MULTISTORIED STATE BUILDING DESIGN IN NEW ZEALAND

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Building heights in New Zealand were limited to a little over 100 feet until quite recently when the restrictions were removed by most local authorities controlling building construction. The design of four public buildings about 200 feet in height is described in this paper: Aitken St. Departmental Building, Wellington, with a composite reinforced concrete and structural steel core, Wellington Postal Centre with a structural steel reinforced concrete frame and two sections of the Science Faculty Buildings Auckland with stiffened shear walls. It is probable that all of these buildings will be instrumented as the site conditions are typical of the localities and the structures amenable to rational analysis.

Seismic Coefficients. An idealised spectral curve defining seismic coefficients, fig. 10, has been adopted for Ministry of Works building designs pending completion of the revision of the national building code. This curve has been based on the maximum el Centro response curve appropriately reduced to allow for a damping coefficient of 10% critical and a ductility factor of 4 (1). It is realised that there are several factors other than damping and ductility likely to cause reductions in building response and that ductility factors much lower than 4 should prove adequate in earthquakes, nevertheless those factors do furnish a convenient means of sealing coefficients to the levels found satisfactory in practice. The curve is applied to all of the usual building materials although for steel damping values are likely to be less and ductility factors higher than for reinforced concrete while the reverse conditions will apply to reinforced masonry. Records of earthquake damage indicate better performance for the ductile materials. However, an assumption that the effects of damping and ductility largely compensate for the range of building materials in common use appears to be reasonable provided that structural steel is used for the tallest and most important buildings and reinforced masonry for low buildings of little importance. As yet microregionalisation surveys have not been carried out in New Zealand so the curve has been cut off at a relatively high level at 1.2 seconds to make some provision for the effects of weaker soils and of more distant earthquakes on the taller buildings (2).

Coefficients derived from the curve are intended to apply to regions of the highest seismicity, 75% of these values to regions of intermediate seismicity and 50% to the regions of least seismicity.

The buildings described in this paper were designed at different times so seismic coefficient may vary a little; however, allowing for small differences in working stresses it will be found that the two

(1) Veletsos, A.S. and Newmark, N.M., "Effect of Inelastic Behaviour on the Response of Simple Systems to Earthquake Motions", Proc. Second World Conference on Earthquake Engineering, July 1960.
(2) Kanai, K., "An Empirical Formula for the Spectrum of Strong Earthquake Motion", Proc. Second World Conference on Earthquake Engineering, July 1960

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buildings in Wellington were designed for coefficients conforming quite closely with the curve values while those in Auckland were designed for 50% of the curve values.

LINK TOWER AND CHEMISTRY RESEARCH BLOCK

UNIVERSITY OF AUCKLAND

Introduction. The two buildings described in this part paper are at present under construction and form part of the $3\frac{1}{2}$ million pounds Science Buildings for the University of Auckland. They are located in an area of New Zealand which is at present widely regarded as being one of low seismic activity. The current Ministry of Works code for the seismic design of public buildings prescribes relatively low horizontal force co-efficients for this zone but the standard of aseismic detailing is not reduced.

Two solutions are presented to the structural problem of providing rigidity for buildings having a high mechanical services content and particular attention has been given to avoid damage induced by shear wall deformation.

General. The principal objectives of the structural designer of Ministry of Works public buildings are to provide efficient and economic protection of life of persons in and around the buildings in exceptionally strong earthquakes and to minimise damage in moderately strong earthquakes. When dealing with laboratory science buildings the following additional important consideration arises; valuable and delicate instruments must be protected from damage due to earthquakes of medium intensity and the undesirable vibrational effects of wind and traffic.

The designer is therefore obliged to adopt the most efficient method of achieving rigidity, that is, a shear wall system. Such a system has the further advantage that small interstorey deflections considerably simplify the connections of precast cladding components (generally using stainless steel fixings) and those of windows.

A serious difficulty arises from a high building services content. Normal shear walls or shear cores seriously interfere with the efficiency of pipelines and ducts or vice versa and furthermore the full requirements are rarely known in the early stages of structural design. To give the maximum freedom to services wall elements were eliminated completely along the lines usually used by services - the corridors. The resulting loss of stiffness was compensated for by other means as is described in detail later.

The problem of shear wall induced seismic damage in members connected to them - well documented by now - was minimized by the measures taken to stiffen the walls and by softening the connections.

In order to preserve the symmetry of the buildings and thereby avoid the little explored problem of torsion in higher buildings all rigid non-structural partitions have been separated by slip joints.

Some loss in damping had to be accepted due to the above procedure but it is thought that this is a minor factor in a ductile building and more than offset by the beneficial effects mentioned above and the greater assurance of obtaining a structure of more predictable behaviour.

Common Design Basis. These buildings were designed in a period when a seismic code based on the dynamic effects of earthquakes and soil conditions was applied on an experimental basis to the design of public buildings in New Zealand. A similar statement can be made with respect to the Load Factors and Undercapacity factors used. The basis of design was therefore not identical for the two buildings. Details are given below:

Materials:

Concrete	4000 p.s.i (28 day cylinder), 12½% co-efficient of variation specified.
Deformed mild steel	33,000 p.s.i. yield point mild steel.
Prestressing steel	Freyssinet 12 - ½" Multistrand

Foundation:

Sandstone capable of allowable bearing pressures in excess of 20,000 p.s.f.

Seismographs: (Chemistry Research Block)

(A) 3 positions in one of the towers at basement, third and eighth floor

(B) 2 positions at extreme ends just below eighth floor

(C) 1 position in basement of a low building some distance away.

A more detailed description of the two buildings follows:

LINK TOWER: (Figs. 1a - 1h)

In function this building is closely related to the adjacent chemistry research building. A large portion is taken up by a services tower and a boilerhouse in structural steel rising above the eighth floor. In order to avoid torsional problems the building was separated by a 6" seismic break from the adjacent buildings but this procedure did result in a rather slender tower and in the necessity to have a separate structural system for a relatively small useful area.

To avoid conflict with services the transverse shear wall was separated from the longitudinal walls at the corridors. Deflection calculations indicated that difficulties would be experienced in a number of places in the building unless special measures were taken.

Design Basis. Computed periods are 0.7 sec. transverse and 0.42 sec longitudinal, base shear co-efficient 0.12 both directions, triangular distribution. This results in a lower ductility requirement for the prestressed wall which seems appropriate. The combined effect of under-capacity factors and load factors was essentially that of the 1963 ACI code.

Transverse Shear Wall. Fig. 1f shows the deformation of the transverse shear wall under seismic loads and deformations induced in more flexible components of the building.

The left half of the figure shows the effect on members framing directly into the wall and the right hand side the deformations imposed on a frame located at some distance away from the wall.

It is seen that the deformations induced in members connecting points 'a' and 'b' (Fig. 1b) are much less severe than those connecting 'd' and 'e' provided the building is relatively flexible between 'c' and 'd'.

The connection between 'a' and 'b' was made only by means of a 5" slab hinged at the outer wall line (Fig. 1d) but incapable of further softening owing to the position of the interior beams. The beam in line with 'd' and 'e' was stopped at 'd' and not carried across the opening to the lift. A detail such as shown in fig. 1c was under consideration to protect the edge of the slab from impact loads at the entry to the lifts while retaining flexibility but was finally considered not warranted in this case.

None of the above measures was thought to be sufficient to guarantee freedom from damage in strong earthquakes, the principal cause being insufficient moment of inertia of the wall. The most effective way of increasing this property was by means of prestress. The level of prestress was determined in the following manner: Tensions in the concrete at levels below the modulus of rupture were assumed to result in possibly a few cracks but no great increase in wall deflections.

On the other hand a high level of tensions would result in many cracks causing the moment of inertia to drop to the level given by the theory of cracked sections for concrete. The maximum tensile stressed due to seismic bending were computed to be about 1350 p.s.i. reduced by dead load compression to 1100 p.s.i. As modulus of rupture of the concrete was taken as 400 p.s.i. the lower 100 ft. of wall was prestressed to a uniform 700 p.s.i.

<u>Practical Aspects</u>. The original design envisaged the use of $1\frac{1}{4}$" dia. MacCalloy bars. This would have permitted the addition of bars storey by storey without any access holes, excepting those for grouting tubes at base and 50' height. Owing to rapid progress of the construction delivery of the bars in the correct lengths with threads was in doubt and a change to Freyssinet 12 - $\frac{1}{2}$" Multistrand cables was requested by the Contractors and approved. Fig. 1g, 1h. This system required recesses 8" x 10" x 15" deep and at 27" centres to be left in the wall to allow the introduction of cables after erection of the wa ll. Fortunately shear stresses are not high in the region just above the raft where the wall is wider and thicker nevertheless the pockets were filled with epoxy concrete.

Keeping in mind that the primary purpose of the prestress was to reduce induced stresses in other building components it was obvious that the lower 100 ft. of wall had to be kept free until elastic shortening due to prestress and part of the creep had taken place. It was specified that connection of side shear wall and prestressed walls at 100 ft. level had to be delayed till 3 months after prestressing and at lower floors for proportionally shorter periods. This posed the added complication of building a 100 ft. cantilever without its major portion of reinforcement in position. By the addition of some mild steel it was possible to construct the wall without the use of expensive temporary lateral supports and at the same time to increase the "ductility factor" of the prestressed wall.

During construction there were some anxious moments when it was thought that duct blockage had occurred. The prestressing firm felt they could develop sufficiently high grouting pressures to overcome a 100 ft. pressure head and chose to omit grouting tubes at 50 ft. level. As it turned out they had to modify their equipment slightly to deal with the high static head and after that no further difficulty was experienced.

Detail 1e shows the method employed at fire breaks. Concrete was used to simplify architectural detail but a clear separation between transverse and longitudinal walls was maintained.

CHEMISTRY RESEARCH BLOCK - Fig. 2 (left hand and centre)

Design Basis.

Transverse period 0.51 sec

Longitudinal period 0.65 sec Max. interstorey deflection 0.055"

Base shear co-efficient 0.08, triangular distribution

Load factors and Undercapacity factors 1963 A.C.I. with minor variations.

Structural Framing and Details. Seismic forces on this building are resisted by 2 pairs of channel shaped towers (Fig.3). For reasons given previously each pair of channels is interconnected at intermediate levels only by a $4\frac{1}{2}$" slab but at foundation and roof levels flanges are interconnected over some 24 ft. forming a Vierendeel, the deformations of which are a fraction of those of individual cantilevers.

The floors are formed of in-situ columns and main beams together with precast secondary beams at 10'-6". Floor slabs are to be precast in approximately $11\frac{1}{2}$' x $9\frac{1}{2}$' panels $4\frac{1}{2}$" thick with a thin levelling screed on top. A 10" wide in-situ junction area with looped reinforcement ensures that the floor functions as a effective diaphragm (Fig. 3f and 3g).

The problem of avoiding intolerably high induced stresses in members framing into the shear towers was here solved in the following manner.

At each side of the shear towers there is a half bay. This arrangement allowed us to eliminate the main longitudinal beams, in this bay so that only slabs frame into the towers. The horizontal translational movement of the towers, however, still has to be dealt with at beam and column junctions even though removed from the towers. Abnormal conditions furthermore existed here in so far as the main beams had to be located on the face of the columns in order to allow the ducts from the fume cupboards in this building to rise straight up the building and in the space between the columns alongside the corridors. Shear wall deformations cause these joints to be subjected to torsional stresses in addition to the shear and torsional stresses from gravity loads. Not much was known about the performance of such a joint and the University of Auckland agreed to do a number of tests on half size models. Investigations are not complete but results to date indicate that the joint can sustain the rotational displacements while supporting its gravity loading; Fig. 3d, 3e show the arrangement of reinforcement found most satisfactory. Alternatives such as inclined hangers from columns to beams and square spiral torsional reinforcement were not sufficiently effective for adoption.

Conclusion. The structural designer of multistorey science laboratories buildings lacking rigid exterior walls is faced with two conflicting requirements. The buildings must be rigid to prevent damage to delicate expensive intruments in moderately strong earthquakes and also in order to ensure satisfactory instrument performance in high winds. The very high building services content on the other hand leads to difficulties with conventional shear walls or cores.

Two solutions to the problem are the prestressed cantilever shear wall and the Vierendeel shear core.

AITKEN STREET GOVERNMENT BUILDING

1. INTRODUCTION. The Aitken Street Government Building is located in Wellington, New Zealand, and consists of a 17 storey tower and 5 storey annexe separated by a seismic gap. Only the tower is described in this paper, and a brief outline is given under fig. 4.

All horizontal forces are resisted by the concrete core (see fig. 5.) of which the two longitudinal walls may be described as coupled shear walls. As this was the first tall coupled shear wall to be designed in this office an investigation was carried out to determine the effects of varying the depth of the beams (over the openings), axial strain in the walls and cracking of the walls under severe loading.

This building is probably the first in this country to use a structural steel - reinforced concrete core.

2. ANALYSIS OF SEISMIC CORE.

(a) Seismic Coefficient. Note was taken of the experience and recommendations presented at the 1960 World Earthquake Conference and the final design of the tower was based on a triangular distribution with a base shear of 0.18g but using ultimate design methods with the maximum allowable stresses fy and $0.8f_c^1$. The first modal period is approximately 0.6 secs.

For comparison purposes the proposed draft of the N.Z. code being considered now (May 1964) would require little change to the design. It recommends a lower base shear but also lower steel and concrete stresses and the final result is approximately the same.

(b) Analysis of the Coupled Shear Wall. For design purposes, although it necessitated certain simplifying assumptions, the moment distribution method of analysis was used and advantage taken of "short cuts" offered by the Naylor (3) and other variants. Despite these "short cuts" the high column to beam stiffness ratio made this method very time consuming. About four complete analyses were necessary on a trial and error basis, to take account of axial strains in the columns.

The shear wall frame was later analysed by the strain energy method of influence coefficients, using an electronic computer to check the design and to take some account of the sloping buttresses and rotation of the base beam, but no change to the design proved necessary.

Because of the need for a quick approximate method some research on analysis of coupled shear walls was begun leading to the draughting of simple analysis charts which are presented in (4).

Fig. 7a shows the column to beam stiffness ratios when bending only is considered, and the additional effects caused by shear strain in the beams, axial strain in the columns and cracking (an indication only) in the "windward" column. Figs 7b and 7c show the respective beam shears and average column moments calculated by the approximate method (4). Bending, shear and axial strain were considered together in the more accurate computer analysis the results of which are also plotted.

It will be noted that the axial strain in the columns, as with shear strain in the beams, has a considerable softening effect on the equivalent beam stiffness and therefore reduces the beam shears but increases the column moments. With stiffer beams this effect is increased. Cracking of the column moves the neutral axis inwards, reducing the effective width of the system and thus increasing the column moments.

(c) Beam Depth. A 4 ft. beam depth and a storey height of 11 ft. was chosen for this building.

Quite large shearing forces are attracted to these beams, necessitating heavy shear reinforcement. It is of interest to note, however, that a reduction in depth, though causing a reduction in shear force, does not reduce the shear stress. A 25% reduction in depth and therefore in shear area only reduces the maximum shear 19% (fig. 7b).

Although there is a compensating effect of reduced dynamic response, it seems that this inverse relationship of higher shear stresses with smaller beam depths applies over a considerable range of stiffness ratios. In simple multiple wall systems the optimum beam depth may now be easily determined from reference 4, but in those systems not amenable to accurate analysis, the beam that has the maximum depth available should prove the most effective.

(d) Dynamic Analysis. The building was analysed by the D.S.I.R. using an electronic computer to determine its elastic dynamic response with 7½% critical damping to an earthquake of El Centro magnitude. The resulting shear forces divided by a factor of 2.5 were approximately equal to the design values. As a ductility factor of 2.5 does not seem unreasonable for this building, the chosen design coefficient would seem to be confirmed (see figs. 6b and 6c.)

Two dynamic analyses of the coupled shear wall (section a-a fig. 2) were carried out. In the first, only bending and shear deformations were considered; in the second allowance was made for cracking and axial strain in the "columns" (see figs. 6b and 6c). The true response of the building can be assumed to lie somewhere within the shaded area.

(e) Seismographs. Three seismographs, one in basement, one on top floor and one in the ground about 100 ft. away from the building, are to be installed in this project.

3. DESIGN OF CORE.

(a) General. The core is reinforced with structural sections and deformed reinforcing both of mild steel and with some prestressing cables in the lower buttressed portion. In this portion one third of the tensile resistance is carried by each system. Above first floor where the prestressing cables stop, the structural steel and reinforcement carry equal tensions and towards the top the structural steel share increases to about 75%.

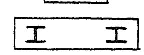
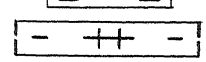
(b) Structural Steel. The structural steel (with fully welded connections) was used to provide a tensile resistance that would remain fully effective even after severe cracking; it was used in the high tension areas where spliced reinforcement would have a reduced effectiveness, and though made into a frame was not designed to have a ny significant lateral resistance by itself. There are heavy steel members ($24 \times 7\frac{1}{2}$ RSJ) in both flanges of the buttresses, vertical members (10" x 10" UCS) in wall corners, each side of main doorways, on the centre line of the end walls, and horizontal members below each floor. Difficulty was experienced at the junction of the buttresses and the core where the structural steel, prestressing cables, and reinforcing in two planes intersect. Fig. 9 gives an indication of this, although some of the reinforcing has been omitted for clarity.

(c) Prestressing Steel. Approximately 100 sq. ins of mild steel (equivalent) was required in the tension zones of the buttresses. As it was not desired to increase the wall thickness above 24 ins. prestressing cables were used because of their high strength to area ratio, and also because they offer an efficient anchorage detail.

(d) Calculation of Steel Areas Required. In designing for both shear and bending, a potential 45° crack was considered. (Refer Fig. 8). This necessitated the extension of the flexural reinforcement approximately one storey plus anchorage distance above the level of the moment considered. This may be conservative but tests published to date, although only on single storey shear walls, exhibit cracks sloping at 45° or steeper.

To encourage the more rapid forms of construction, design details were made to allow lift slab and climbing form methods to be used. However, the successful tenderer preferred, and is using the conventional forms of construction.

4. COSTS. This building is very high by New Zealand standards yet the extra height together with the greater design seismic coefficient did not appreciably affect the cost of the building. Refer to Table below.

		Struct. Cost/sq.ft. (1964 rates)	Height	Ultimate Seismic Coeff.	Floor Plan
Aitken Street,	Wellington	36/6	200'	0 to 0.33	
Bowen Street,	Wellington	32/-	141'	0.15	
Bledisloe,	Auckland	37/-	122'	0.15	

5. CONCLUSION. Considerable thought was given to the analysis of the seismic core to take account of all those effects which seemed important. It was clear that axial strain must be considered, also that there was an optimum beam depth at which shear stress, seismic factor and storey height would be minimised.

In designing the seismic core, the use of structural steel proved to be an effective way of overcoming some of the inherent disadvantages in bar reinforcing. It minimised the weakness of splicing reinforcing in tension areas and it gave a greater ductility to the structure.

Although the use of structural steel may bring extra practical difficulties to reinforced concrete construction it has a further advantage in holding reinforcing in position prior to concreting in Wellington's frequent strong winds.

NEW WELLINGTON POSTAL CENTRE AND ADMINISTRATION BUILDING

This is the largest multi-storey building yet designed in New Zealand, with a floor area of 375,000 square feet, and a height of $16\frac{1}{2}$ feet between floors on the lower storeys. As shown in fig. 12 it can be separated into two main portions, the bottom five storeys, 4 bays wide, which function as a mail handling centre for the main part of the Wellington city area; and the upper portion of eight storeys, 2 bays wide, which is the national administration headquarters for the New Zealand post office organization. In addition, there is a complete basement storey containing boilerhouse and other services, a large conveyor, storage areas, private box lobby and an electricity substation.

The general form of this building is shown in the perspective view and typical section. The requirements that the postal centre storeys should allow utmost flexibility for arranging mechanical mail handling, and should enable overhead conveyors to run in any direction, automatically ruled out shear walls for resisting lateral forces, and required a spacing between columns of the order of 30 feet both ways. All lateral seismic forces have to be resisted by frames. To allow head room for overhead conveyors, all storeys in the postal centre were given a height of $16\frac{1}{2}$ ft.

For calculation of seismic forces, a triangular distribution of seismic coefficient was initially assumed, increasing linearly from zero at the base to 0.20g at the top heavy floor (i.e. 12th floor). The coefficient of 0.20g was applied also to all masses above the 12th floor.

Preliminary member sizes were calculated from these figures, and the D.S.I.R. analogue computer (5) was set up with the beam and column sizes represented in circuit components. The response spectrum for the El Centro 1940 earthquake was combined with the curves for several smaller earthquakes, scaled up to the same size, to obtain an envelope of all the curves. The envelope values were applied to the analogue set up, to obtain the building response for the first three modes of vibration, assuming all movement to be in the elastic range. Damping of 5% critical was assumed. The storey shears resulting from this analysis were divided by a ductility factor of 4, and then compared with the shears resulting from triangular distribution above. The design shear at each storey was taken as the larger of the two values as shown in table 1.

The step down in width at the 5th floor, from 4 bays to 2 bays, is considered to have increased the dynamic coefficients in the top stories to greater values than those for a uniform frame.

(3) "Side Sway in Symmetrical Building Frames" by N. Naylor. The Structural Engineer, April 1950.

(4) "Approximate Method of Analysing Coupled Shear Walls Subject to Triangular Loading" by R.J. Burns. Proceedings of Third World Conference on Earthquake Engineering.

(5) Adams, K.M., Morris, R.A. and Skinner, R.I., "An Analogue Computer for the Determination of the Earthquake Response of Buildings in Bending and Shear Modes". Proc. Second World Conference on Earthquake Engineering July 1960.

TABLE 1 - SEISMIC SHEARS APPLIED TO TRANSVERSE FRAME

Floor	Seismic Mass Attached to One Transverse Frame (Kips)	M.O.W. Code Coefficient (0 to 0.20g)	Horizontal Floor Force (Kips)	Seismic Shear (Kips)	Factor Dynamic Shear divided by Code Shear	Design Shear (Kips)
Roof	290	0.20	58.0	58.0	1.60	93
12	324	0.20	64.5	122.5	2.02	248
11	340	0.19	65.0	187.5	1.37	257
10	349	0.175	61.0	248.5	1.11	276
9	359	0.16	57.5	306	1.0	306
8	371	0.15	55.8	361.8	< 1.0	361.8
7	388	0.135	52.4	414.2	"	414.2
6	398	0.12	47.8	462	"	462
5	773	0.11	85.2	547.2	"	547.2
4	974	0.09	87.7	634.9	"	634.9
3	981	0.07	68.8	703.7	"	703.7
2	999	0.05	50.0	753.7	"	753.7
1	1034	0.03	31.0	784.7	"	784.7
Grd.	1062	0.01	10.6	795.3	"	795.3
Bast.		0				

Building periods were:-

	Mode 1	Mode 2	Mode 3	Mode 4
Transverse	1.52 sec.	0.74	0.53	0.34
Longitudinal	2.17 sec.	0.90	0.23	-

The penthouse frames were stiffened after analogue studies showed the preliminary design to give the fundamental mode of the penthouse frame approximately the same period as the second mode of the building.

Arrangements are now being made with the Earthquake Research Institute at Tokyo University for an investigation of the elasto-plastic response of the building with the SERAC computer.

Bending moments in the transverse frames were calculated on an I.B.M. 650 computer. Moments from dead and live loads were computed with a moment distribution library programme, while seismic moments were computed with a matrix package programme from slope-deflection equations. For calculating moments from vertical loading, the overall time required, including that for preparing and checking data cards, was no less than that required for manual calculation. For calculating moments from lateral loads, the computer cut the overall time down to less than half that for manual calculation, but a considerable amount of time was spent on preliminary training for the engineer who set out the data.

Because of the height and importance of the building, and the seismic history of Wellington, a form of construction giving good performance in a destructive earthquake was required. The form finally selected was a variation of the composite structural steel-reinforced concrete frame developed in Japan as structural steel was estimated to be too costly. The steel members in the typical Japanese form are built up by rivetting, but rivetted details are very expensive under New Zealand conditions, and in this building the members are built up as welded plate girders. Castellated webs replace the typical Japanese web system of spaced batten plates, in most members, to give an open web which can be fabricated economically by automatic welding. In the columns the steel skeleton consists of castellated plate girders crossed at right angles, as shown in figures 14 and 15. The castellations of each web pass through the holes of the web crossing it, and they can be temporarily moved over to one side of the holes to allow the other castellations to be welded by automatic equipment. Typical column and beam sections, and typical structural steel details are shown in figures 13 to 15.

Columns will generally be fabricated in two storey lengths, complete with beam stubs. In erection, each length will be connected to the length below by a splice at mid storey height, employing high tensile bolts and splice plates. Beam steelwork will be bolted to beam stubs by high tensile bolts, site welding being eliminated as far as possible. All the connecting bolts are friction grip bolts, and mating surfaces at bolted joints will be sand blasted to provide a reliable coefficient of friction.

The strength of the structural steel frame encased in concrete is calculated on the principles set out in the Japanese National Code. Approximately half the seismic loading is taken on the structural steel skeleton, and approximately half on the encasing reinforced concrete. Columns are designed in accordance with the Japanese Code, in that axial force is assumed taken by the reinforced concrete alone. All members are designed on a "working stress" basis, using elastic stress distribution over every member. For resisting shear forces, the castellations in the steel skeleton were calculated to act in the same manner as stirrups in reinforced concrete sections.

For reduction in member size and for economy, it was decided to fabricate the skeleton from the new B.S.968-1962 high yield steel with a yield strength of 22.5 tons per square inch. The specification for this material sets maximum percentages for chemical composition as follows:

Carbon	Silicon	Manganese	Chromium	Sulphur	Phosphorus	Mn + Cr	Cu
0.20	0.35	1.50	0.50	0.05	0.05	1.60	0.50

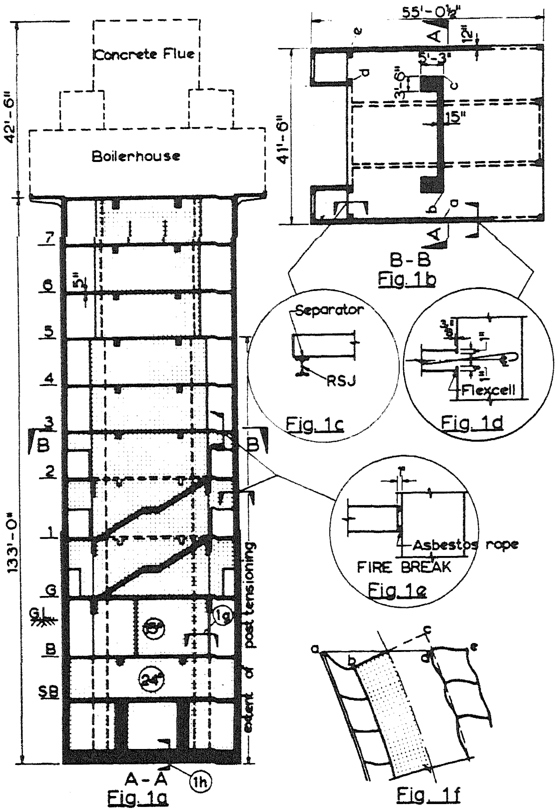
To avoid any risk of brittle fracture resulting from the welding of this material, a fairly comprehensive programme of preliminary procedure tests is being embarked upon, to determine preheats and other points of procedure required to maintain notch toughness in the welds and heat affected zones, and to check the performance of heavily restrained welds. The steel for the building will have a notch ductility of 20 ft lbs. Charpy at $-15^{\circ}C$ specified, an optional requirement in the 1962 B.S.968 specification. The performance of ultrasonic inspection and radiography in picking up faults will also be tested.

To keep down the overall height of the building, air ducts in the administration storeys are passed through 12 inch diameter holes in the transverse beams. A solid web reinforced with stiffeners is provided round each hole, designed to take the full member shear.

In the lower storeys, large perimeter ducts are run outside the exterior columns, above the top of the windows. Precast concrete facing panels outside these ducts and below them give the building a distinctive exterior appearance. (See fig. 12). Transverse branch ducts are accommodated under haunched up midspan sections of longitudinal beams.

Fixings for precast panels, window frames and partitions have been detailed to allow calculated interstorey deflections of up to $\frac{3}{4}$ inch. The same applies to stairs, which are constructed of precast treads and landings fixed on steel stringers.

In conclusion, it must first be pointed out that the composite structural frame system adopted has presented difficulties. Considerably more design work has been involved, drawings being required for both a structural steel frame and a reinforced concrete frame. Detailing at beam to column intersections has been difficult, as the structural steel joints have to allow clearance for reinforcing steel to pass through in 3 dimensions, and the whole assembly has to allow concrete to be placed and vibrated. However, it is thought that a good seismic performance and prevention of damage to non-structural components has been achieved economically in this building.



A-A
Fig. 1a

B-B
Fig. 1b

Fig. 1c

Fig. 1d

Fig. 1e

Fig. 1f

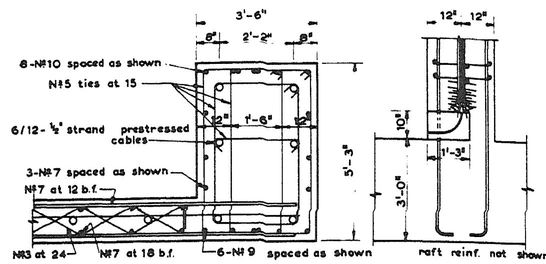


Fig. 1g

Fig. 1h

SCIENCE BUILDINGS - UNIVERSITY OF AUCKLAND

Fig. 1	Link Tower	
Fig. 2	Chemistry	Research. (centre)
Fig. 3	Chemistry	Research.

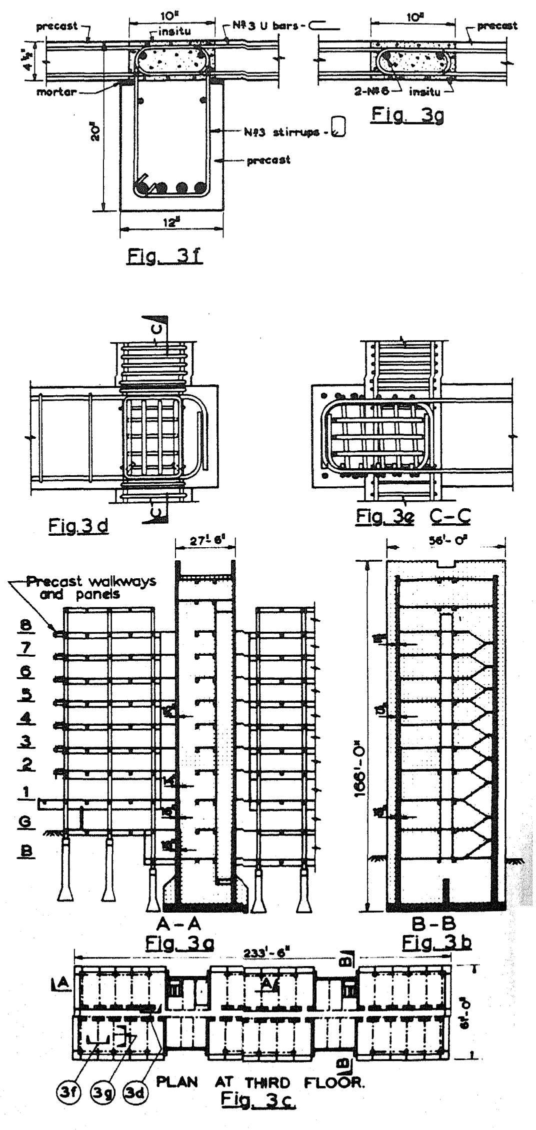


Fig. 3f

Fig. 3g

Fig. 3d

Fig. 3e C-C

A-A
Fig. 3a

B-B
Fig. 3b

Fig. 3c

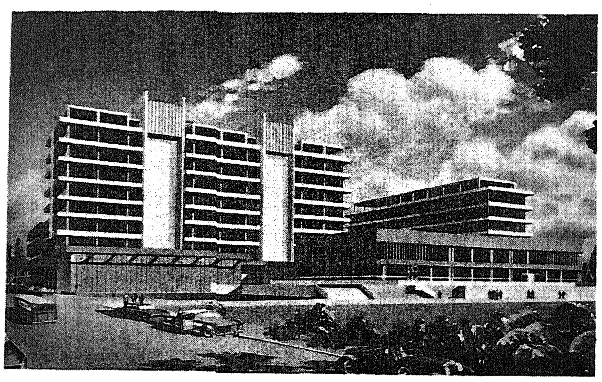
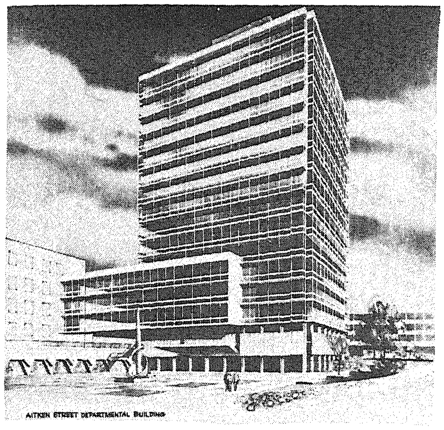


Fig. 2



Total Floor Area (tower only)	:	190,000 sq. ft
Structure (tower only)	:	36' x 60' central S.S.R.C. core with 12" waffle slabs simply supported at core and on steel columns placed at 12' c.c on window lines.
Height	:	209' 16 office floors
Foundation	:	3' raft on preconsolidated silty clays – design bearing capacity 3 T/sq. ft
Equipment	:	Forced ventilation with hot water convector heaters.

FIG. 4 : AITKEN ST. BUILDING
Artist's Sketch & Descriptive Outline

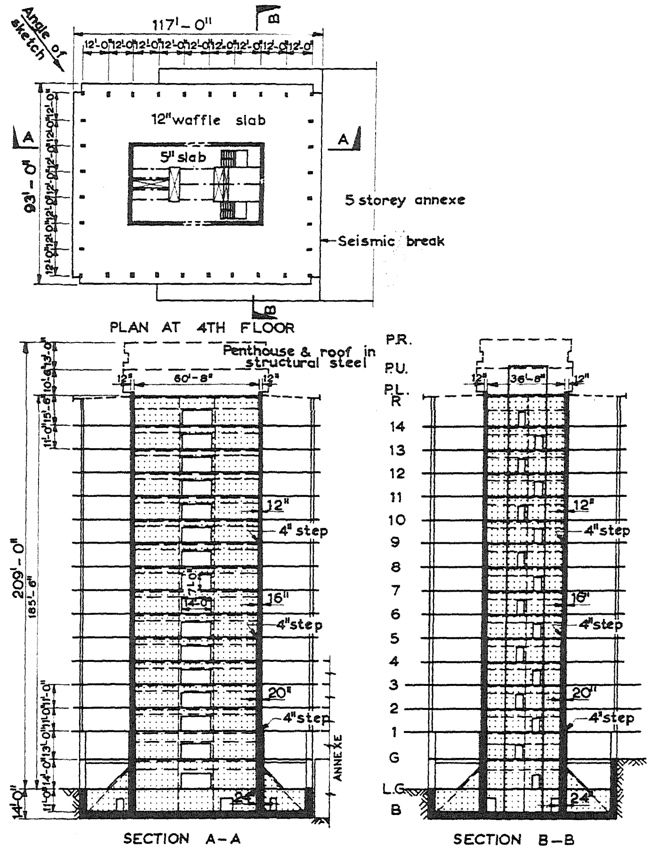


FIG 5 : AITKEN ST. BUILDING
Structural Outline

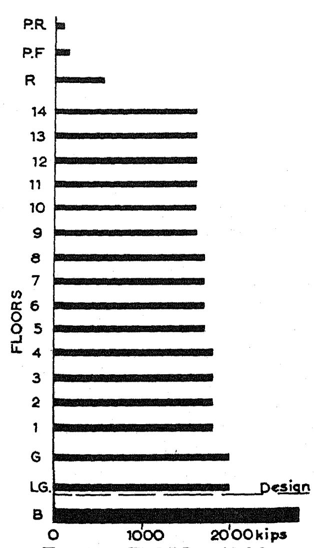


FIG. 6a : FLOOR MASS

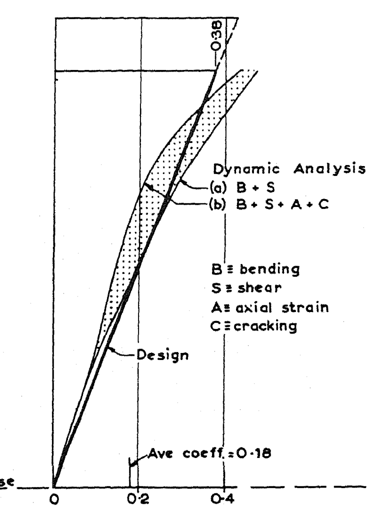


FIG. 6b: SEISMIC COEFF.

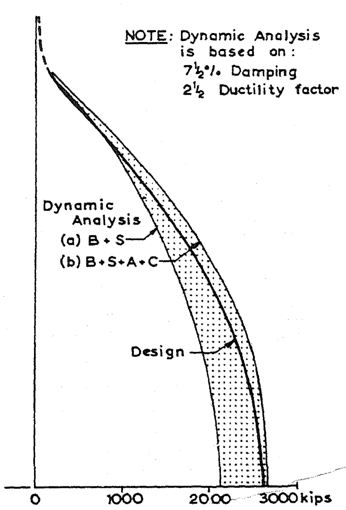


FIG. 6c : STOREY SHEAR

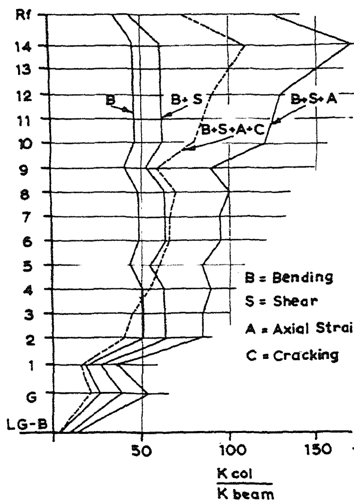


FIG. 7a : STIFFNESS RATIO

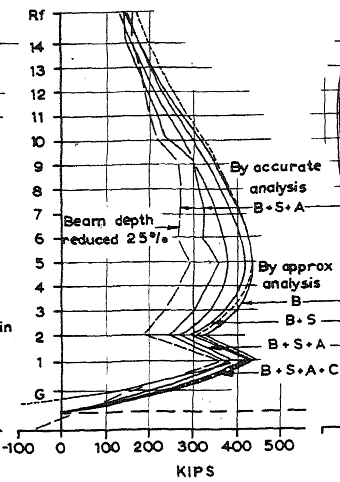


FIG. 7b : BEAM SHEAR FORCE

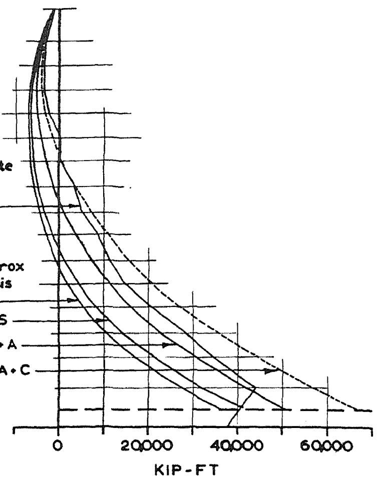
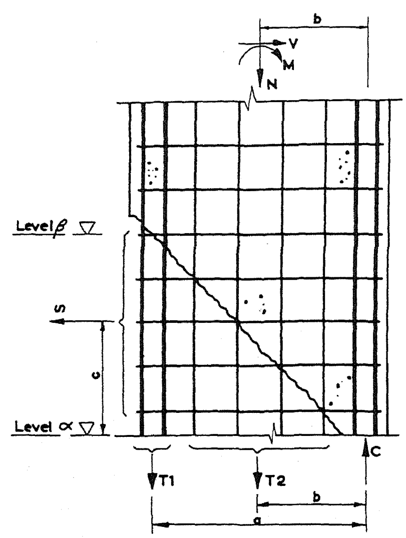


FIG. 7c : AVE. COLUMN MOMENT



From Statics $a.T1 = M\alpha - b.T2 - cS - bN > M_\beta$

FIG. 8: EFFECT OF CRACK ON REINFORCEMENT

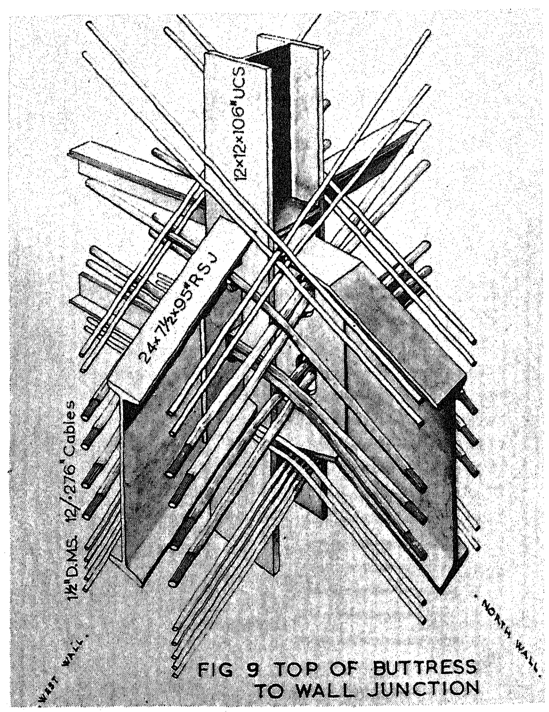


FIG 9 TOP OF BUTTRESS TO WALL JUNCTION

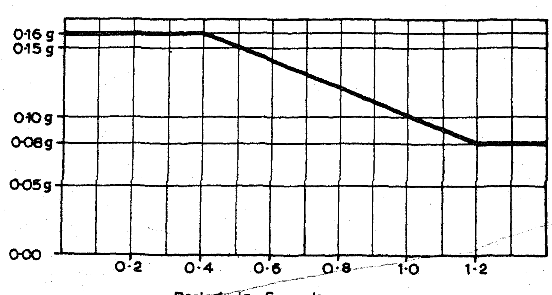


FIG.10: BASIC SEISMIC COEFFICIENT

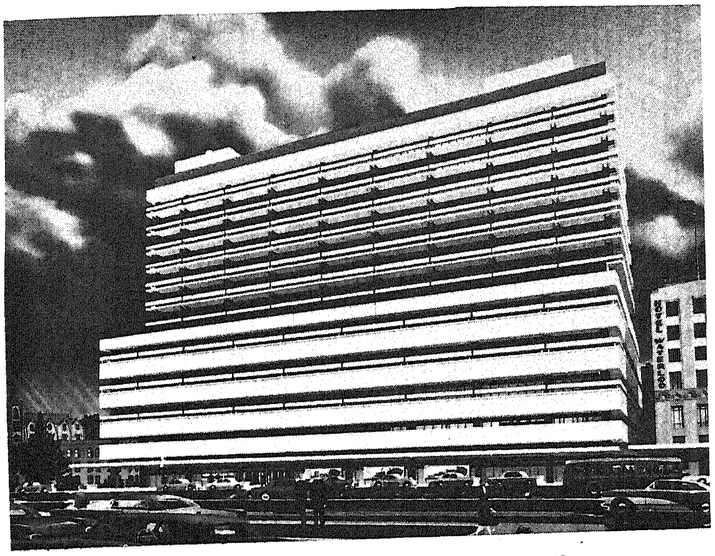


FIG.11 PERSPECTIVE VIEW OF POSTAL CENTRE.

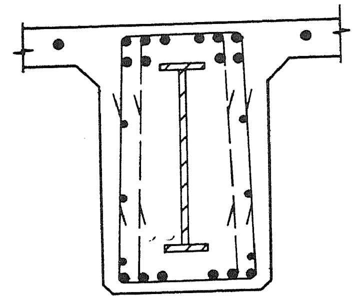


FIG. 13 TYPICAL BEAM SECTION

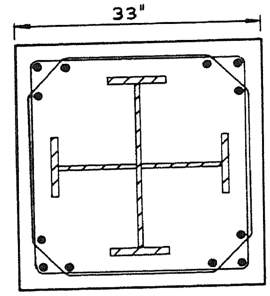


FIG. 14 TYPICAL COLUMN SECTION

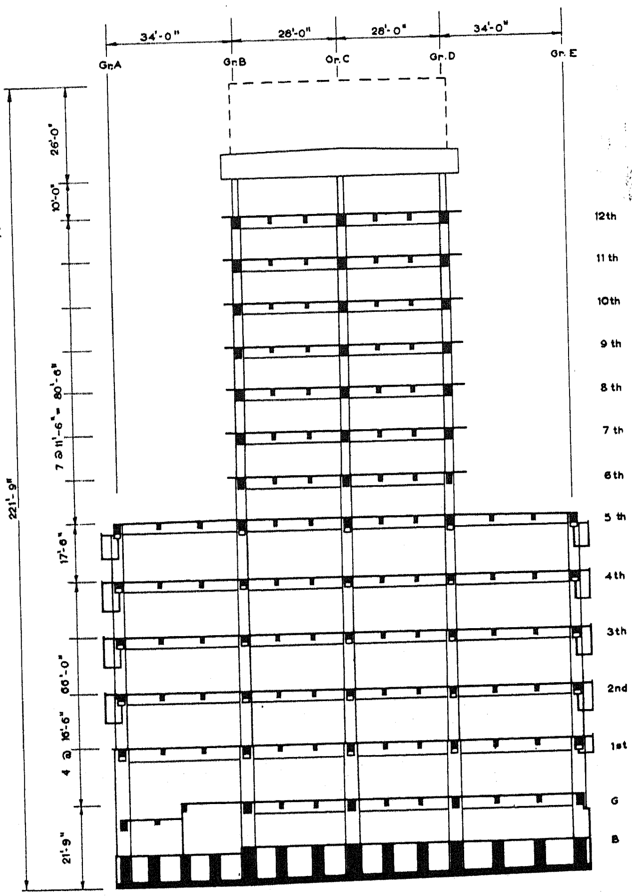


FIG. 12. CROSS-SECTION OF POSTAL CENTRE

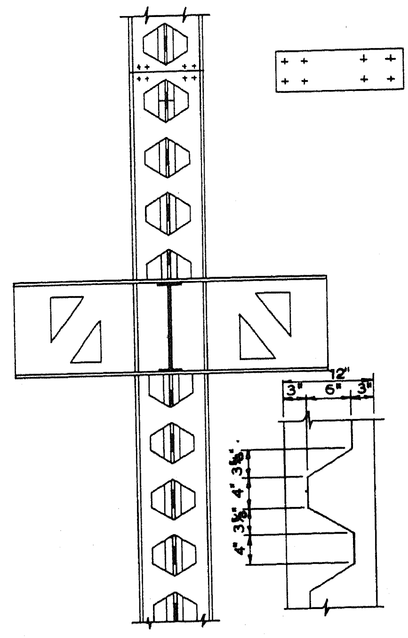


FIG. 15. COLUMN STEEL DETAILS.

DETAILS OF POSTAL CENTRE

MULTISTORY STATE BUILDING DESIGN IN NEW ZEALAND

BY J.A.R. JOHNSTON, O.A. GLOGAU, C.F. CANDY AND G.H.F. McKENZIE

QUESTION BY: E. ROSENBLUETH - MEXICO

Would the authors wish to elaborate on reduction influences, other than ductility, to give spectral ordinates smaller than obtained from normal records; and would they speak on torsional provisions used in the design of these buildings?

REPLY BY: J.A.R. JOHNSTON

Damping coefficients and ductility factors were first used about 5 years ago to reconcile the results obtained from dynamic analyses with the seismic coefficients commonly used in design in New Zealand although it was realised that many other factors lead to reduction in response such as:

(1) damping and ductility in the foundation subsoils

(2) inefficiencies in coupling between building and subsoil

(3) energy feedback from building to subsoil

(4) an assumption of instantaneous response regardless of buildings dimensions and

(5) the practice of using shear envelope curves rather than instantaneous shear response curves in design particularly so in computing overturning moments.

It would probably be more rational to use smaller damping coefficients and ductility factors in conjunction with a further arbitrary reduction factor but it is felt that results would not be greatly affected until the effects referred to above can be computed with reasonable precision.

REPLY BY: C.F. CANDY

All of the buildings described are symmetrical as a result of co-operation between Engineers and Architects at an early stage of the design.

To provide for accidental torsion a minimum eccentricity of 5% of the maximum building dimension was allowed for.

QUESTION BY: D.J.H. PERCY - NEW ZEALAND

How do you justify in the Postal Centre Building using the larger shear for every member from two analyses (1) linearly varying seismic coefficient based implicitly on a very simple dynamic model and (2) dynamic analysis with analog computer more rationally based on the actual building?

REPLY BY: G.H.F. McKENZIE

A dynamic analysis will give the correct building response only if the correct earthquake is applied. The earthquake spectrum used was an envelope of El Centro 1940 and several other earthquakes scaled up to the same size but we do not know whether a destructive Wellington earthquake would lie within this envelope. In view of the nearness of this building to an active fault, the possibility of a single large pulse has to be considered, and this could give larger shears in the lower stories than the vibrations applied in dynamic analysis would. Further points are that the reduction factor of 4, to allow for ductility, foundation coupling and other effects is only a very approximate one, and that the dynamic analysis is only an elastic one and not an elasto-plastic one. In view of all these considerations there appeared to be no justification for making the lower portion of the building weaker than the normal design for a building of the same vibration period under the Ministry of Works design code. The dynamic analysis gave a method of taking some account of the "Whip" effect in the upper part of the building due to higher modes of response, which gave appreciably higher shears in the upper stories.

The basis of design is supported by results just to hand, from an elasto-plastic analysis carried out on an equivalent system of masses on the Serac analogue equipment in Japan, by Dr. Osawa. This analysis shows the strength and ductility factors of the lower portion of the building to be in balance with those in the upper portion when the building is responding to various applied earthquakes - 0.33g and 0.165g El Centro NS 1940, 0.33g and 0.165g Taft EW 1952, 0.33g and 0.50g Sendai NS 1962. Two cases, 5% and 7% damping were examined.

QUESTION BY: J.I. BUSTAMANTE - MEXICO

The proper use of an earthquake input for the analysis of structures is of great importance. Several people, coming from different regions with different geological conditions, seem to favour the application of the N.S. component of the El Centro earthquake 18 May, 1940. I wonder if the authors would like to comment on this matter?

REPLY BY: O.A. GLOGAU

The 4 buildings described in this paper are founded on hard ground. N.Z. code coefficients are derived from an idealized envelope curve of a number of Earthquakes scaled up to El Centro intensity.

For results of a dynamic analysis using earthquakes other than El Centro I refer you to the reply by Mr. McKenzie to question by Dr. Percy.

It is further hoped to obtain the response of the Postal Centre to the application of a single heavy pulse.

QUESTION BY: Del VALLE E. - MEXICO

In the Chemistry Building you have rigid frames coupled with the shear walls in the transverse direction. Can you explain how you computed the period of vibration of such a system?

REPLY BY: O.A. GLOGAU

The transverse frames are coupled in parallel with the shear towers. Their stiffness is of a much lower order than that of the walls (refer to fig. 3g and 3b) so that they are unable to substantially alter the deflected shape of the towers. The period of the system is dominated by that of the towers. On the other hand stresses induced in the frames by the towers were checked. It should be noted that the connection between towers and frames is only through a 4½" slab spanning 10' 6". Thus in both directions we are concerned only with the effects of the translational movements of the towers and the propping forces of the frames. In fig. 3c the heavy rectangular shapes along each side of the corridor are duct holes and not stiff columns.

QUESTION BY: A.W. SMITH - NEW ZEALAND

(a) I understand that the M.O.W. code is similar to the draft Chapter 8 N.Z.S.S. 1900 in limiting interstorey deflections. Presumably the limits have been set on studies of secondary structural damage but sometimes it is necessary to increase building member sizes over those required for stress and normal code deflections, and the extra stiffness shortens the computed building period and increases the seismic loadings. If special measures are taken to isolate secondary construction would the authors consider small increases to computed frame deflections?

(b) It is reported from Anchorage that most elevators became unservicable when counterweights and hoisting machinery broke from their guides and fixings. What design criteria do the authors adopt for the fixings and loads applied to the main resisting structure for such equipment.

REPLY BY: J.A.R. JOHNSTON

(a) The limits for interstorey deflections laid down in the revised loading code are 25% greater than those in the M.O.W. code. In view of the fact that ductility reduces response but not deflections it is our practice to allow for at least three times the computed deformations in separating nonstructural components when these computations are based on reduced elastic response.

Minor variations to computed frame deflections should be viewed in this light.

(b) Lift machinery details are usually designed by service engineers, however it is considered that the usual basis of design is adequate for the main resisting structure but that fixing for counterweights guides etc. should be designed for at least twice maximum basis coefficient in a material of adequate ductility.