natural periods (less than 0.1 sec) for the motion recorded in Acapulco. On the other hand, for periods longer than 0.2 sec, the El Centro ordinates are at least twice those of Acapulco.

Recognition of differences in earthquake effects according to the duration of the strong part of the motion and to the frequency content leads to better estimates of seismic risk at given sites. This requires knowledge of the conditional probability distribution of those parameters once an earthquake of given magnitude and focal distance is known to have occurred. At least for the case of San Salvador an approximate formulation in those terms seems advisable; earthquakes affecting the city belong to either of two groups: one includes small-magnitude motions at small depths and epicentral distances, and the other concerns bigger earthquakes generated along faults in the Pacific trench. Earthquakes in the first group have short duration in San Salvador, while those in the second share the known characteristics of earthquakes of intermediate duration on hard ground.

3. Behavior of foundations

The structures where some of the records of fig 1 were obtained belong to a group of over 130 apartment buildings distributed over an area of 1.5 km^2, north of downtown Mexico City. Their heights vary from 1 to 22 stories. Those taller than 12 stories have partially compensated foundations, and reinforced concrete friction piles with lengths varying from 16 to 22 m. Allowable foundation settlements were taken as 30 to 40 cm and the piles were designed accordingly. Their capacities were deduced from theoretical formulas and load tests. Despite efforts to avoid eccentricities in the foundation reaction to dead and live loads, unacceptable differential settlements were observed in some of the tall buildings under static conditions. In order to reduce them, asymmetrical dead load was added in foundation cells.

Practically no structural damage was experienced by these buildings during the July 1964 earthquake. However, the behavior of the foundations brought out the variability of the local soil properties. Thus, in one of the five type L 14 stories-high buildings, differential settlements occurred during the earthquake caused a 56 cm (1.4 per cent of height) horizontal deviaton of its top with respect to its original position. Buildings type K, 13 stories-high, showed differential settlements of 1 to 4 cm and horizontal displacements of their top smaller than 30 cm. For buildings type M. 22 stories-high, the deviations from the vertical lied between 4 and 36 cm, and out the seven buildings type N, also 22 stories-high, one did not suffer appreciable tilting, five had lateral deflections at their top varying from 5 to 48 cm, and one of them became about 1 m out-of-plumb (1.7 per cent of height). Nineteen of these twenty-three buildings mentioned had been ballasted. Hence, contact pressures at some zones resulted higher than advisable. Such pressures were increased by overturning moments during the earthquake. The ensuing tilt was in some cases opposite to that observed before the motion.

An evaluation of actual working conditions was carried out after the earthquake. Vertical loads were seen to be in one case 8 per cent higher than those assumed in design. The bearing capacity of the foundation was computed assuming that pile-clay bond equals two-thirds of cohesion, and applying Meyerhof's theory for footings on cohesive soils. The total capacity was conservatively computed as the sum of the soil shear capacity plus the one due to the bond of all the piles. Vertical pressures due to dead and live load, excavation, and overturning moment were included as acting forces. A safety factor of 1 was obtained in some cases for overturning moments of the order of 40 to 60 per cent of those required by the local building code. Hence, underpinning was advisable.

The corrective measures taken consisted mainly of addition of electrometallic piles, 2 in. in diameter, under the foundations of all buildings whose factor of safety, for the loading conditions specified by the Mexico City Building Code, including earthquake, was smaller than 1.1. These piles were selected because they may be driven with light equipment, they do not produce local remolding, and the development of their maximum capacity may be accelerated by means of electro osmotic treatment. Their capacity is mainly developed by bond with the clay and they do not restrict the general consolidation of the clay formation. Besides, the foundations of some buildings were widened. The procedures for decreasing tilting were mainly based on differential consolidation obtained by application of asymmetrical pumping. Up to June 1968 all the foundations treated by pumping have shown satisfactory behavior. The rate of settlement has decreased considerably with regard to that observed before the earthquake. No important differential movements have taken place. The earthquake of November 1965 did not produce significant displacements beyond slight correcting movements that took some buildings closer to their vertical position.

4. Torsion of structural members

Local torsional stresses due to earthquake are usually neglected in design. This practice receives support from the argument that, although the occurrence of such stresses is recognized, equilibrium can be generally accomplished without their participation. This reasoning fails to recognize that brittle failures due to torsion may prevent structural members from developing other stresses, required for equilibrium, and thus lead to general failure. An example of this situation is shown in fig 4, which illustrates the behavior of a four-story school building during the Lima earthquake. A schematic drawing of the ground floor is presented, showing the one-bay structure. All columns are circular, and some of them are embedded along more than half of the story height in a stiff concrete block. This leads to greater linear and torsional stiffnesses than for the rest of the columns. Important story torsions caused by this arrangement must have taken place during the earthquake, producing, in turn, column shears and twisting. Since all columns have the same cross section, the torque in each column was, in essence, inversely proportional to its free length. Fig 4b shows the consequent spiral cracking in the short columns. None of the long columns exhibit any damage, not even those farthest from the center of torsion. Twisting failure of columns was also observed in Mexico City in 1957 and in Acapulco[9] in 1962.

Important local and general damage was suffered by some buildings in Lima and Caracas, as a result of the practice of ignoring the effects of girder twisting.[10] In some cases, as in the hotel shown in fig 5, these effects were mostly related to the neglect of torsional stiffness of girders, and hence of the moments that they produce at the ends of the secondary bear supported by them. This explains the tension failure caused by bending at those places.

Much more dangerous are those instances where lateral strength of frames depends strongly on torsional capacity of girders. Such is the case in structures having girders in one direction and flat slabs (waffle slab or shallow concrete ribs) in the other, which will be called the weak direction. This system is widely employed in both Lima and Caracas. Important twisting of girders results as a consequence of equilibrium and compatibili ty requirements when lateral loads act in the weak direction. Observed behavior is consistent with this: torsional craking of some girders at a three-story school building in Lima, and of practically all girders in the lowest three of a 16-story building in Caracas (fig 4c, d). At least for the last case, it is known that torsional stresses had been ignored in design, even though the framing system in the weak direction consisted only of shallow concrete ribs whose end bending moments could be transmitted to the columns only through girder twisting.

Had the floor system been a flat-slab with adequate capitals, it might be argued that proper continuity of slab and columns would have been possible without girder participation. Still, compatibility of deformations may give place to high twisting stresses, the associated failure is of a brittle nature, and should this failure occur, it might leave the girder in poor conditions to work in the strong direction. The comments of the following section, about the need for giving explicit consideration to ductility requirements when designing, find wide applicability in these cases.

5. Strength, stiffness, and ductility

Although most cases of earthquake-caused failures of structures are in some way related to brittle behavior, special consideration is given in this section to constructions that satisfy all strength requirements of modern codes, but for which overstrength against ductile modes of failure gives place to brittle fracture in modes having lower safety factors.

Modern school buildings in many countries have some features in common: they are long in plan and have one or two bays; one or two of the longitudinal frames have free columns, while the other has wall panels that rise from the floor to about 50-100 cm below the floor system of the upper story. This leaves the columns embedded in the walls throughout most of the story height, except at the uppermost 50-100 cm. These free portions have to take the greatest percentage of the lateral forces, since they are in general more than 30 times stiffer than the columns of the other longitudinal frames. The depth-length ratio of the free portions of the restrained columns are such that, unless special reinforcing and detailing precautions are taken, their bending capacity (ductile mode) is always greater than their ability to withstand the associated shearing forces (brittle mode). Structural damage caused by this deficiency, ranging from diagonal tension cracking of some columns to partial collapse, was observed in Acapulco, San Salvador, Lima, and Caracas. Hence, it is hard to justify the construction of new buildings with these characteristics. At least two solutions that preserve the essential architectural features are feasible: use of flexible joints between wall panels and frames, or construction of them in different planes. The behavior of the now famous hotel in fig 5 during the Caracas earthquake also had its origin in the failure of conventional design criteria to fulfill basic principles leading to ductile behavior. A cross section of the structure is sketched on fig 5a. Pairs of shear walls rise from elevation 3 to the top. This may have required much energy to be dissipated through inelastic deformation of the frames below. Sway of the lowest story, from right to left,is prevented by a retaining wall and its fill. All columns shown have constant circular cross section throughout the lowest three stories.

Those at the interior longitudinal frames are 1.2 m in diameter and are rein forced with 30 plain bars 1-1/8 in. in diameter and 3/8 in. ties at 15 cm on centers. Columns at the exterior frames are 1 m in diameter. Girder stiffnesses at elevations 1 and 2 are substantially lower than those of the columns.

The structure had been designed for a base shear coefficient of 0.05. On this basis, and using approximate values of member stiffnesses, and dead and live loads, the bending moment diagram of fig 5b was obtained; it has only one inflection point, and maxima occur at elevations 0 and 3. Fig 5c shows a view of some columns of the third story after the earthquake: diago nal cracks crossing the columns, all in the same direction. The same type of failure occurred in all the columns of row C, near floor 3, as well as at one of row B, at the same elevation. None of the columns of the main frames A-D suffered appreciable damage at the lower stories, despite the fact that horizontal shears must have been greater there.

An approximate stress analysis assuming lateral forces acting from right to left led to approximately equal safety factors in diagonal tension and in tension caused by axial load and bending . The effects of axial load and bending moment on shearing force capacity were taken into account.[11] It was concluded that diagonal tension stresses due to the combination of axial load, bending and shear were the cause of failure. Because shear strength is minimum where bending moment is maximum, failure occurred at the third story, while the second story remained practically undamaged.

The danger of overemphasizing ductility, in demerit of strength and stiffness requirements, was brought out by instability failure of some canopies that had ductile columns (fig 5d).

6. Overturning moment

Drastic reductions of overturning moment are permitted by some earthquake design codes. Although their analytical justification is not adequate in all cases, the behavior of actual structures seemed to support them up to recently. Some doubts about the limitations of this practice have arisen as a consequence of the great number of buildings where this phenomenon was the cause of important structural damage during the Caracas earthquake.

Typical buildings in Caracas are particulary vulnerable by overturning effects. Their slenderness, measured as the height-to-width ratio, in many cases exceeds 5 or even 7, which are not common values in buildings that have been subjected to intense earthquakes in other regions. End frames are usually filled with masonry panels at all but the first story. Therefore, seismic shears in upper stories are mostly taken by the end frames, while shear in the first story has a more uniform horizontal distribution. In these frames, the ratio of axial load to column bending is higher than for buildings of more usual proportions, and failure stresses are reached at relative ly small values of lateral forces. Also, local ductility at the sections of maximum moment is lower than in usual buildings, since it decreases as the ratio of column compression to axial load capacity increases.[8]

Both problems turn more significant when, as usual in Caracas, no special provisions are taken to properly confine concrete and compression steel.

There were many buildings whose damage can be traced to these causes: compression failure of corner columns, usually in the first story, or at the column-girder intersection in the first floor, thus bringing out the advisability of providing column ties at joints.

It is good to remember that current methods of analysis systematically underestimate the contribution of permanent (dead and live) lo: . to compression in corner columns. This tends to decrease the ratio of the safety factor in compression to that in bending, and hence to provoke brittle behavior during earthquakes.

7. Continuous joints in reinforced concrete frames

Careless detailing of joints in reinforced concrete structures has been the cause of important earthquake damage. Experimental research on the behavior of beam-column joints have shown that concrete confinement is essential to the attainment of strength and ductility. The behavior of actual structures during earthquakes indicates that lack of adequate lateral confinement is especially critical for knee-joints, which are those connecting only two members that intersect at right angles or nearly so. Several joints of this type failed in Lima and Caracas (fig 6), leading in some cases to structural collapse. Most buildings in these two cities have beams in only one direction. Hence, lateral confinement of joints is not furnished entirely by slabs or beams, but depends, instead, on the amount of ties. Other than the compression failures due to overturning moment (fig 6d), practically all the joints that failed were of the knee type. A simple qualitative study of stress distribution at and around joints indicates that the capacity of a knee-joint depends on diagonal tension, bond and stress concentration problems, which partially disappear at joints connecting more than two members. For instance, column compression is very helpful for reducing the strength needed to overcome these three problems at joints connecting two columns and one girder, even though it creates problems related to compression failure of unconfined concrete and buckling of unrestricted longitudinal reinforcement. In other cases, such as at top floor joints, which connect two beams and one column, the presence of the second beam brings about reductions in stress concentrations and in bond requirements.

Laboratory studies agree with the foregoing remarks: at ordinary joints the essential problem is lateral confinement; at knee-joints, bond and stress concentration problems are especially significant[12]. This explains the observed failure of more knee-joints than of any other type.

The problem depicted in fig 6a is virtually ignored by many designers. The presence of masonry wall panels filling the bays of reinforced concrete frames may produce interaction stresses at the ends of the members. Since the stiffness of the masonry panel is usually several times the one of the frame, a great portion of the seismic shears is taken by the wall. This in turn induces exceptionally high interaction stresses. The photographs in fig 6b were taken in Acapulco and Lima. They show the type of failure which may occur as a consequence of neglecting this interaction. A quantitative criterion for providing reinforcement intended to avoid this type of failure has apparently not been developed. From the behavior of laboratory specimens, diagonal cracking of the frame is known not to occur before that of the wall.[13] Therefore, if wall failure is not expected for sufficiently strong earthquakes, no special provisions seem necessary. However, if the wall is not sufficiently strong, it is advisable to substitute it by an even weaker panel and thus limit its interaction stresses with the frame. This recommendation stems from the advantages of the strength-plateau behavior and from the fact that it is usually more expensive repairing the frame corner than building a new wall panel. Alternate solutions comprise construction of flexible joints between wall and frame, or reinforcement of the latter's corners. Experimental information is desirable in order to accomplish economical solutions of the last type.

8. Variation of natural periods of buildings

The fundamental periods in two directions of eight buildings were measured before the Lima earthquake and immediately following it, by recording vibrations induced by wind and by other sources of small amplitude motion. The building heights ranged from 11 to 22 stories. Excepting two buildings, the variations were not greater than 15 per cent. In these two, the final values were 3.8, 2.6, 2.0 and 1.35 times the original ones. Only in these structures some damage was apparent; it consisted of slight cracking of partitions. Similar observations performed after the Mexico City earthquake of 1964 on the buildings described in section 3 showd 50 per cent increases in natural periods, caused by plaster cracking. Replastering returned periods to their original values.[14]

9. Conclusions

Evaluation of seismic risk must consider the occurrence of earthquakes being originated in different neighboring regions. In some cases, the properties of each earthquake greatly depend on the characteristics of the region where it is generated.

The behavior of structures in Acapulco and San Salvador confirms the fact that the destructive power of earthquakes is imperfectly described by simple quantitative parameters such as the maximum values of ground velocity and acceleration. At least the duration and the frequency content must be explicitly considered when designing models to simulate seismic records. The accelerograms and spectra obtained for Acapulco partially contradict the assumptions of recent work concerning the spectral density fuction for earthquakes of given magnitudes and focal distances. It is not clear whether this discrepancy is significant or is due to the difference in soil properties at the recording sites.

Response spectra at the top of layered soft soil may be predicted in terms of spectra at the base rock, provided that motions are sufficiently moderate as to give place to linear response of the layered medium. Undamped acceleration spectra corresponding to records obtained on the foundations of tall buildings preserve all the peaks that correspond to records obtained directly on the ground, but they are slightly shifted towards the long period range. Amplitudes are reduced in the short period range. No significant changes were observed for structural damping greater than 0.02.

Foundation and structural failures due to overturning moment are an actual problem. Protection against them should include adoption of load fac tors greater than those used for story shears, as well as explicit consider ations of ductility requirements at columns and joints.

There are many instances of inadequate behavior of structures designed for reasonable lateral strength requirements, and built according to high standards. In most of them the behavior may be explained in terms of available knowledge. It seems advisable that design codes incorporate explicit limitations, intended to decrease the probability of brittle failures. Ductile modes of failure should serve as safety valves to avoid brittle behavior.

10. References

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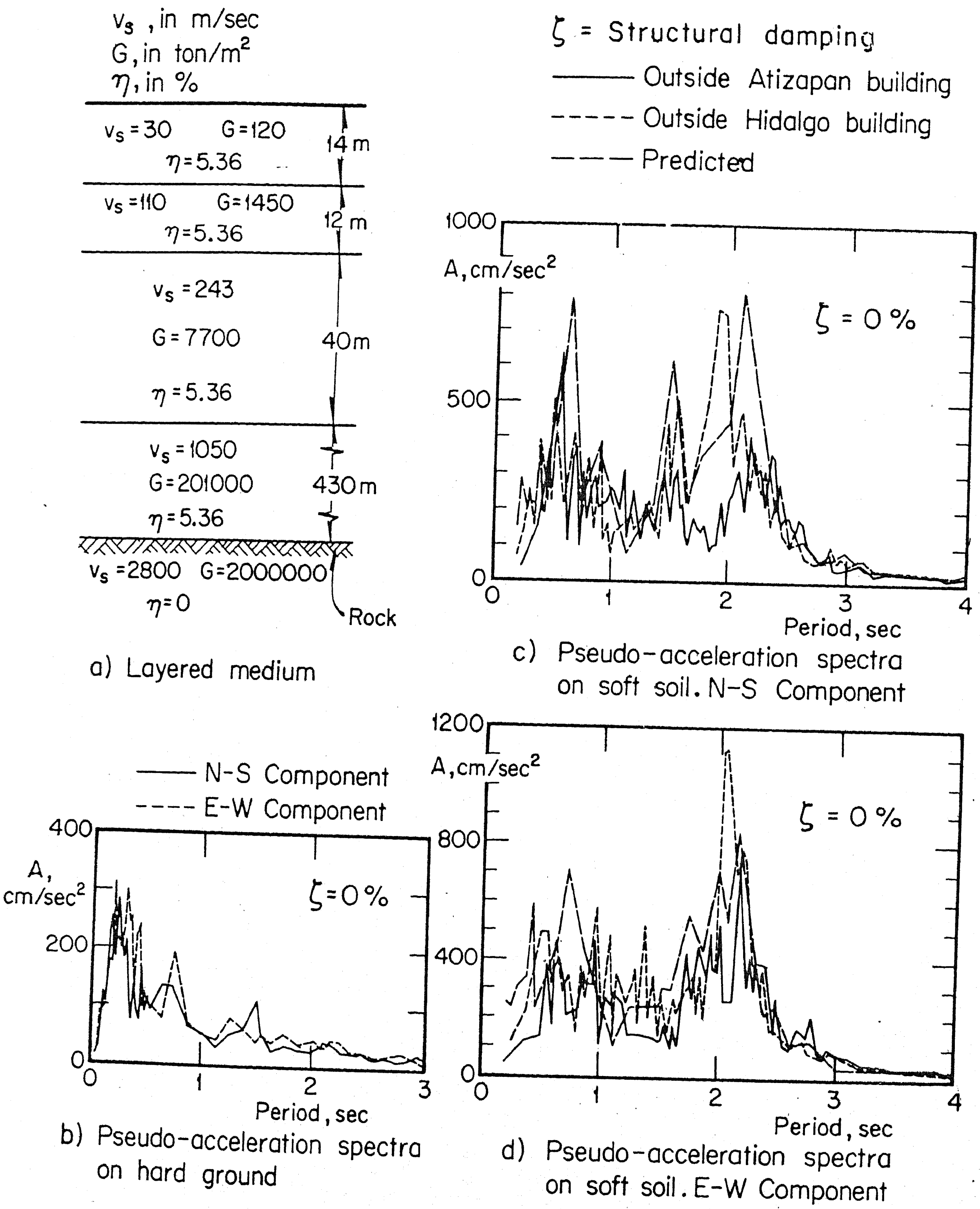


Fig. 1 Comparison between observed and predicted undamped spectra on soft soil

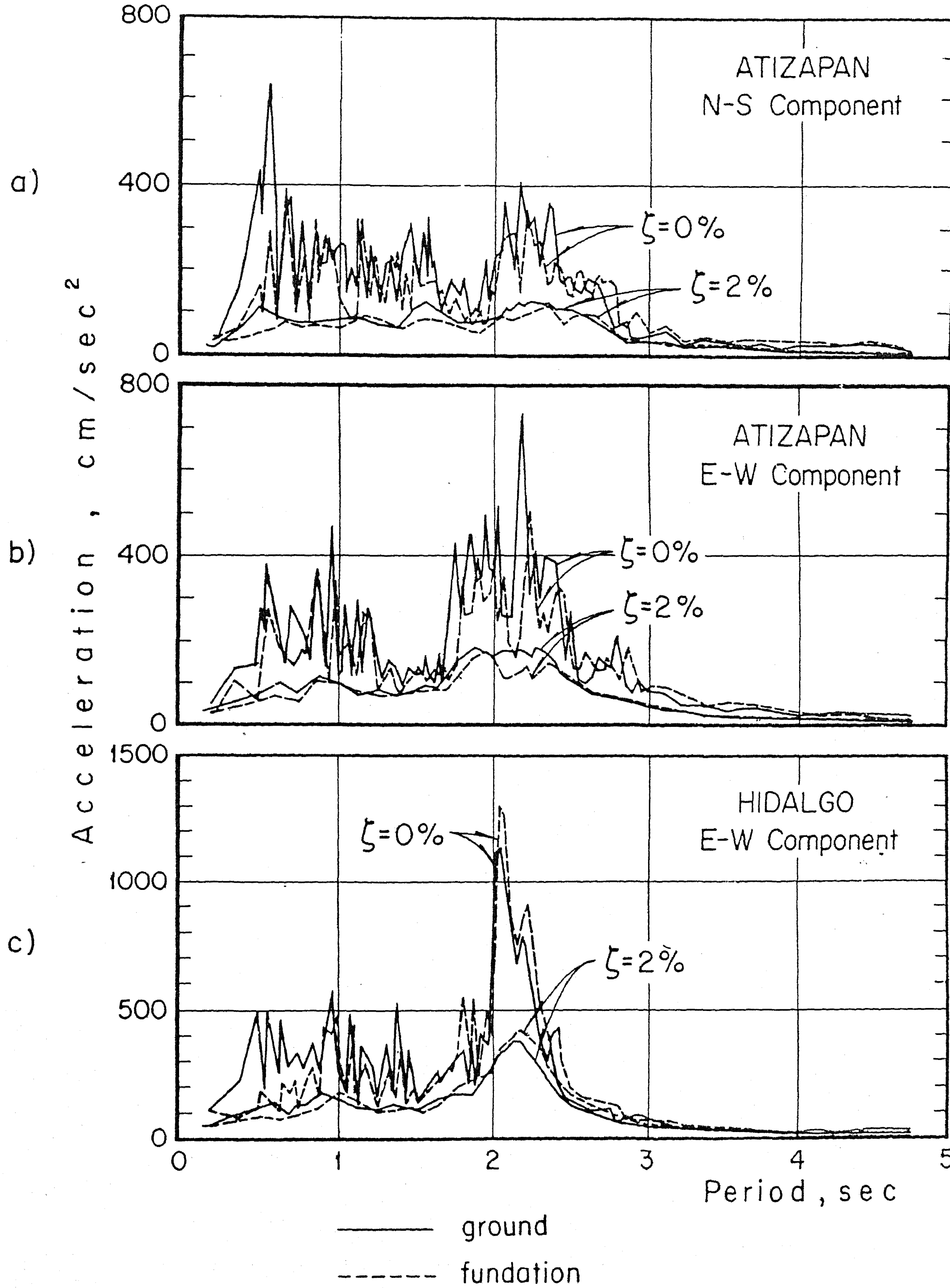
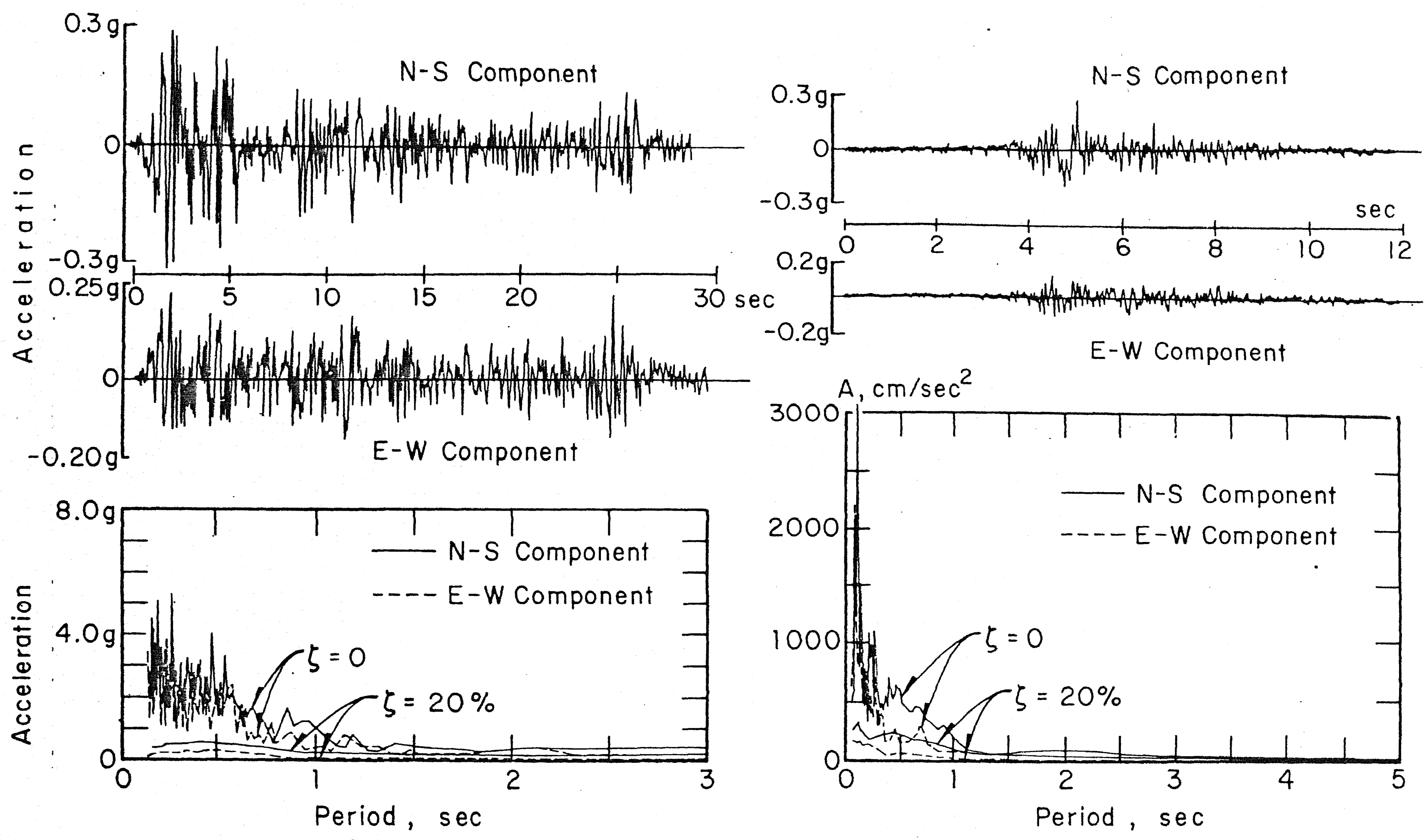


Fig 2 Comparison between spectra on the ground and on building fundations



a) El Centro, California, 18 may, 1940 b) Acapulco, México, 9 december, 1965

Fig. 3 Characteristics of motions on hard ground at short focal distances

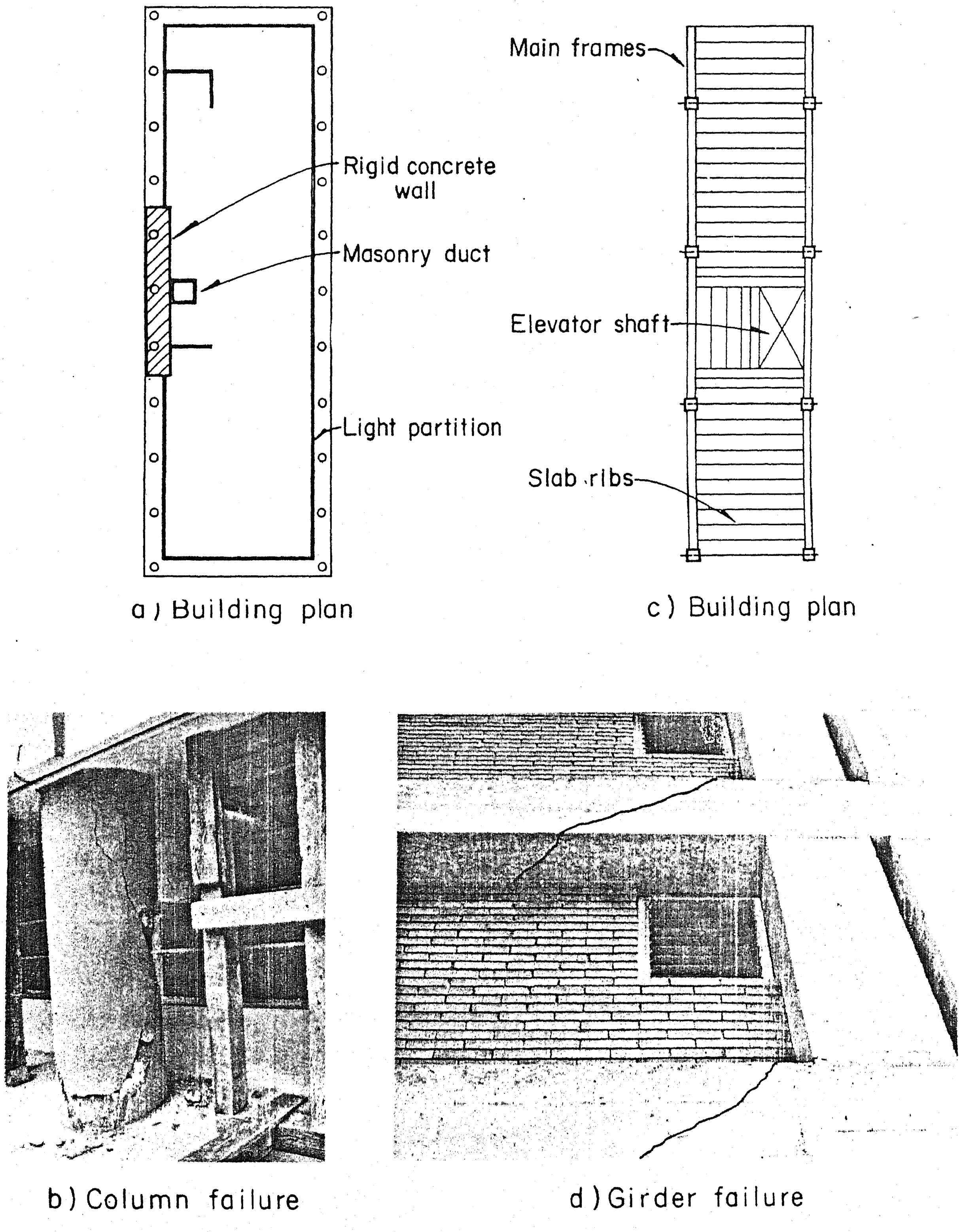
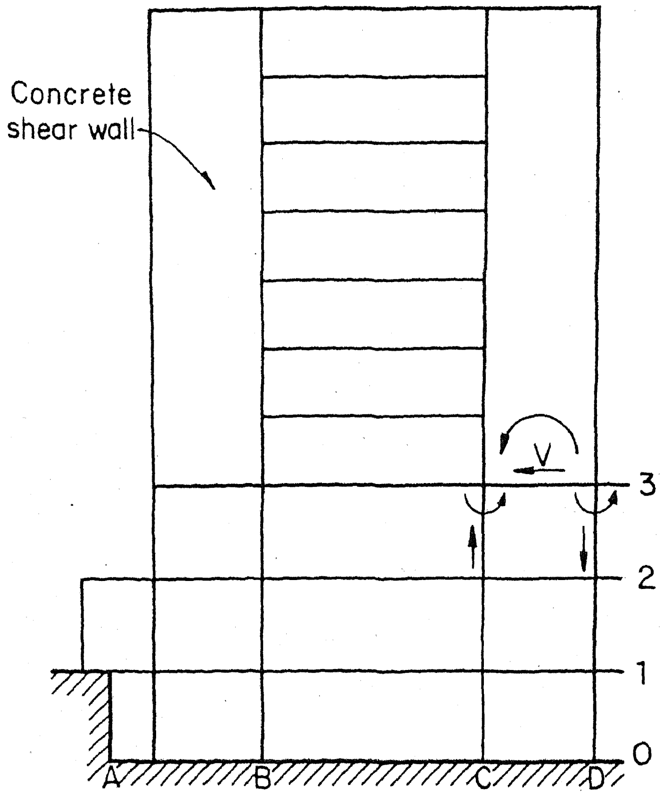
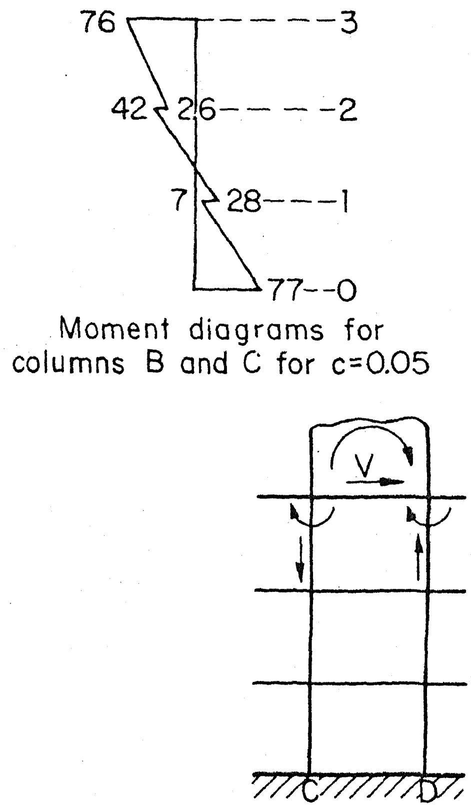


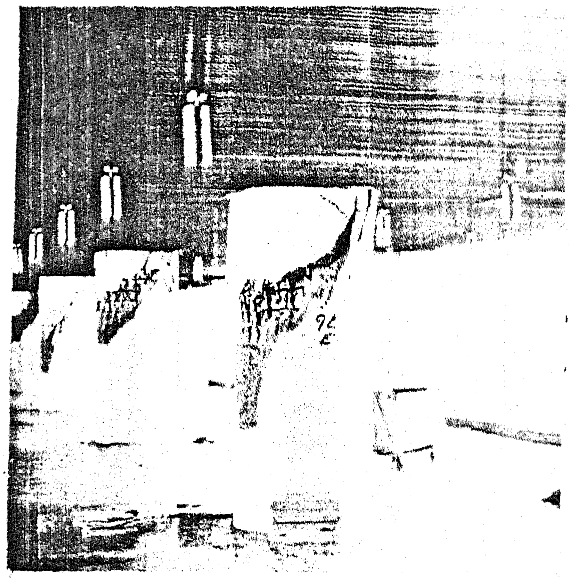
Fig 4 Torsional failure of concrete members



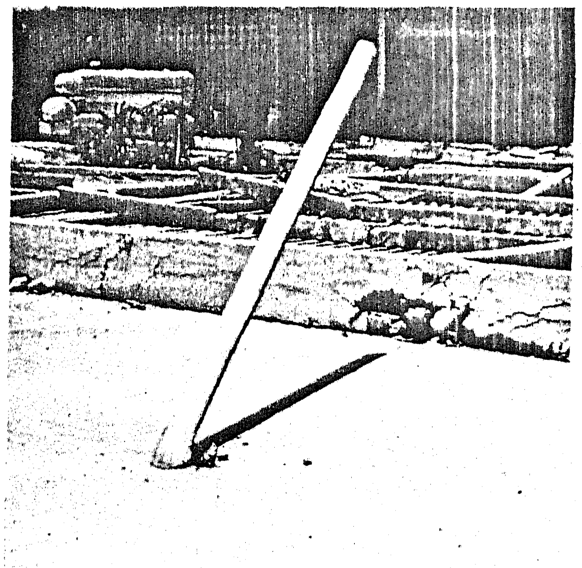
a) Structure of a hotel



b) Approximate analysis

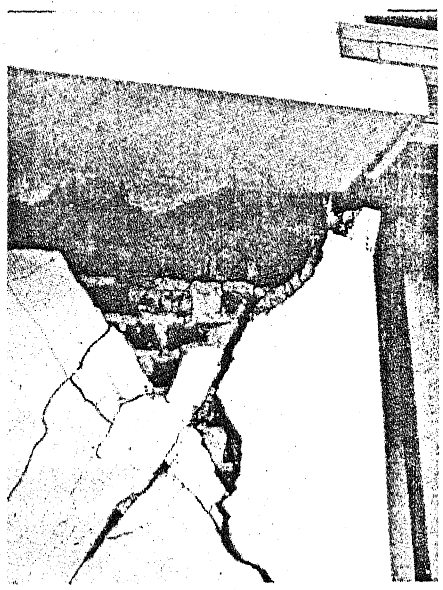


c) Brittle behavior of strong columns

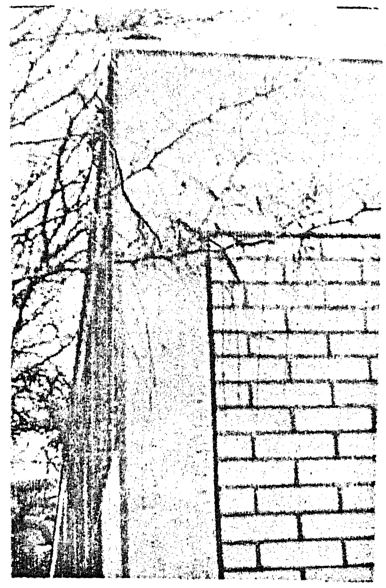


d) Instability of ductile, slender columns

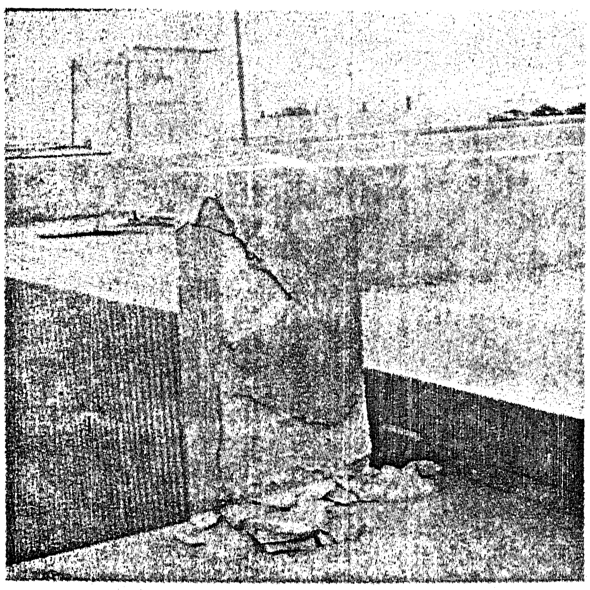
Fig 5 Ductility, strength and stiffness



a) Thrust of wall panels on frame corners



b) Failure of frames confining wall panels



c) Imperfect continuity at corners of reinforced concrete frames

d) Elimination of column ties at intersection with girders

Fig 6 Unadequate reinforcement at girder-column joints