PRESTRESSED CONCRETE EARTHQUAKE RESISTANT STRUCTURES - DEVELOPMENT, PERFORMANCE AND CURRENT RESEARCH

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Presented on behalf of the N.Z. Prestressed Concrete Institute

Introduction

Prestressed concrete has long been accepted in statically loaded structures. Thus for many years now we have seen the construction of prestressed concrete bridges, dams, runways, pipelines, reservoirs and various structures including more recently atomic reactor pressure vessels. These stand as irrefutable proof of engineers' confidence as to the integrity of this new material.

In recent years prestressed concrete has been used in seismic resistant structures. Just like any other new material, it will attract criticism and comment, sometimes by people who may not have had the opportunity of full investigation of the material in question. Furthermore, today engineers are more critical of any new material or technique and will seldom accept them unless conclusive evidence of their performance can be produced. This is as it should be.

The purpose of this report is to observe the application of prestressed concrete to seismic resistant multi-storey structures. However, this report should be read bearing in mind the fact that the widest application of prestressed concrete (to bridge and kindred structures) has been in the countries subject to earthquake and with operative seismic codes.

Synopsis

The report is divided into three sections.

1. Current practice. This section covers various buildings already constructed using prestressed concrete in some form together with an outline of the basis of design which was used.

2. The behaviour of prestressed concrete in some recent earthquakes. A description is given of the performance of some structures incorporating prestressed concrete in the earthquakes at Skopje, Anchorage and Niigata.

3. Current research into the properties of prestressed concrete under dynamic loading. Various research reports are commented upon.

The writer also offers certain conclusions and recommendations.

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SECTION 1 - Current Practice

Many thousands of structures have already been constructed in which prestressed concrete has been incorporated. Large frame structures have been wholly constructed in prestressed concrete and these have performed satisfactorily under normal static and wind loading.

However, only a limited number of buildings can be said to have been designed in accordance with seismic codes, and this section is therefore confined to buildings in countries which have such codes. The countries covered in this section are New Zealand, the West Coast of the U.S.A., Hawaii and Japan. There has also been extensive use of prestressed concrete in seismic resistant structures in Russia but unfortunately no details were available at the date of preparation of my report. It is appreciated that many buildings not covered by this report may well be fully seismic resistant by virtue of the type of construction or the choice of the designer who has, for his own peace of mind, allowed for such a contingency.

In general buildings are subdivided into three main categories:

(i) Structures in which prestressed concrete components are incorporated purely as flooring or secondary units but with main seismic resistant frame not in prestressed concrete. There are so many examples of this type of construction that it would be quite impracticable to enumerate them. It is sufficient to say that the use of prestressed concrete floor slabs, joists, beams etc. is now well accepted by the majority of engineers and there is no technical or economic justification for not using this type of component.

(ii) Buildings in which the main seismic resistance is provided by shear walls or towers of any type of material. This particular category of structure lends itself very favourably to the application of prestressed concrete. Many hundreds of structures have already been completed utilising prestressed flat slabs or combinations of prestressed beams and slabs with separate shear towers or shear walls providing the seismic resistance. Recently considerable interest has been shown in the use of prestressing tendons incorporated in the shear towers and walls.

(iii) Fully framed prestressed concrete seismic resistant structures. The main reason for the relatively slow acceptance of this type of structure is probably due;

(a) to the absence of information regarding the practical performance of such buildings in earthquakes, and

(b) the apparent shortage of information as to the properties of prestressed concrete in respect of energy absorption, damping activity and general performance under dynamic conditions.

However, I suggest that there is ample performance data to prove that the various properties of prestressed concrete are in fact ideal for a seismic resistant material, and I refer to this in greater detail in Section 3 of this report.

Approximately 50 fully-framed multi-storey buildings have been constructed to date in accordance with full seismic resistant codes. I will now examine certain buildings incorporating prestressed concrete in whole or in part. I will deal with these country by country as below.

A. NEW ZEALAND

In New Zealand there is a standard building code but it is not adopted by all local bodies, who often have their own building bye-laws. These bye-laws are subject to a variety of interpretations and this leads to confusion.

It is generally assumed by designers in this country that prestressed concrete without additional unstressed reinforcement is a material suitable for seismic resistant construction, and that the code shears for other forms of construction are applicable. There is at present no code of practice for prestressed concrete construction, although a draft code has recently been circulated for comment. All prestressing tendons have been grouted. The buildings described have not been subjected to major earthquakes.

DESIGN APPROACH

Framed Buildings

In framed buildings with prestressed beams and prestressed or normally reinforced columns (the latter used for higher buildings), the prestressing of the beams has generally been designed to resist gravity loadings, and the prestressing has been so arranged that the vertical dead and seismic live loadings are "balanced" by the upward loading produced by draped post-tensioned tendons, anchored in the columns. Under this condition of loading, the columns and beams are subjected to direct compressive forces only and are therefore able to resist the design lateral loadings applied in either direction.

In some buildings, the post-tensioning cables have been extended through the end columns and the anchorages for each beam encased in a concrete block. This has avoided congestion of reinforcing in the columns, and an interesting architectural effect has been obtained.

Cast in place and precast construction methods are used. In the case of precasting, mortar joints are generally used between the elements. Precast frames assembled from units and post-tensioned together have been assumed to be equivalent to similar cast in place frames. In some cases, provision has been made to prevent the mortar from dropping out of vertical mortar joints should large tension cracks occur during a severe earthquake.

In addition to simple frames assembled by post-tensioning beams and columns, several flat slab structures have been built in which frame action is provided by moment transfer between columns and slabs. Both solid and waffle slabs have been used.

Shear Wall Buildings

Concrete shear walls or shear cores with vertical post-tensioning have been used in a number of buildings. Post-tensioning has been considered to have advantages over normal mild steel reinforcing in that it provides for full length reinforcing tendons without the need for splicing at points of high tensile stress, and congestion of reinforcing in slender walls is reduced.

Other Uses of Prestressing

Several seismic resistant chimneys and towers have been built using vertical post-tensioning between precast elements. Horizontal prestressing has been used in a number of slab structures in which horizontal diaphragm action is required to transmit seismic shears to the vertical resisting elements.

Design of Sections for Flexure

Both elastic and ultimate strength methods have been used in the design of sections for flexure but the latter method has been considered to be more applicable. Maximum concrete tensile stresses under design earthquake moments (as compared with "ultimate" earthquake moments) have not generally been considered to be critical, since the limiting values will depend on the shape of the section (or of the mortar joint) and on the presence of existing cracks.

In ultimate strength design, the prestressing tendons on the compressive side of the section are assumed to lose most of their prestress due to the compressive strain of the surrounding concrete, so that the effect of these tendons on the ultimate resisting moment is small and can sometimes be ignored.

The flexural factor used in ultimate strength design in some framed buildings has been:-

$$U = 1.0\ (D + L) + K\ (E)$$

D = Dead Load

where L = seismic live load

E = Code earthquake load

A value for K of 1.8 has been used by some designers, but others have adopted higher values. A value for K of approximately 1.5 would be more comparable to the case of a reinforced concrete member subjected to seismic bending only (no dead and live load moments) and designed to the normal 33-1/3% elastic overstress. The use of a factor K = 1.8 has given apparent tensile stresses in excess of 700 p.s.i. in an uncracked I section subjected to the design earthquake moments. For a rectangular section, the apparent tensions have been higher.

The following are examples of prestressed concrete buildings in New Zealand.

(a) (Figure No.1)
Library building for Victoria University College, Wellington

This building, which will be completed at about the time of the Conference, is ten storeys high. The structural system consists of a factory precast waffle slab supported on normal reinforced columns on a 42 feet by 27 feet grid. The slab units are up to 60 feet long and 3 feet wide and are pretensioned with draped tendons. These units were assembled to form a two-way slab by post-tensioning transversely together with draped cables.

A rigid connection is formed between the columns and the slabs to provide framing action to resist seismic forces. Base shear was taken at 22%g. and members were designed on a basis of linear stress distribution and limiting stresses of 5,000 p.s.i. for concrete and 220,000 p.s.i. for steel. Members were further checked to be capable of rotation at an ultimate load four times that of design moments. Shear distribution to the frames was based on live load on each floor being concentrated in half the floor area. The building has an estimated first mode period of the order of 1 second and has been designed for instantaneous maximum accelerations at each level obtained from a dynamic analysis using an electric analogue with base accelerations equivalent to the El-Centro earthquake. A component of vertical accelerations of 20%g. was allowed.

Wall cladding, stairs and lift wall are precast and these and the partitions and windows are designed to permit the estimated interstorey deflections based on the design forces. The building was designed between 1959 and 1960.

(b) (Figure No.2)
Parking Building for Christchurch City Council

The 4 storey building at present under construction is 240 feet long and 126 feet wide; seismic resistance is provided by prestressed concrete frames in the transverse direction and by normally reinforced shear walls in the longitudinal direction. The building is in 2 structurally independent halves, each 60 feet clear span, linked by ramped slabs. Main beams and floor slabs are precast and pretensioned, but the columns are precast and normally reinforced with mild steel. The columns and beams are assembled into rigid frames by post-tensioning, the prestressing anchors being incorporated in the columns. Beam to column joints are mortared, and no mild steel passes through. The prestressing cables are spread uniformly over the depth of the beam at the column junction to obtain equal seismic resistance in both directions. A load factor method of design was used in calculating the seismic resisting moment of the prestressed beams.

(c) (Figure No.3)
Herbert Gardens, Wellington

The "Herbert Gardens" building is a 14 storey structure of equivalent height of 132 feet from foundation to top floor slab. The floors are prestressed flat slabs disposed to be dynamically balanced about the main seismic resisting component, a prestressed concrete central core structure.

Seismic design loads at working stresses are base shear 15%g. applied as a triangular distribution and reduced to 40 per cent for overturning moments. The fundamental period of the structure was estimated to be in the vicinity of 1½ seconds in the east-west direction and 1-1/8 seconds in the north-south direction. Prestressing the core was chosen to resist the uplift forces due to seismic action in the east-west direction. Design of core was checked for ultimate at factor of 1.5 and for first crack at 1.2. Design of slabs and slab to column connections were checked at working stresses for bending at a factor of 2.5 times working seismic loads. Torsion was calculated in the usual manner using the calculated lever arm increased by 1/20 the longest dimension of the building.

(d) (Figure No.4)
Parking Building for Williams Parking Centre Limited, Wellington

Figure No.4 shows a typical transverse frame of the 160 feet long building. Seismic resistance is provided by the frames in the transverse direction and by shear walls in the longitudinal direction. The building is cast in place and is normally reinforced except for the main beams which are post-tensioned. The transverse frames are 14" thick and spaced at 16'0" centres. The exterior columns are designed as simple props and are relatively flexible so that the beams shorten towards the rigid centre column during post-tensioning. In addition to the post-tensioning cables in the main beams, some mild steel is provided to allow for losses of prestress to adjacent rigid structural elements. This additional reinforcing was not assumed to contribute to the seismic resisting moment, which was calculated for the prestressing cables only, using a load factor method. This building is at present under construction.

(e) (Figure No.5)
Office Building for J. Yock & Company, Auckland

This small office building was designed in 1957 and built in 1958. 10%g. base shear was used and the members were designed for dead load plus live load upon 3 plus earthquake with linear stress distribution and limiting stresses of 4,500 p.s.i. for concrete and 200,000 p.s.i. for steel.

(f) (Figure No.6)
Newmarket Car Parking Building, Auckland

This building is a single storey structure with foundations and base columns designed for four storeys. The floor is a continuous insitu flat waffle slab of 14" overall depth stressed in both directions, with columns at approximately 34 feet centres. The frame design is the result of combining light flexible slabs with short stiff columns. Under live loads the columns tend to prevent any carry over of moment from one slab bay to the next. Under horizontal seismic forces the columns provide almost complete resistance from the frame action of the comparatively flexible slab. The main slabs act as plates to ensure that all columns deflect equally horizontally under seismic forces. The vertical stressing in the columns, assisted by the dead load of the building itself, induces a uniform prestress over the column sections sufficient to resist the reversible stresses in both directions from seismic forces. The foundation tie beams were reinforced concrete and designed to accommodate column base moments and seismic shear forces.

(g) (Figure No.7)
Naval Workshops, Auckland

The structure consists of rigid frames of two 40 feet spans with a height to eaves of 37 feet with provision for travelling cranes. The columns, which were precast, used high grade concrete and high tensile steel, this proving more economical due to the large reversals of stress due to wind governed lateral forces. Reasonably high ductility was aimed at. The beams were prestressed with one cable before erection and post-tensioned in position to give the continuity required at the joints and added strength to the beams. The purlins and girts were also of prestressed concrete and were bolted to the frames.

In New Zealand approximately 150 buildings incorporate prestressing in some form and 30 of these are framed structures. Interesting examples other than those described above are Clyde Quay Overseas Passenger Terminal Wellington, TEAL Workshops Mangere, Tasman Hotel New Plymouth, C.P.D. Building Wellington, Albert Street Parking Building Auckland, and the Dunedin Swimming Pool.

In addition to these structures, prestressed concrete is also being used in this country for major bridge works, and a fine example of this is the Newmarket Viaduct at Auckland (Figure No.8) which is at present under construction. Although it is not a multi-storey building it would be unrealistic to omit any reference to this structure in a report which dealt with the use of prestressed concrete in seismic resistant frames. The Viaduct is 2,406 feet long and 88 feet wide and is designed to carry six lanes of traffic across the borough of Newmarket. The superstructure consists of prestressed box girders 8 feet deep. There are four expansion joints in the length of the bridge. Span lengths vary up to a maximum of 200 feet, and maximum height of piers is 70 feet clear. Each pier consists of two 10 feet by 4 feet hollow columns, 43'6" apart on centres. These piers are framed into the superstructure and hinged at their bases.

The design makes provision for earthquake on a dynamic basis. The Viaduct is anchored generally in rock, so response spectrum technique was used. Studies were made of probable magnitude of 100 year 'quake for design purposes and Richter 6.7 was adopted and the El Centro pattern was used in design. Assuming a dominant pier height of 60 feet, the following figures were obtained:-

Natural period	1.6 seconds
Spectrum velocity	0.49 ft/sec.
Relative displacement	1.5 inches
Absolute acceleration	0.06G.

B. WEST COAST, U.S.A.

The codes in current use in the United States of America are similar to those used in New Zealand.

Section 2313 (j) has been a most controversial section of the American code and is undoubtedly responsible for the limited number of prestressed concrete structures over 13 storeys high. The wording of this section is somewhat arbitrary, and indicates probably a lack of understanding of the true nature of the problem of energy absorption. Research has been conducted since the adoption of this sub-section and in view of testing programmes on rigid joints including the reversal of stress and works in limit design and plastic design, an enlarged concept of ductility and energy absorption will soon emerge. As soon as sub-section (j) is modified to permit higher concrete buildings many of the high rise prestressed concrete structures which are already in the planning and design stages will be built in California and the other west coast States. At present there are on the west coast less than 10 prestressed buildings in excess of 13 storeys high, but 20 to 40 prestressed concrete buildings with 5 to 13 storeys. All of these are designed for zone 3 lateral loading. Most of these buildings are shear wall structures. Generally they consist of precast prestressed horizontal and vertical elements and poured in place diaphragms. In most cases the floors are designed to ensure composite action of precast elements with poured in place concrete. Continuity generally is provided by mild steel reinforcing, although post-tensioning is becoming popular also. The following are examples of prestressed seismic resistant buildings on the west coast of the United States.

Figure No.9 - Kaiser Foundation Hospital Parking Structure, City of Los Angeles
6 storey garage with over 200,000 square feet of precast, pretensioned double tees, girders and columns, designed for two additional future storeys.

Figure No.10 - Stivers Investment Co. Office Building, Pasadena
5 storey office building with precast, pretensioned double tees and full length tilt-up walls.

Figure No.11 - Beverly Hilton Hotel Parking Structure, Beverly Hills
6 level garage with over 200,000 square feet of pretensioned double tees. Structure, except for elevator shafts and ramps, is completely precast and factory produced.

Figure No.12 - Court Professional Building, Downey
4 storey office building with precast, prestressed floors featuring 52 feet high and 8 feet wide pretensioned double tees used for the first time as wall panels (seismic shear walls).

Figure No.13 - Collins Electronics Plant, Long Beach
Precast walls, pretensioned 119 feet clear span, 8 feet wide double tees.

Figure No.14 - Beverly Hills Parking Structure No.2
5 storey, precast rigid frame building with 75 feet clear spans - post-tensioned connections.

Figure No.15 - Administration Building, University of California at Davis
9 storey, precast prestressed channel slabs; precast, prestressed columns; precast wall panels.

Figure No.16 - Shoreham Towers, Los Angeles
16 storey, cast-in-place, post-tensioned flat slabs.

Figure No.17 - 600 East Ocean Apartments, Long Beach
15 storey, cast-in-place, post-tensioned flat slabs. Post-tensioned core walls for earthquake forces.

Notwithstanding the adverse effect of section 2313 sub-section (j) of the code, the development of prestressed concrete on the West Coast of the U.S.A. has been spectacular. This has been brought about by the enthusiasm of a relatively small band of prestressing engineers combined with the enormous resources of large precasting factories and a demand for rapid construction.

C. HAWAII

The seismic code for the City of Honolulu is of course identical with that of the west coast of the United States of America, except that the controversial section 2313 sub-section (j) is not followed. Probably stimulated by the boom in construction and the proximity of the west coast of the United States, there has been extensive application of precast prestressed concrete to seismic resistant structures.

The following are typical buildings in Hawaii incorporating prestressed concrete.

Figure No.18 - Ilikai Hotel

Figure No.19 - Ala Moana Building

Figure No.20 - East West Centre

D. JAPAN

Many buildings in Japan incorporate prestressed concrete in some form. Several of them have been subjected to earthquake forces such as that experienced in Niigata in 1964. It is pleasing to record that these buildings performed well.

However, the national building code of Japan tended to restrict the use of prestressed concrete for major seismic frames though a preliminary code of practice for prestressed concrete in seismic structures has been in operation for some years and is currently being modified to move into line with modern practice. Notwithstanding this, however, many structures including railway workshops and administration buildings, and in particular the canopy roof at Niigata railway station, have been built making extensive use of prestressed concrete. Two interesting examples are the Gozu City Public Office (Figure No.21) and the Saitama Prefecture Agricultural Hall at Urawa City (Figure No.22).

SECTION 2 - The behaviour of prestressed concrete in some recent earthquakes

Skopje, 1963

Several buildings in Skopje incorporated prestressing in some form, but no failure of any actual prestressed concrete member has been reported. One structure incorporating prestressed concrete tie beams at the base of a reinforced concrete dome suffered complete collapse due to failure of the supporting structure. The tie beams themselves did not fail and when they were eventually broken up it was found that the bonding of the fully-grouted cables in the tie beams had been satisfactory. It is also interesting to note that large prestressed concrete electric transmission poles behaved satisfactorily and suffered no damage.

Alaska 1964

The following is based on my own observations during a two-day visit to Alaska some six weeks after the earthquake. No doubt other investigators will be publishing their reports which deal with this subject more fully, but I will record my own observations on the performance of some 27 buildings which incorporated prestressed concrete.

Of these, 13 suffered no damage other than very minor cracking of blockwork. Examples of these undamaged prestressed concrete buildings are the National Public Safety Building (Figure No.23) and the National Bank of Alaska (Figure No.24). Both these buildings utilised prestressed concrete tees and columns as frames.

However, there were a number of buildings which failed and which incorporated prestressed concrete. Apart from the Four Seasons Apartment building which I will refer to at greater length later, there were no failures of the prestressed concrete members themselves. In nearly every case the supporting structure, which was built of traditional materials, collapsed or connections and bearings were ineffective (see Figure No.25 and Figure No.26). Four buildings collapsed completely due to the failure of the supporting structure and three buildings suffered serious damage due to bad detailing of connections. One building under construction suffered partial collapse (bad connection details were evident). The main points which could be observed with regard to the functioning of the prestressed tees in these buildings were as follows.

(a) Inadequacy of weld plate shear connections between units. There were many instances where site welding of weld plates, bearing plates and other connections failed, and it would appear that site welding is very susceptible to non-ductile failure due either to faulty workmanship or the heat treating of the adjoining steel. Other forms of connections capable of providing a ductile unit seem preferable. Failure of one connection will normally cause greater stress on the others, and this is likely to induce progressive failure. Thus every connection must be a good one.

Unsatisfactory attachment of weld plates to the prestressed unit (See Figure No.27). The anchor rods holding the weld plates into the unit tended to pull out of the very thin flanges of many tee-sectioned units. These flanges were approximately 1½" thick and it would appear that this thickness is inadequate for good fixing of shear connections.

In many cases the anchor rods literally pulled right out of the concrete or large pieces of the flange broke off as soon as the weld plates were subject to bending.

(b) Need for good quality cast insitu concrete topping. Many of the structures incorporating prestressed tees without a cast insitu topping with a suitable steel mesh failed. Toppings with suitable mesh can provide the necessary medium for transmitting seismic forces.

(c) Inadequate provision for seating in connection of units to supporting structures (Figure No.28). Many failures in Alaska were attributed to inadequate seating.

In general the prestressed units that were incorporated in the buildings in Alaska had been designed so that the theoretical forces due to earthquake were allowed for. Unfortunately the actual forces encountered during the earthquake were considerably greater than the estimated values, with the result that those connections which were not capable of withstanding considerable excess either in the elastic or plastic range failed.

4. Four Seasons Apartment Building; this structure consisted of six lifted prestressed slabs with horizontal seismic forces carried by lift and stairwell towers. Steel columns were encased in a lightweight fireproof material. The slabs were prestressed with ½" unbonded cables. The whole building collapsed completely during the earthquake. Whilst it is difficult to decide exactly how the structure first failed, the following are my personal observations (Figure No.29).

(i) The reinforced concrete shear towers appear to have crushed at their base on one side and thus allowed the whole structure to fall endwise under the effect of the horizontal forces. The towers were supported on a reinforced concrete foundation which also served as a basement car park. This part of the structure was still intact and was being used for storing materials. On examination of the shear towers which had collapsed, it appeared that there was insufficient mild steel and stirruping to restrain the concrete under the ultimate load. It was evident that the stiffness of the towers was not equal due to unsymmetrical placing of the door openings, and this no doubt caused one tower to draw extra load to it.

(ii) The prestressed slabs themselves appear to have failed before the complete structure collapsed to the ground, and this could be ascertained from the placement of the slabs as they lie (Figure No.30) and also from the fact that many of the prestressing tendons which are ½" unbonded cables flew out of the slabs while the structure was still in the air (Figure No.31). In fact, some of the strands actually penetrated the roof of an adjoining house, and this could only have occurred before the slabs had fallen to the ground.

It will be appreciated that as soon as the cables have lost their prestress, the resistance of the slabs is lost and the connection between the slabs and the columns is destroyed, with the result that there is nothing to prevent the slabs sliding down the columns. There is evidence to show that this did occur. (See Figure No. 32).

There also appeared to be an absence of shear head reinforcement round the collars. It is possible that the reinforced concrete pour strip between the slabs may have failed causing the cable anchorages to be destroyed and the prestress thus released. However, this does not account for the fact that the transverse cables also failed and can be seen protruding from the sides of the slabs. Many of the anchorages became detached from the concrete and it appears that the wedges failed at less than the ultimate strength of the wire. Wedges were found several blocks away and prestressing cables were thrown across the adjoining street. This would have been avoided had grouted cables been used. The failure of the connection of the prestressed slabs to the shear towers is most noticeable, and eyewitnesses report that the slabs were seen to slide down the columns and towers before the main structure collapsed. This again indicates the need for careful detailing of the connections of all units in a structure.

Recent reports from Alaska suggest that the final welding of all collars to columns had not been completed at the time of the Four Seasons Apartment collapse. This would reduce any frame action between slabs and columns, but it is difficult to see how it would have affected failure of the slabs and the collars.

The lessons of the Alaskan earthquake are many. There is obviously a need for closer attention to connections and bearings and supporting structures, and the use of unbonded prestressing cables without doubt contributed to the failure of the Four Seasons Apartment Building.

Niigata, Japan, 1964

The following is based on my observations during a short visit to Niigata the day after the earthquake.

Most of the major structures in Niigata are reinforced concrete or encased structural steel. Only 3 structures incorporated prestressing to any material degree, and I devoted as much time as possible to a study of these. Two were bridges and in both cases the prestressed members performed excellently. One is 660 feet long with a 24 feet carriageway with 66 feet spans over the Shinano River. The piers were cast insitu on precast piles with a deck consisting of precast beams post-tensioned longitudinally and transversely through the diaphragms of the deck. All tendons were fully grouted and bonded, and all beams were adequately connected to the piers against horizontal forces. Despite considerable settlement and displacement of the foundations of the abutments and possibly of the river bed, the bridge has suffered no visible damage at all.

The other bridge is one of which there have been so many photographs in the Press. This bridge has a structural steel centre span, the approach spans consisting of prestressed I beams with a cast insitu reinforced concrete deck. There was considerable movement of the piers and because the centre span was not properly secured, it pulled off the supports and collapsed over the railway line.

The third structure - which appeared to be the only building in Niigata incorporating prestressing - is the railway station. Here the entire canopy over the foyer is of post-tensioned precast members on precast columns and beams assembled with mortar joints and grouted post-tensioned cables. The whole building is apparently founded on piles and although the foundation material appears to have subsided by two feet, the structure itself has withstood the movement exceedingly well and no damage is reported to the prestressed members.

Apart from these three structures in Niigata, the numerous other buildings and hundreds of bridges incorporating prestressed concrete in the rest of the area affected were subjected to varying forces during this earthquake, but no damage has been reported.

SECTION 3 - Current Research into the properties of prestressed concrete under dynamic loading

Whilst the behaviour of prestressed concrete elements under combined horizontal and vertical loadings of a static nature is completely accepted, many engineers may be unaware of the experimental work recently carried out to determine behaviour under dynamic loadings. A fuller understanding of this research will, I believe, assist us in the preparation and application of more realistic building codes. Most building codes today base their rules on empirical constants or factors to allow for what appear to be damping characteristics of particular types or shapes of buildings.

Research work which has been undertaken recently is that by

(i) Dr. Nakano on two and four storey structures (reference 1 & 2).
(ii) Mr. Oladapo on true dynamic loading of prestressed concrete beams (reference 3).
(iii) the programme at present being carried out by Mr. Spencer at the University of Auckland, and
(iv) Professor Penzine on the damping behaviour of prestressed concrete (reference 4).

Similar work is also being carried out in Russia but unfortunately no details were available at the date of preparation of this report.

Dr. Nakano's first experiments were on three prestressed concrete portal frames which were loaded horizontally. He measured ductility and resistance against lateral force, and he examined the dynamic behaviour of the frames. As a result of his experiments he concluded that:

i. Ultimate design method is applicable for prestressed concrete portal frames.
ii. Deterioration of deflection characteristics under repeated loading is negligible.
iii. Ample ductilities are guaranteed.
iv. Increments of periods of natural vibration, and of fractions of critical damping, in accordance with increments of load, are small.

Figures Nos. 33 and 34 show two pages extracted from this report which are of particular interest.

Dr. Nakano then went on to conduct tests on a prestressed concrete four-storey structure. He concluded from these tests that:

1. Statically indeterminate structures having sufficient ductility can be designed for dynamic loading in prestressed concrete.
2. Prestressed concrete structures have elastic deflection characteristics even under several times the designed horizontal force. Practical criteria for judging the soundness of prestressed concrete structures should be ductility in the plastic range and relative deflection at the moment the structure enters the plastic range.

3. To ensure ductility of the frame, it is necessary to:

(a) ensure good design of joint panel
(b) to prevent premature formation of plastic hinges in the column.

4. Properly designed prestressed concrete structures having proper finishings may suffer smaller damage than reinforced concrete structures under the same intensity of earthquake. He also suggests that a prestressed structure would be more economical than a reinforced concrete structure when designed to the same degree of damage.

5. The above conclusions are based on the assumption that structures behave monolithically and that the prestress is permanently maintained.

The following points should be noted.

(a) Care must be taken to ensure rigidity of the slabs to provide diaphragm action to ensure that they behave monolithically with the frame.

(b) Rupture of the prestressing tendons must be avoided and prestressing steel having sufficient elongation at rupture is essential. All cable tendons should be grouted.

(c) Torsional effects of members should be considered.

In general Dr. Nakano's experiments provided sufficient evidence of performance of prestressed concrete under dynamic loading. However, there was obviously need for further study of energy absorption under actual dynamic loading rather than that calculated from static hysterisis loops. Such further study was in fact carried out by Mr. Oladapo during a series of experiments at the engineering laboratory in the University of Cambridge. Oladapo found that under dynamic loading, prestressed concrete considerably increased its capacity for sustaining elastic deformation and for absorbing energy by inelastic deformation. I believe that Oladapo's research is most significant and I reproduce his concluding paragraph in full.

"The characteristics of prestressed concrete under dynamic loading differ considerably from those under static loading. Under dynamic loading prestressed concrete is much more elastic and has a greater capacity for energy absorption than under static loading. Dynamic loading also causes significant increases in the ultimate moment, the cracking strength and the curvature at rupture."

Further work is also being carried out as part of a doctorial thesis by R.A. Spencer at the School of Engineering, University of Auckland, under the supervision of Professor N.A. Mowbray. Tests are being made on concrete members of varying types, both reinforced and prestressed. A periodic load is applied to the member. This can be made large enough to cause it to fail. The frequency of the applied periodic load is in the range often encountered in practice when buildings are subjected to a seismic loading, i.e. .5 to 5 cycles per second.

The damping energy of the test member is noted directly so that no assumptions of viscous damping or elastic behaviour are necessary. This makes it possible to investigate the effect on damping energy of such parameters as: the magnitude of concrete stress, the volume stressed and the stress distribution, the arrangement of reinforcing steel, the bond area and the amount of prestress, if any, concrete cracking, deformation beyond the elastic limit and the rotation of hinges in areas of high stress. The load capacity of the test members under dynamic loading can be found and compared with the values obtained from the static tests which are also being made.

The specimens being tested initially are beams 6 inches by 4 inches by 140 inches, either reinforced or prestressed concrete with steel areas, steel placement and total bond area and prestress, if any, being varied. Concrete quality is being kept constant. Prestress will be applied, with small diameter tendons approximating scale proportions of normal construction.

Uniform periodic bending moment is applied over a test length between 60 inches and 120 inches, by applying equal and opposite end rotations either end of the test length. Bending moment has a maximum value of $\pm$ 150,000 lbs. inches. Both the amplitude and frequency of the applied moment can be varied without stopping the machine.

The applied end rotations R and the corresponding bending moment M are both measured continuously and the MvsR hysterisis loop is displayed on an X-Y oscilloscope and recorded photographically. The area of this loop is directly proportional to the damping energy per cycle of the test beam.

The tests completed at the date of preparation of this paper show that useful information about the load deflection properties and damping energy of concrete beams can be found. It is hoped in the future to test beams under other kinds of applied load, simple assemblies of members, and individual joints of various kinds. From the prediction of the load deflection properties and total damping energy of the members it is hoped that analytical methods for the calculation of the dynamic response of structures may be possible. It is also hoped that the actual mechanism by which damping occurs will be understood. Some progress has already been made in this direction.

It is regrettable that in this age of efficient lines of communication, a greater effort is not made to co-ordinate research and development in such fields of engineering. One of the aims of the F.I.P. Commission on Seismic Structures is to seek out any research which is either being undertaken or proposed to be undertaken and to assist in every way possible in the co-ordination and publication of such research. By this process it should be possible to expedite the development and perhaps the acceptance of prestressed concrete into seismic resistant structures.

CONCLUSIONS

EXPERIENCE	* Dynamically loaded	Bridges, Sleepers, Forge Foundations etc.	These structures have been adequately tested under all conditions and proven satisfactory
	* Seismic Resistant Structures	Shear wall buildings, Bridges etc.	
RESEARCH	* Tests Completed	Indicate: Ductility, Energy Absorption, Damping	PRESTRESSED CONCRETE MULTISTOREY FRAME STRUCTURES WILL PROVE EFFICIENT and ECONOMICAL
	* Tests Required	To prove: Behaviour of prestressed structures under simulated seismic loadings	
DESIGN	* Revision and establishment of codes on the basis of dynamic loading		
	* New conception of design of structures taking into account the specific behaviour of prestressed concrete		
EXECUTION	* Greater understanding of construction methods and special attention to such details as jointing connections and grouting of cables		

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7. Lin : "Design of prestressed concrete buildings for earthquake resistance", A.S.C.E.-P.C.I. Joint Conference, San Francisco, October 1963.

8. P.C.I. "Prestressed Concrete in Multi-storey buildings", Technical Session II, P.C.I. Convention, September 1964.

Acknowledgments

The author wishes to thank Mr. K.E.Williamson for assistance in the preparation of the New Zealand report, Mr. S. Galezewski and Professor T.Y. Lin for details of the American report, and Dr. Nakano for permission to reproduce sections of his research reports.

The author also wishes to thank those engineers who have so readily assisted in the supply of information regarding various projects.

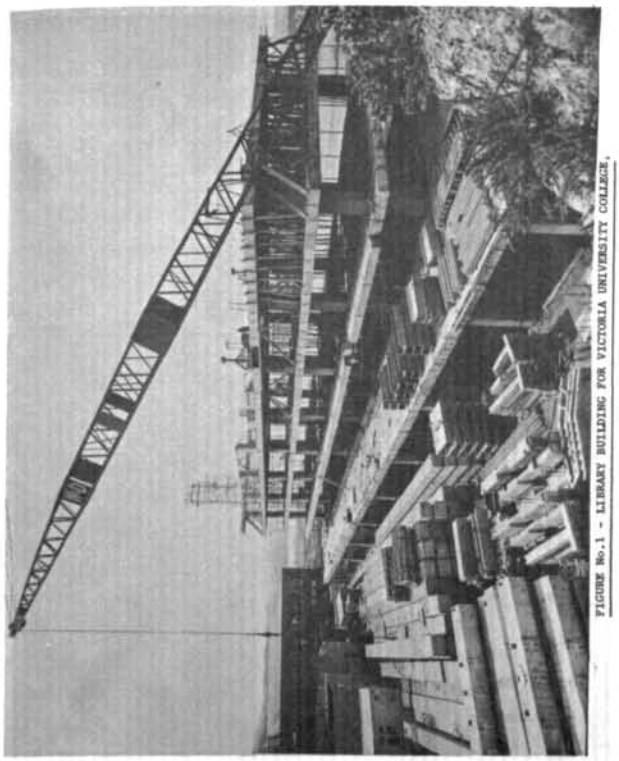


FIGURE No.1 - LIBRARY BUILDING FOR VICTORIA UNIVERSITY COLLEGE, WELLINGTON

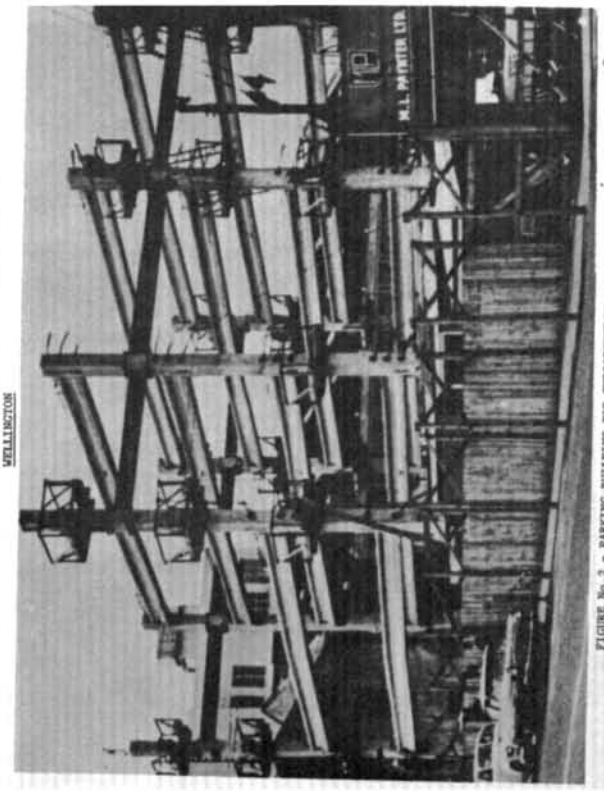


FIGURE No.2 - PARKING BUILDING FOR CHRISTCHURCH CITY COUNCIL

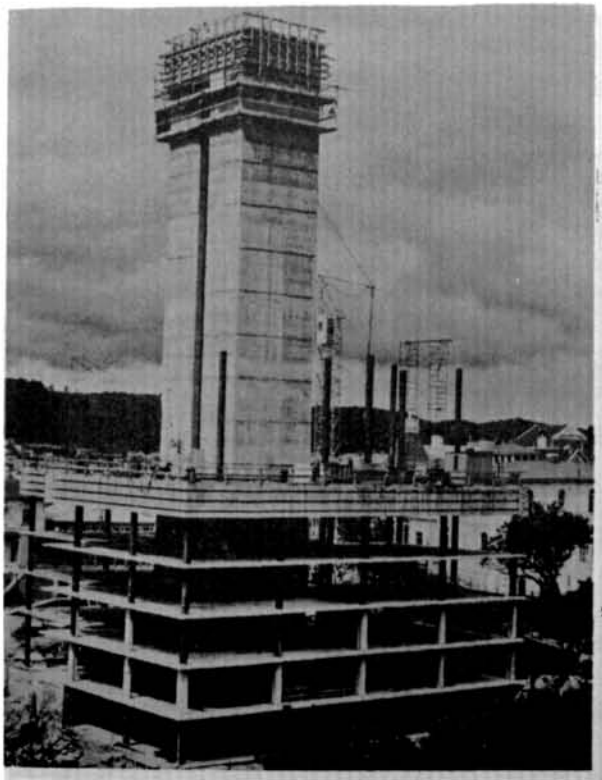


FIGURE No.3 - HERBERT GARDENS, WELLINGTON

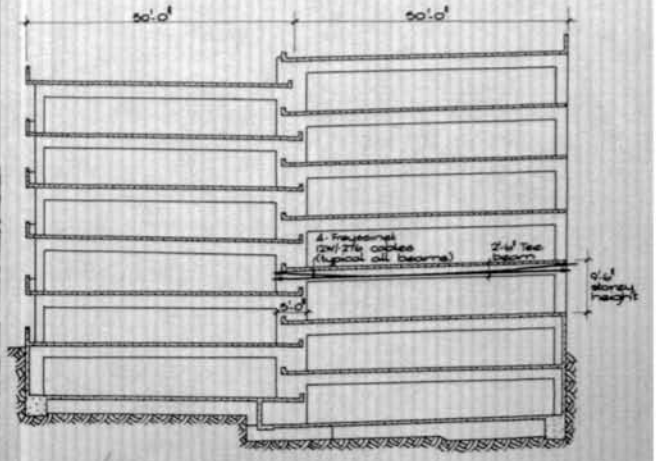


Figure No.4 - Parking Building for Williams Parking Centre Limited, Wellington

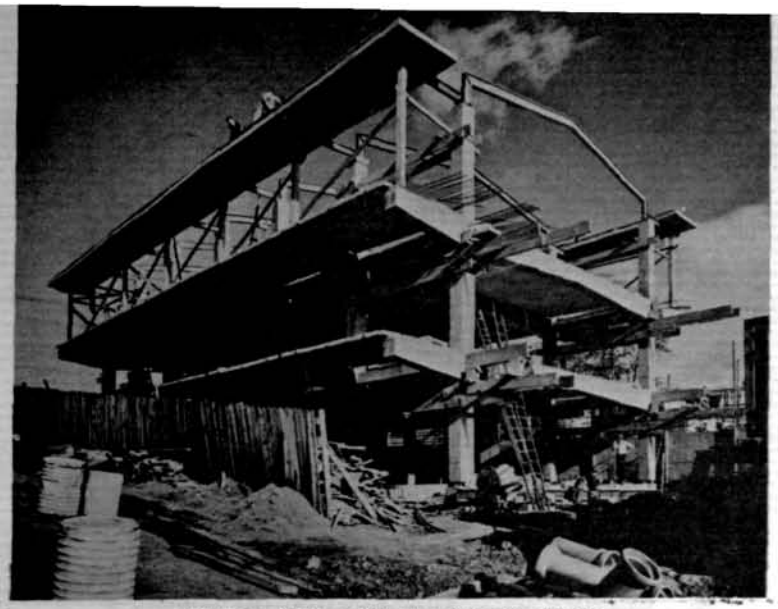


FIGURE No.5 - OFFICE BUILDING FOR J. YOCK & CO., AUCKLAND

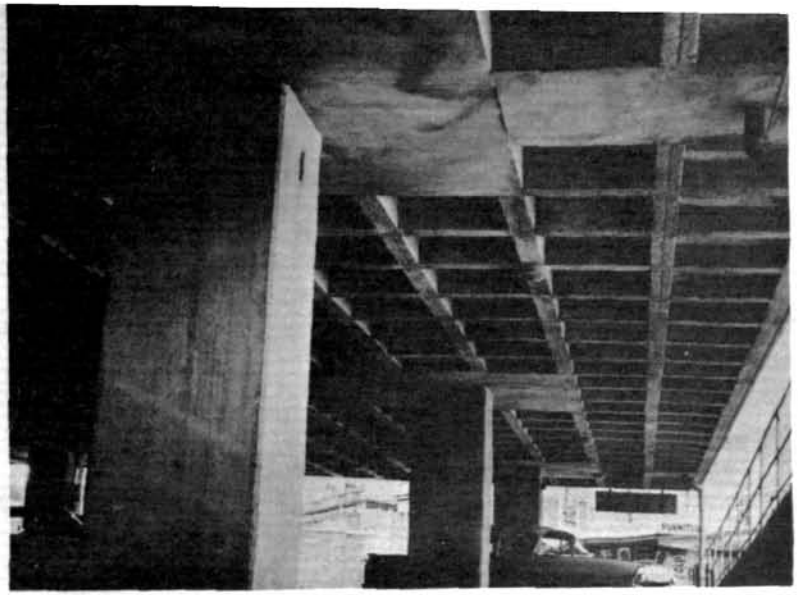


FIGURE No.6 - NEWMARKET CAR PARKING BUILDING, AUCKLAND

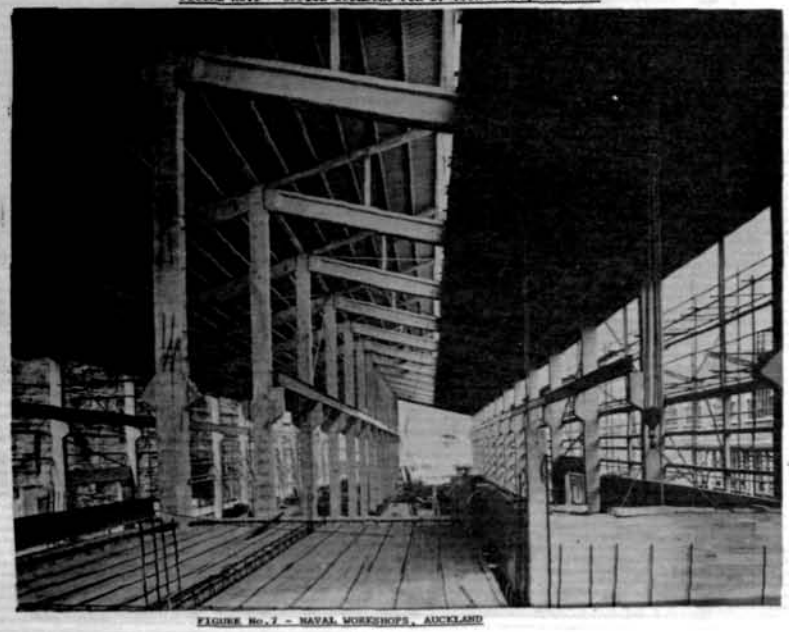


FIGURE No.7 - NAVAL WORKSHOPS, AUCKLAND

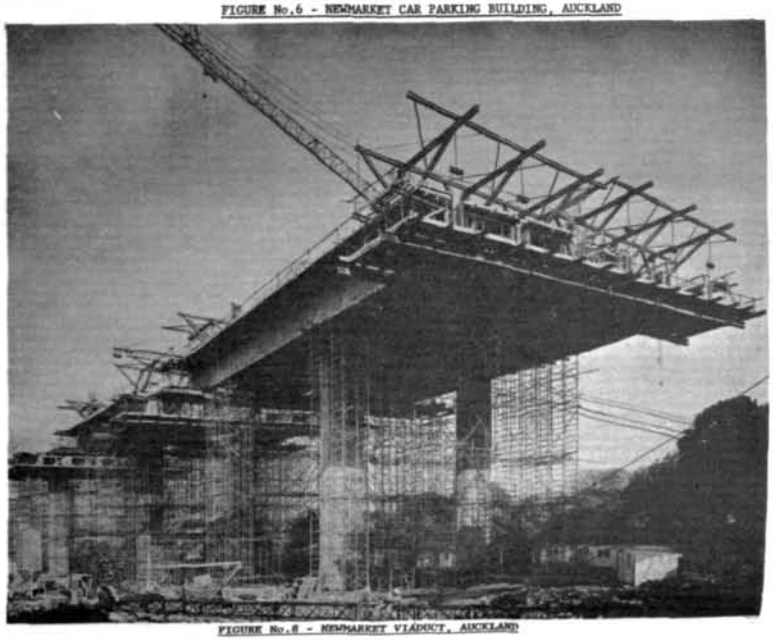


FIGURE No.8 - NEWMARKET VIADUCT, AUCKLAND

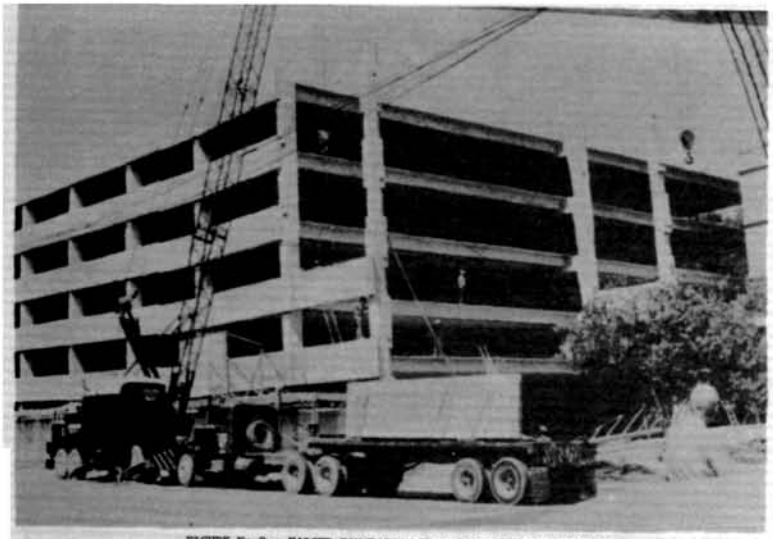


FIGURE No.9 - KAISER FOUNDATION HOSPITAL PARKING STRUCTURE, CITY OF LOS ANGELES

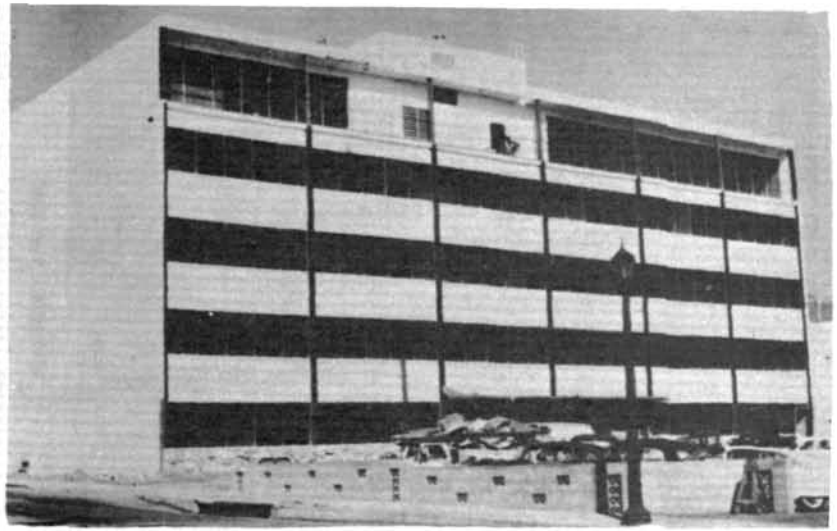


FIGURE No.10 - STIVERS INVESTMENT CO. OFFICE BUILDING, PASADENA

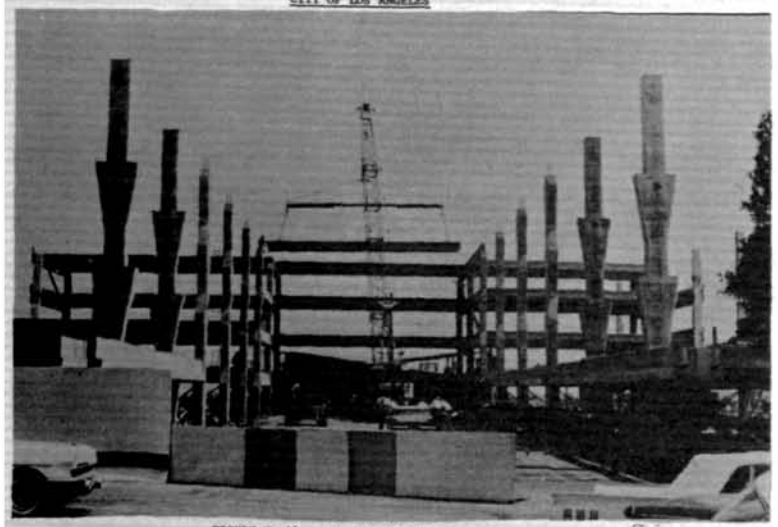


FIGURE No.11 - BEVERLY HILTON HOTEL PARKING STRUCTURE, BEVERLY HILLS

FIGURE No.12 - COURT PROFESSIONAL BUILDING, DOWNEY

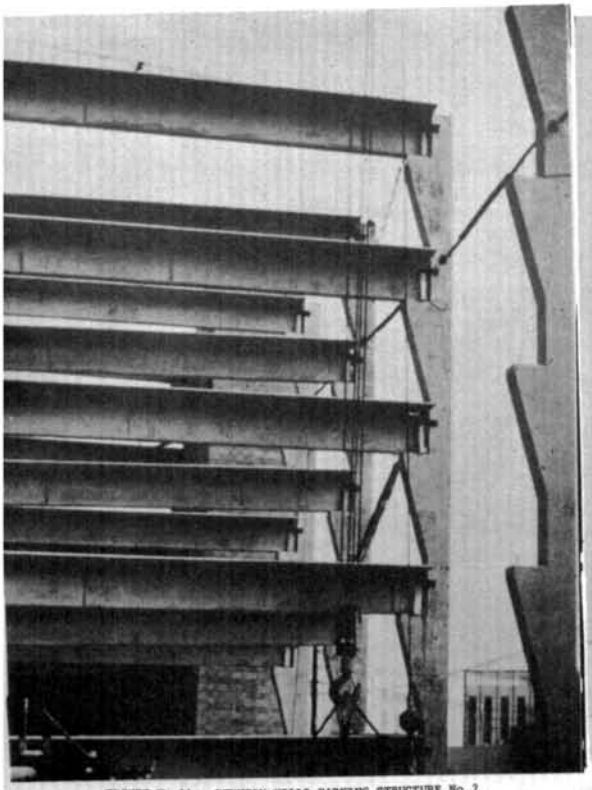


FIGURE No.14 - BEVERLY HILLS PARKING STRUCTURE No.7

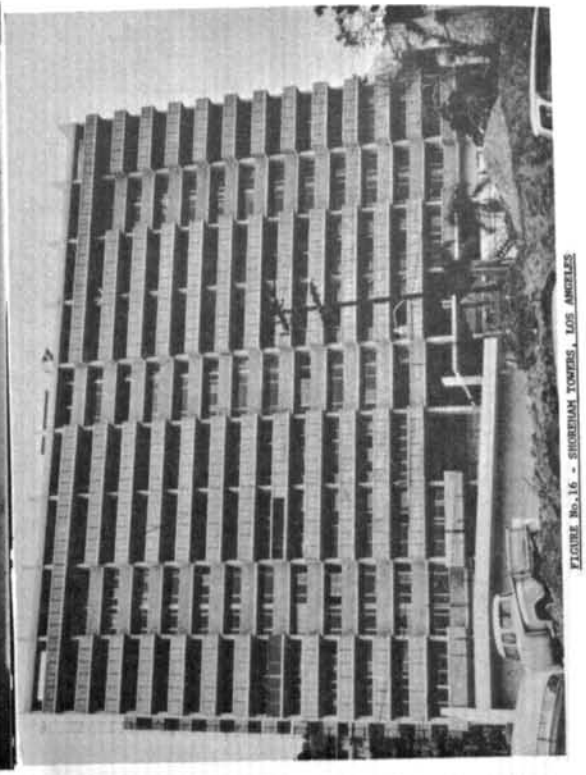


FIGURE No.16 - SHOREHAM TOWERS, LOS ANGELES

FIGURE No.13 - COLLINS ELECTRONICS PLANT, LONG BEACH

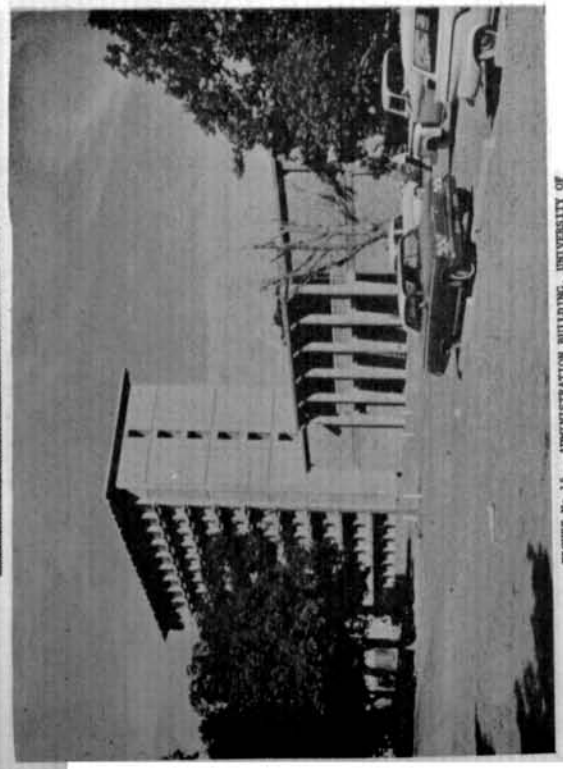


FIGURE No.15 - ADMINISTRATION BUILDING, UNIVERSITY OF CALIFORNIA AT DAVIS

FIGURE No.18 - ILIKAI HOTEL

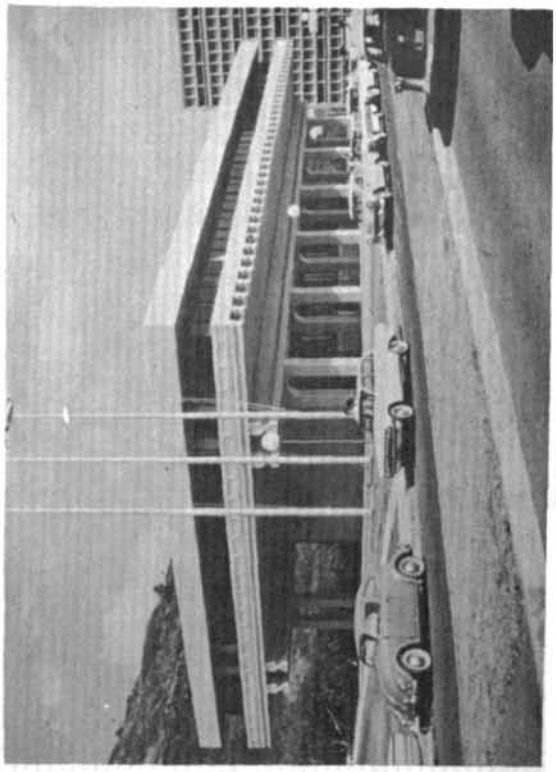


FIGURE No.20 - EAST WEST CENTRE

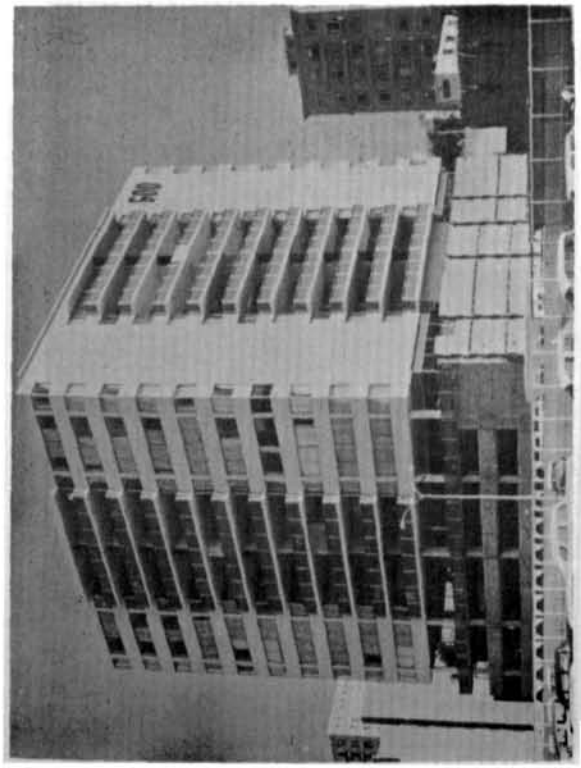


FIGURE No.17 - 600 EAST OCEAN APARTMENTS, LONG BEACH

FIGURE No.19 - ALA MOANA BUILDING

FIGURE No.21 - GOZU CITY PUBLIC OFFICE

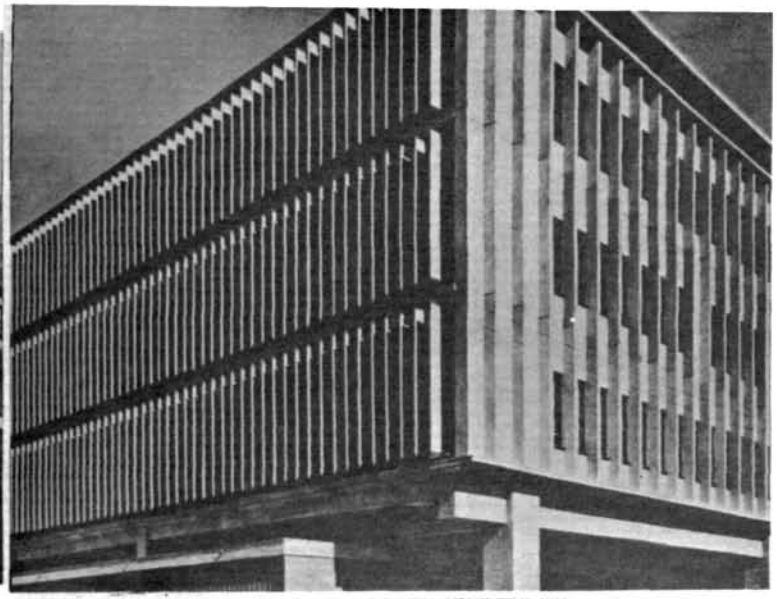


FIGURE No.22 - SAITAMA PREFECTURE AGRICULTURAL HALL AT URAWA CITY

FIGURE No.23 - NATIONAL PUBLIC SAFETY BUILDING

FIGURE No.24 - NATIONAL BANK OF ALASKA

FIGURE No.25 - FAILURE OF SUPPORTING STRUCTURES

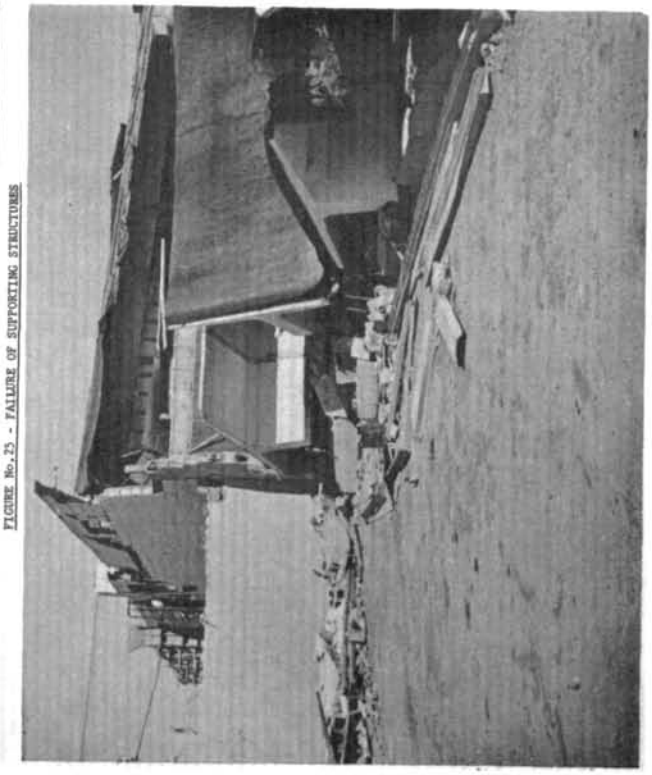


FIGURE No.26 - FAILURE OF SUPPORTING WALLS

FIGURE No.27A - FAILURE OF WELD PLATE CONNECTION

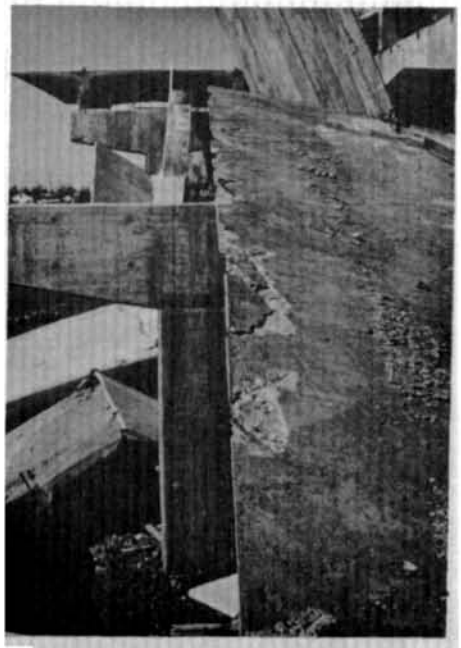


FIGURE No.27 - FAILURE OF WELD PLATE CONNECTION

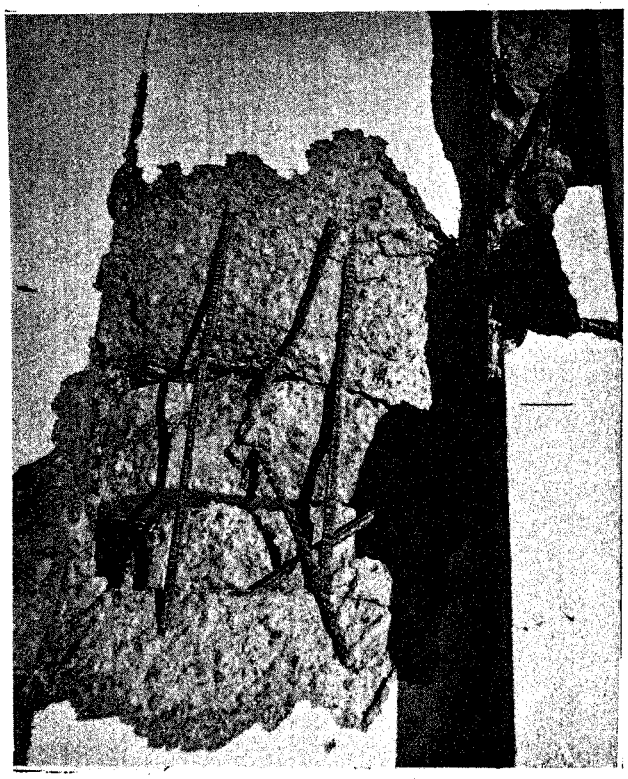


FIGURE No. 28 - INADEQUATE BEAM SEATING

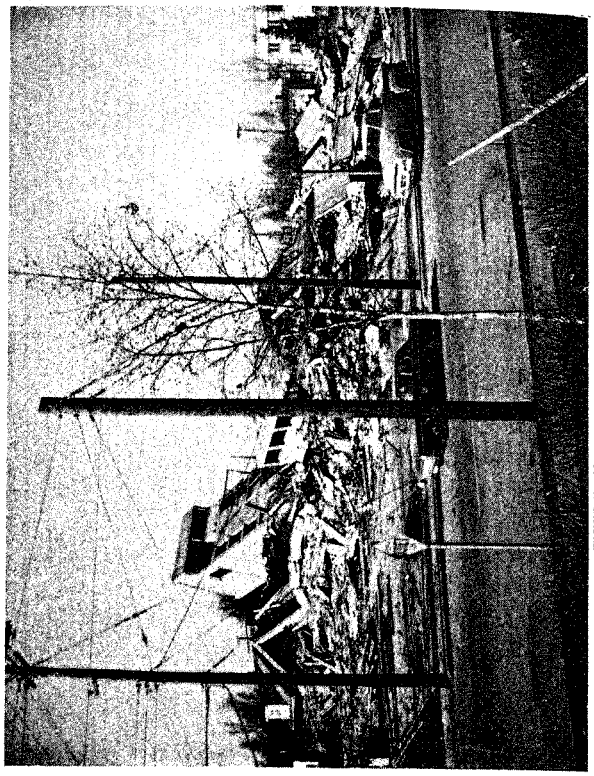


FIGURE No. 29 - FOUR SEASONS APARTMENT BUILDING.
GENERAL VIEW

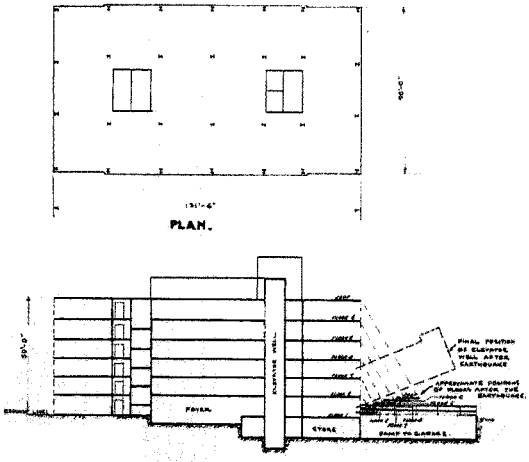


FIGURE No. 30 - FOUR SEASONS APARTMENT BUILDING
PLACEMENT OF SLABS

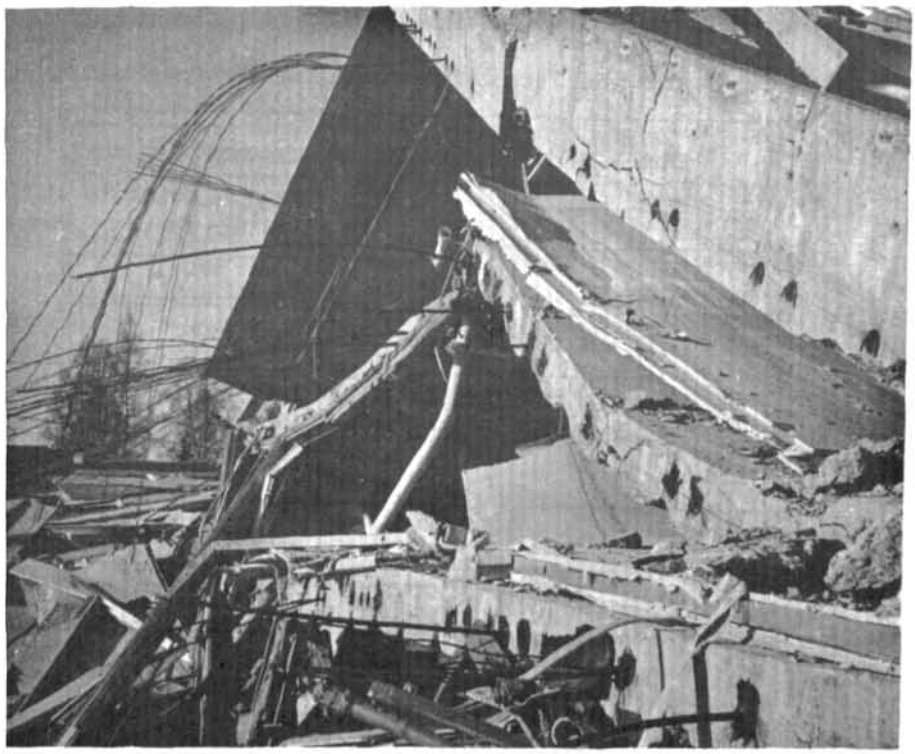


FIGURE No.31 - FOUR SEASONS APARTMENT BUILDING,
FAILURE OF UNBONDED CABLES

FIGURE No.32 - FOUR SEASONS APARTMENT BUILDING,

(6.2) Maximum horizontal deflection of specimens. Ductility.

Diagrams of (horizontal force)-(horizontal deflection) are shown in Fig.8.

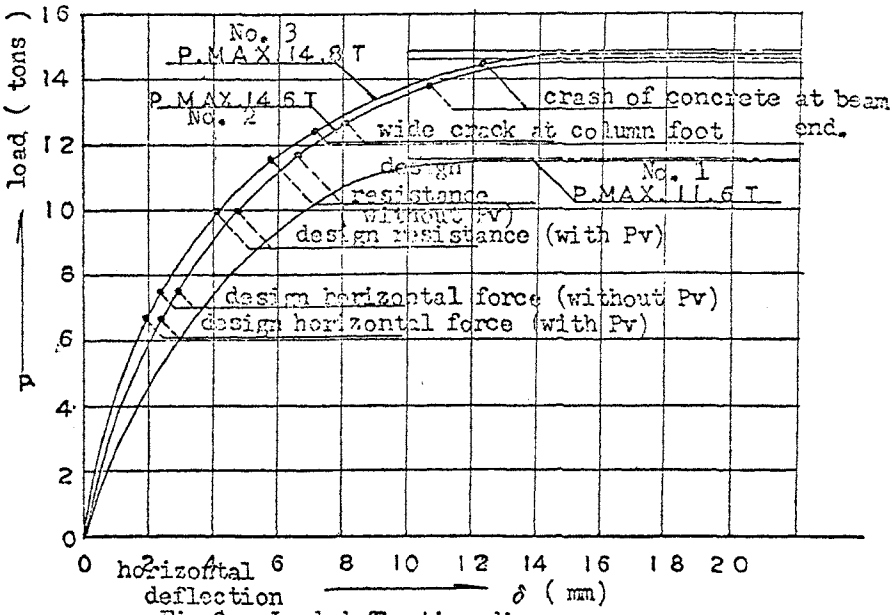


Fig.8 Load-deflection diagram.

The following facts are induced from the above diagram.

(i) All of the specimens show increment of load up to the moment when the last plastic hinge is formed.

(ii) The shape of the diagram for No.2 specimen and that of No.3 specimen differs only slightly. The shape of the diagram for No.1 specimen differs considerably from that of No.2 & 3 specimen. That means rigidity of No.1 specimen is considerably less than that of No.2 & 3 specimen, due to the lack of grouting.

(iii) Measured horizontal deflection of each specimens is more than 30 mm. That is, deformability of the specimens is more than 9,4 times of measured horizontal deflection corresponding to the design horizontal force for No.1 specimen, and 12.5 times for No.2 and No.3 specimen.

FIGURE No.33 - LOAD DEFLECTION DIAGRAM

(6.3) Deflection due to alternative loading.

Load-deflection diagrams under alternative loading are shown in

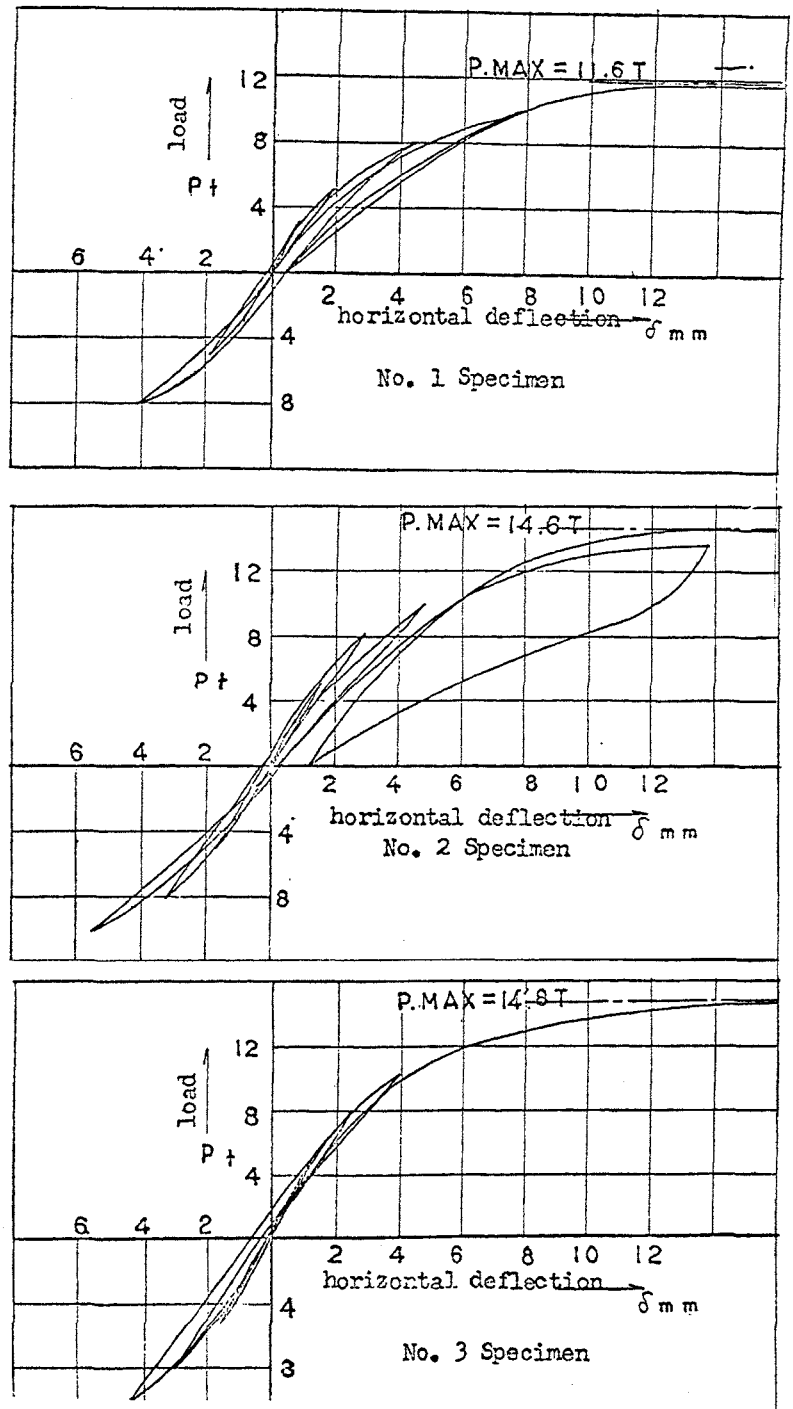


FIGURE No.34 - DEFLECTION DUE TO ALTERNATIVE LOADING

PRESTRESSED CONCRETE EARTHQUAKE RESISTANT STRUCTURES, DEVELOPMENT, PERFORMANCE AND CURRENT RESEARCH

BY W.M. SUTHERLAND

QUESTION BY: O.A. GLOGAU - NEW ZEALAND

It appears that the confusion about the terms ductility, energy absorption and energy dissipation continues. Under earthquake motions of the single heavy pulse type, energy absorption or ductility, in the sense that Mr. Sutherland uses it, is a valid criterion and prestressed concrete can be expected to perform in a superior manner. Under earthquake motions of the worst type of high intensity, energy dissipation by fat hysterisis loops is necessary to prevent a large response. Prestressed concrete under any type of loading does not appear to give us these fat loops. As Mr. Sutherland has said - the material is never at fault but the designer who does not recognise this property and hence design for larger, perhaps very much larger forces does not provide for adequate resistance to collapse.

QUESTION BY: C.F. CANDY - NEW ZEALAND

I note Mr. Sutherland has quoted Mr. Oladapo as concluding "prestressed concrete is much more elastic". Would Mr. Sutherland care to say which has the greater response to an earthquake - an elastic structure or a plastic structure?

AUTHOR'S REPLY: Prior to the ultimate strength of the structure being reached, prestressed concrete structures will behave as a non-linear non-hysteric system in contrast to reinforced concrete structures which behave as non-linear hysteretic systems. As a result the deflections produced in prestressed concrete structures can be expected to be greater than those produced in corresponding reinforced concrete structures, but this does not mean they will be any more susceptible to damage or collapse. The relation between deflection and damage for a structure will depend on the structural system and the material properties and will be different for every different type of building. Hence the knowledge that Building A will deform more than Building B in an earthquake does not mean that Building A is more likely to be damaged or more likely to fall down.

Mr. Glogau makes special mention of the effect of large earthquakes. The first point to be made is that it will always be possible to imagine an earthquake large enough to bring down any building, so any discussion on the effect of large earthquakes must be restricted to the largest earthquake that can be reasonably well expected to occur. The second point to be made is that in the region of the ultimate strength, yielding does occur in pre-stressed concrete structures so that they change to hysteretic systems and dissipate energy. The third point is that in such earthquakes the fatigue strength of structures in the region of their ultimate strength will be the most important factor determining their ability to resist destruction and in this respect prestressed concrete structures can be expected to be the equal if not the superior of reinforced concrete structures.

QUESTION BY: G.H.F. McKENZIE - NEW ZEALAND

It is fairly widely accepted that the ductility of reinforced concrete members is greatly increased by the addition of compression steel adequately stirruped against buckling. One would expect this to have a similar effect on prestressed members. Has the author any comments on the advisability of adding compression steel to improve the seismic performance of prestressed members. I realise that this will cause difficulty at joints between precast sections.

AUTHOR'S REPLY: It can be said that, as in the ultimate strength condition prestressed concrete acts very much in the same way as reinforced concrete, one would expect similar measures to have similar effects. If there was a possibility of compression failure in the concrete the addition of the compression steel would probably be advisable. Otherwise its use may not be warranted. There seems little point in increasing ductility for its own sake; it will depend on whether the engineer considers the structure already possesses sufficient ductility. As yet there is no precise definition of what is "sufficient ductility", and our only knowledge of the ductility properties of structures - both reinforced and prestressed concrete - is from a few research workers who have generally concluded that the structures they investigated possessed "sufficient ductility". Until considerably more research is done on this topic - and Dr. Nakano's paper is the only paper to be presented to this Conference on this topic - it is doubtful whether our understanding of the earthquake resistant design of structures can be much further advanced.

QUESTION BY: F. KRATKY - NEW ZEALAND

Is there any information about the behaviour of prestressed concrete buildings 20 storeys high and more (in Hawaii), under earthquake conditions.

AUTHOR'S REPLY: As far as I am aware there is no information about the behaviour of prestressed concrete buildings twenty storeys high and more under earthquakes.

PRESTRESSED CONCRETE EARTHQUAKE RESISTANT STRUCTURES DEVELOPMENT PERFORMANCE AND CURRENT RESEARCH

BY W.M. SUTHERLAND

DISCUSSION

BY L. BENUSKA*

Because of the widespread interest among structural engineers regarding the "Four Seasons" Building failure in Anchorage, Mr. Sutherland's comments about the mode of failure must be amplified and corrected, so that proper conclusions can be made.

(1) The Alaskan Earthquake.

The magnitude of the March 27, 1964 Alaskan Earthquake has been estimated at 8.4 on the Richter scale, with the epicenter at about 75 miles away from Anchorage. At this distance, the higher frequency components of the ground motion were attenuated relative to the lower frequency components. Thus, the structural damage in Anchorage was less significant for the rigid buildings and more pronounced for the flexible ones when damage was principally the result of ground motion.

By correlating observed damage trends with the velocity response spectrum description of earthquake effects, an elastic spectral velocity in the range of 7 to 12 inches per second is indicated for the Four Seasons structure ($T = 0.37$ sec., damping = 5%). This would indicate elastic lateral force levels of 3 to 5 times the design values.

(2) Structural System of the Four Seasons Apartment Building

The structural system was composed of post-tensioned concrete flat slabs supported on steel columns and two reinforced concrete stair shafts (Figure A). The flat slabs were cast on the ground and lifted into position, and then the stair shafts were poured. Spread footings were used for the substructure. Resistance to lateral loads was provided by the two stair shafts. At the time of the earthquake, the structural construction was completed, the slab collars were welded to the steel columns; but apparently no intention was made to develop moment connections between the slabs and the columns, and the moment transferred between them could not be counted on to provide lateral strength.

(3) Lateral Resistance of the Stair Shafts

The stair shafts were designed to resist the lateral forces specified in the Uniform Building Code, 1961 Edition. A check (Table 1) of the structural plans indicated an adequate design under the specified criteria, the shafts possessing a total overturning resistance of 43,800 k-ft at yield of steel (Figure C) as against 20,000 k-ft of moment produced by the code design forces - a factor of safety of 2.2.

The actual earthquake would have produced stresses in the stair shafts of 3 to 5 times the code design values if they remained elastic. The stresses could not reach these values of course, and ductile yielding of the shafts

* Associate, T.Y. Lin and Associates, Van Nuys, California, U.S.A.

was necessary to absorb the earthquake energy.

The actual collapse of the building, resulted from the failure of the shafts near their base (Figure D). All bars were spliced with a 20-bar diameter lap at one level. It was noted by a number of observers, including investigators of the Portland Cement Association who were there shortly after the earthquake, that the large bars, # 8s and # 11s, all failed at the splice, while the # 4 bars were broken with a ductile rupture. The failure of the splice of the large bars can be explained. The 20-bar diameter splice would have resulted in an average bond stress of 560 psi (Figure E) when the steel was stressed to the yield point. This stress exceeds the ultimate strength values given in ACI 318-63 (Section 805 (b) and Chapter 18) for large bars.

(4) Rotational Movement of the Shafts

As can be seen from the photographs of Figure 1 and Figure 2, the shafts were severed near their base (level 1, Figure A) and overturned to the north, carrying the slabs with them. The photograph of Figure 2 shows the corner of the roof overhang (see column line 6, Figure A) displaced northward about 35 feet. This northward movement would have been greater (about 52 feet) if the slabs completely followed the shafts to the ground, describing an arc with the base of the shafts as the center. Apparently, some slabs began to slip downward as the shafts were turning over, hence a combination of overturning and vertical sliding movements took place.

The speculation that the slabs had failed first and slid down vertically before collapse of the shafts is not founded. If the slabs had fallen to the ground before the shafts failed, the reduction in elevated mass would have prevented collapse of the shafts.

(5) Secondary Effects

A rather spectacular secondary effect was the release of many prestressed cables ($\frac{1}{2}$" greased and wrapped tendons) as the structure collapsed. The photograph of Figure 4 shows the severe twisting imparted to the slabs as the shafts rotated toward the north. Figure F describes how this twisting would deform the concrete around the anchorages located at the slab junction with the shaft outer wall. These wedge type anchorages were located in an area of maximum slab distortion and released the cable as soon as they were broken loose from the concrete.

Also, severe wrenching of the slabs, after the shafts had listed, broke loose some of the tendons anchored near the closure strips. Upon impact with the ground, many of the remaining anchorages were released.

The appearance of cables on the roof of neighboring buildings is explained by their release during the tilting of the shafts. It is noted that the slabs were still quite high up until the shafts were displaced well beyond the vertical.

During the descent of the slabs, severe distortion at the column connection resulted in the shear head connection punching through the slab.

The slab concrete had varying degrees of prestress due to the progressive release of prestress cables as the shafts rotated northward. The photograph of Figure 6 shows a typical steel column, in a buckled configuration with shear heads in place.

(6) Conclusion

The collapse of the Four Seasons building under the Alaskan Earthquake resulted from the overturning of the shafts which, in turn, was attributed to the bond failure of the main bars in the shafts. Bond failure with all bars spliced at one joint was brittle and could not supply the ductility necessary to absorb the energy imparted by the serious earthquake. The loosening of the tendons was an after-effect resulting from the severe twisting movement between the shafts and the slabs and was not the cause of the collapse.

AUTHOR'S REPLY: Mr. Benuska's contribution to the discussion on the failure of the Four Seasons Apartment building is appreciated. At the time of writing the paper, the author had believed that the towers were surrounded by slabs on all four sides. All the structural details had not at that stage been received, with the result that a proper analysis could not be made.

Mr. Benuska's conclusions are quite correct, but attention should be drawn to the question of what would have happened to the building if the main towers had not collapsed completely due to the non-ductile behaviour of the reinforced concrete.

In paragraph 5 of this report, Mr. Benuska points out that the wedge-type anchorages located in an area of maximum slab distortion released the cables as soon as they were broken loose from the concrete. With unbonded cable tendons the effect of this type of anchorage failure or damage will obviously cause a release of prestress in the slabs, and the result could well be that the towers remain upright while the slabs collapse.

Seismic resistant designs should be based on the ultimate capacity of a system and as it is recognised that any unbonded system has an ultimate capacity of approximately 20% less than a bonded system, it is difficult to see how the use of unbonded tendons can be either technically or economically justified.

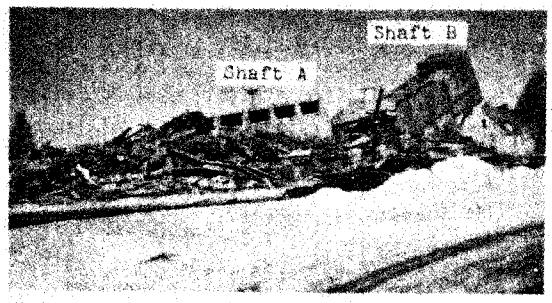


Figure 1. GENERAL VIEW

Figure 2. SHAFT B AND ROOF SLAB DISPLACED TO THE NORTH

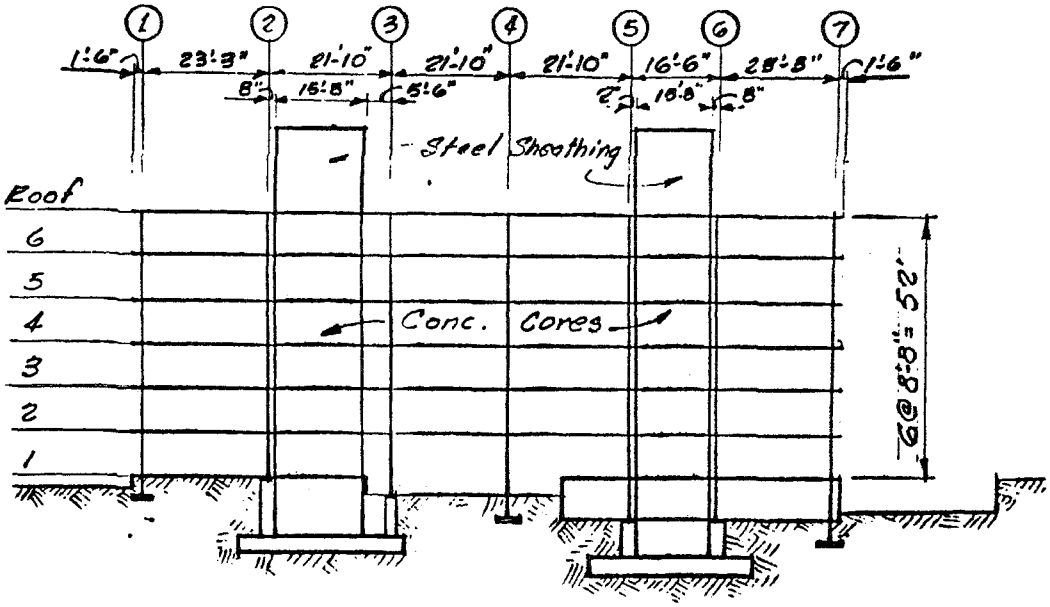
Figure 3. WEST SIDE OF SHAFT A

Figure 4. WEST EDGE OF ROOF SLAB

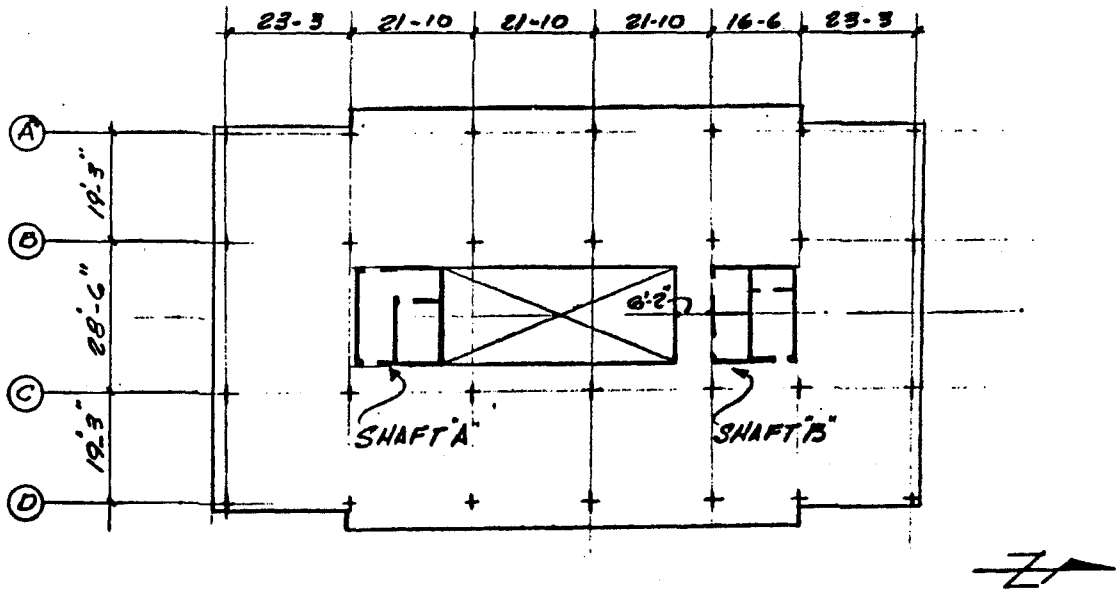
FOUR SEASONS
APARTMENT
After
March 27, 1964
Earthquake

Figure 5. NORTH SIDE OF SHAFT B BASE AFTER CLEAN-UP

Figure 6. STEEL COLUMN



Longitudinal Section



Typical Floor

Figure A. FOUR SEASONS BUILDING

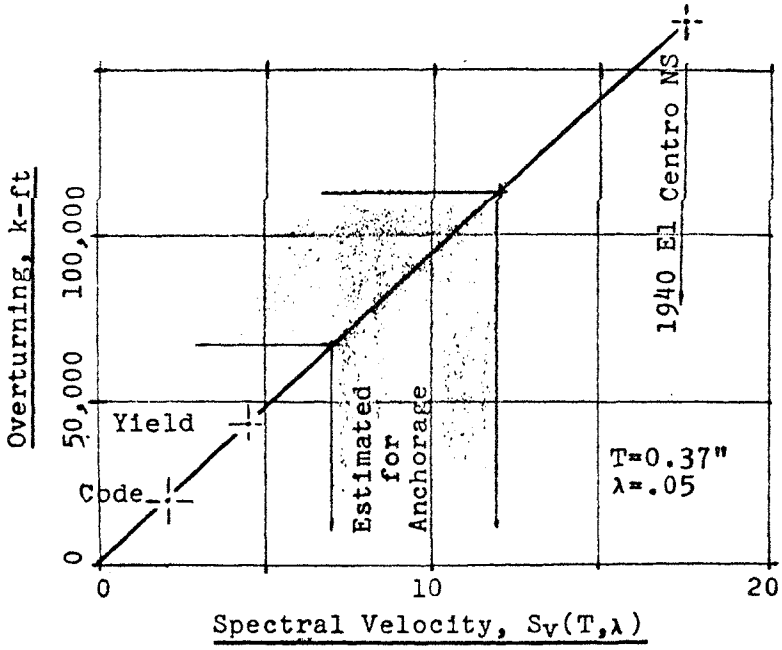


Figure B. ELASTIC RESPONSE vs EARTHQUAKE INTENSITY

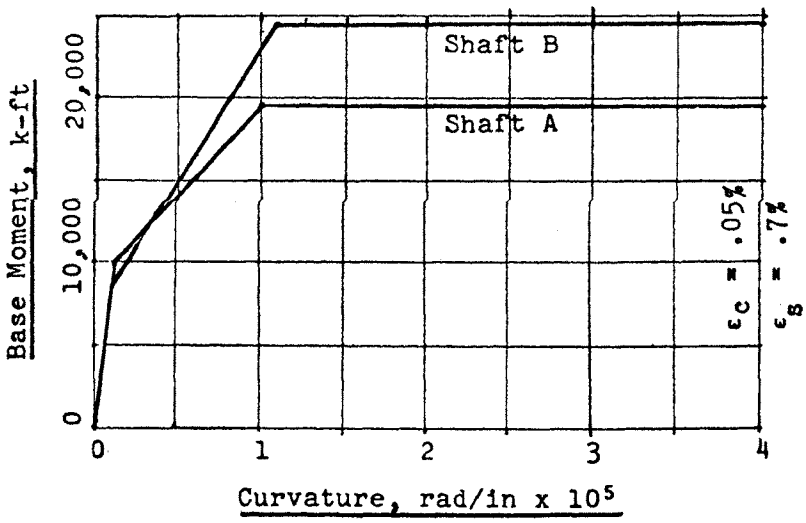
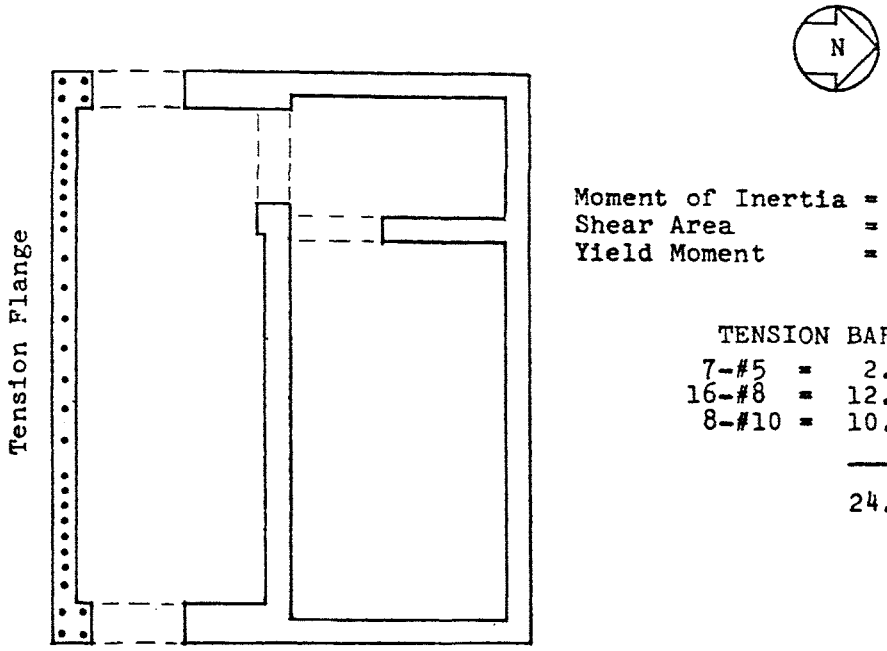


Figure C. BENDING CURVATURE RELATIONSHIP AT BASE OF SHAFTS

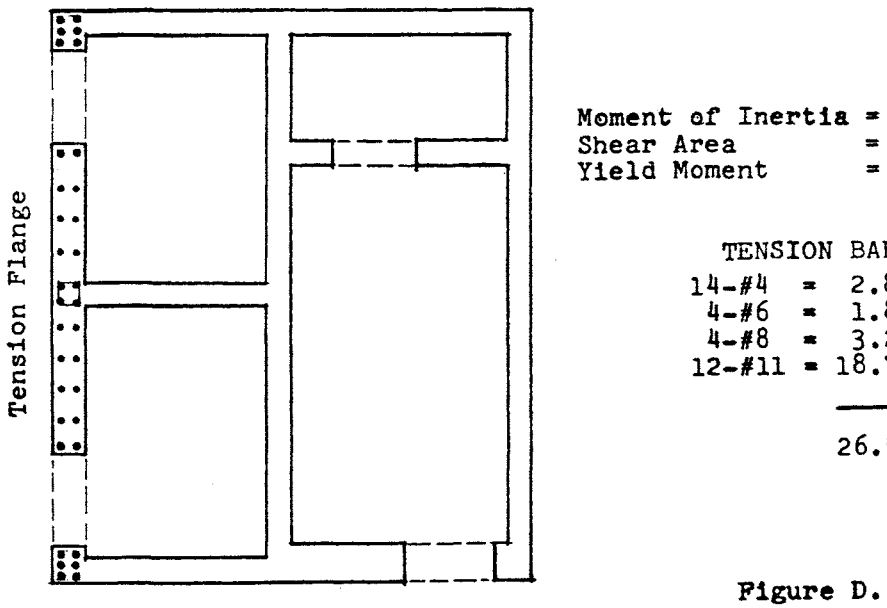


Moment of Inertia = 2290 ft.^{4}
Shear Area = 25 ft.^{2}
Yield Moment = 19,500k-ft.

TENSION BARS

7-#5	=	2.1 sq.in.
16-#8	=	12.6
8-#10	=	10.2
		24.9 sq.in.

SHAFT A
Scale: 3/16" = 1'-0"



Moment of Inertia = 2010 ft.^{4}
Shear Area = 25 ft.^{2}
Yield Moment = 24,600k-ft.

TENSION BARS

14-#4	=	2.8 sq.in.
4-#6	=	1.8
4-#8	=	3.2
12-#11	=	18.7
		26.5 sq.in.

SHAFT B
Scale: 3/16" = 1'-0"

Figure D.

SHAFT PROPERTIES

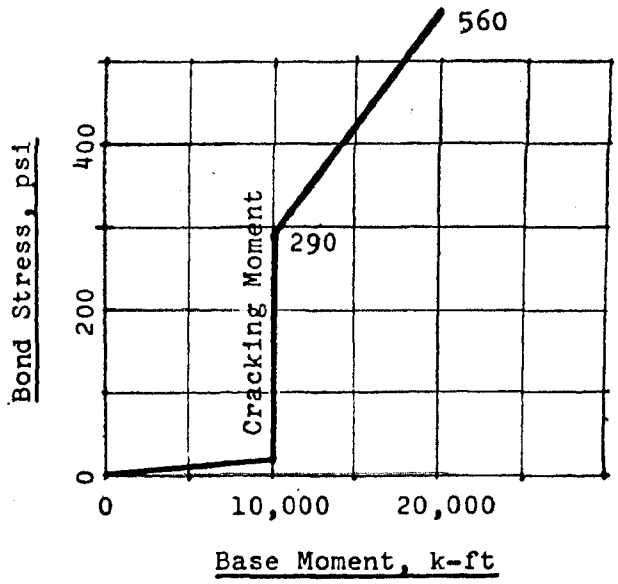
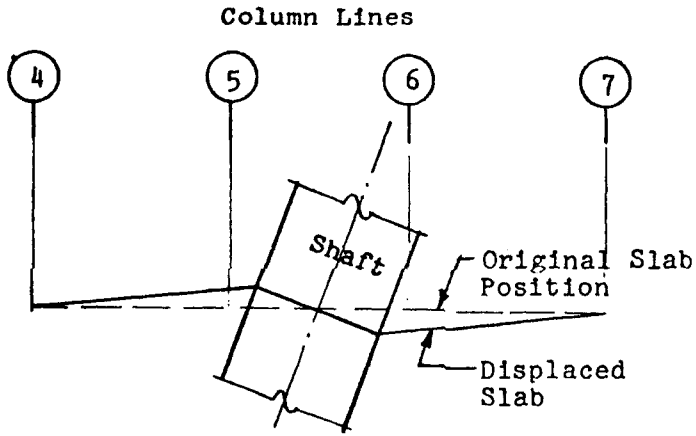
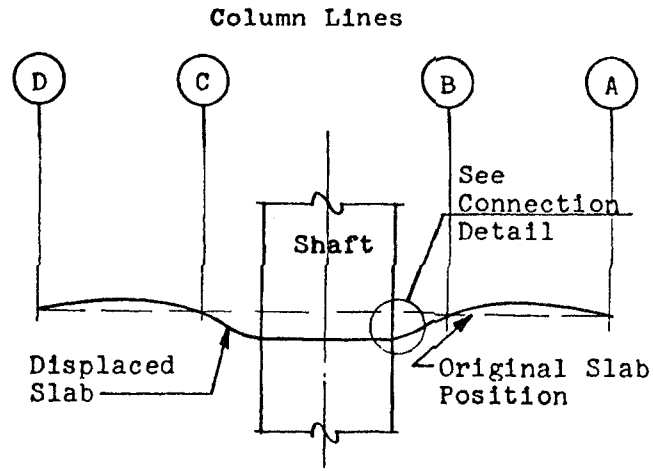


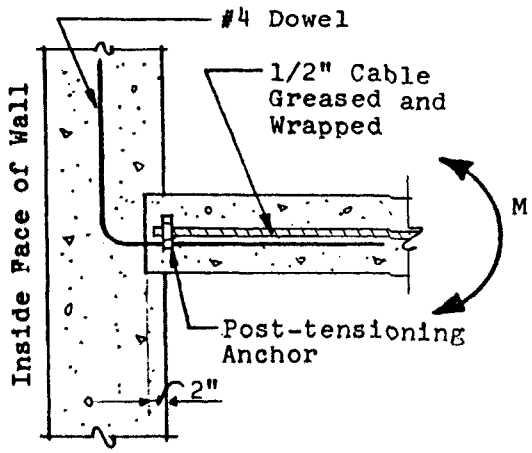
Figure E. BOND STRESS AT BASE OF SHAFT A



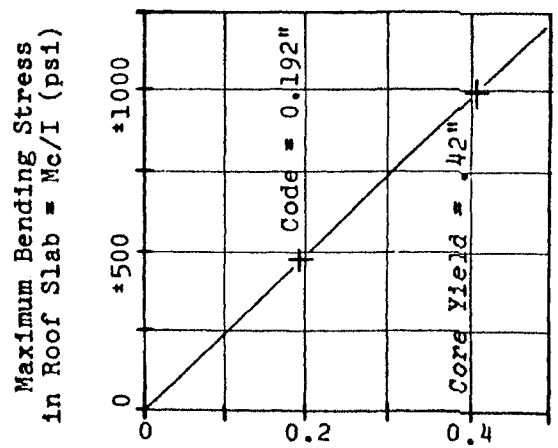
a. East Elevation



b. North Elevation



c. Connection Detail



d. Roof Sidesway, (in.)

Figure F. SECONDARY SLAB STRESSES

AT JUNCTION WITH SHAFT WALL

TABLE 1. Code Seismic Force Requirement

Level	Estimated Weight	Shear(1)	Overturning	Sidesway Displacement(2)
Roof	1080 k			.192 in.
		152 k		
6	1080		1300 k-ft.	.159
		278		
5	1080		3700	.124
		379		
4	1080		7000	.090
		455		
3	1080		11000	.059
		506		
2	1080		15400	.032
		530		
1			20000	.011

Notes:

1. Base shear calculated from SEAOC recommendations

   $T = .05H/(d)^{1/2} = 0.23$ sec

   $C = .05/(T)^{1/3} = 0.082$

   $Z = 1.0$, $K = 1.0$

   $V = (1.0)(1.0)(.082)(6480k) = 530$ k

2. Shafts were assumed fixed at 8.67 feet below level 1 (Figure A.).

TABLE 2. Longitudinal Elastic Vibration
Modes and Effective Weights

	Mode Shapes (1)		
Level	T_1=0.371"	T_2=.087"	T_3=0.042"
Roof	.3822	-.2837	.2406
6	.3154	-.0796	-.0756
5	.2470	.1204	-.2807
4	.1797	.2633	-.1965
3	.1172	.3128	.0913
2	.0633	.2624	.3124
1(2)	.0229	.1406	.2649
Effective Weight =	5350 k	1650 k	385 k

Notes:

1. The standard Jacobi Diagonalization procedure was used to calculate mode shapes and frequencies, utilizing SHARE subroutine F 2 MIHD 13.

2. Shafts were assumed fixed at 8.67 feet below level 1 (Figure A.).

DISCUSSION

BY K.S. ZAVRIEV*

Dynamics and Seismo-Proofness of Pre-Stressed Reinforced Concrete Constructions

By using pre-stressed reinforced concrete in seismic areas, reliable connection is reached between the parts of structures, their favourable spatial work being thereby sound. The fault of pre-stressed reinforced concrete in relation to its seismic resistance lies in the high resistance to the development of plastic deformations which makes concrete more vulnerable to perception of seismic shocks. The increase of dynamic effect is also reached by decreasing oscillation decrement. The following reasons however, arrive at a conclusion that the above-mentioned faults are not of particular importance in the appreciation of seismo-proofness of pre-stressed reinforced concrete constructions.

In order not to rise very much the cost of earthquake resistant arrangements when designing earthquake resistant structures, the requirement is to nondamage in case of inconsiderable earthquakes. As to the heavy earthquakes which do not occur too often, it is necessary just to eliminate possibility of collapse and heavy damage which might endanger human life or safety of valuable property. Destruction of reinforced concrete structures including pre-stressed reinforced concrete structures begins with the development of plastic deformation and appearance of cracks.

This leads not only to decrease the resistance, but also to diminish values of seismic forces as a result of their weakened rigidity and increased inner resistances. If the process of drop in resistance outstrips the decrease in seismic coercions, collapse occurs.

In the structures made of pre-stressed reinforced concrete resistance to the development of plastic deformations and formation of cracks is much higher than in the case of plain reinforced concrete. This results in better resistance of former to inconsiderable earthquakes. As to the heavy earthquakes the coercion of which spells destruction, the latter is preceded by the development of plastic deformations and crack building where by the pre-stresses disappear.

In the case of rational design of pre-stressed reinforced concrete constructions, the possibility of their brittle destruction except behavior of pre-stressed reinforced concrete with formation of cracks is coming closer to the behaviour of plain reinforced concrete and before these constructions are faced with destruction they will behave as if they were not prestressed.

Thus, pre-stressed reinforced concrete constructions resist coercion of inconsiderable earthquakes better, and no worse than that of plain reinforced concrete constructions to heavy earthquakes. Therefore, with

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* Member of the Academy of Sciences of the Georgian S.S.R.

due consideration to all other advantages of pre-stressed reinforced concrete the use of the latter in seismicity areas may be considered expedient.