A BRIEF HISTORY OF DAMAGING EARTHQUAKES IN WELLINGTON CITY AND DEVELOPMENTS IN MULTI-STOREY BUILDING CONSTRUCTION IN NEW ZEALAND

by J.A.R. Johnston*

This paper describes Wellington City, the most populous centre in New Zealand's strongly active seismic zone, its earthquake experience especially during the Wairarapa earthquakes of 1942, its buildings and the extent and nature of the building damage resulting from those earthquakes.

The principal features of a number of multistoried earthquake resistant buildings designed by officers of the Ministry of Works are described, current trends in building construction reviewed and unresolved structural problems outlined.

WELLINGTON CITY

The city with its population of some 250,000 people is disposed along the foreshore of Port Nicholson, an extensive inlet at the south-western extremity of North Island and in the valleys and on the slopes of the surrounding hills. It is almost bisected by the Wellington Fault, a strong feature running slightly across the grain of the country and is practically contained between two others, the Ohariu Fault to the west and the West Wairarapa Fault to the east. These faults extend through North Island to the Kermadecs in the north and merge to the south into the Great Alpine Fault of South Island. Most of the commercial and industrial section of the town lies to the east of the fault at a distance of from one-quarter to three-quarters of a mile while the harbour is a depressed area terminating at the fault escarpment; see figures 1 and 2.

Apart from a few volcanic intrusions, the hills are comprised of greywacke and argillite for the most part severely contorted, extensively fissured and deeply weathered. The sediments in both harbour and valley floors are derived from the greywacke usually in the form of compact lenses of angular gravels, sands, silts and composites, casually deposited. Beach sands are fine, uniform in size and loosely compacted while the beach gravels are inclined to be coarse, shallow and loose.

Wellington was occupied rather sparsely by Maoris for several hundred years and by Europeans since 1840. Although there is some evidence of earth movements in Maori folk-lore the first recorded heavy earthquakes occurred in 1848 and even greater ones in 1855 which tilted about 1,000 square miles at the south-western end of the island and uplifted the city from 5 to 10 feet. This earthquake resulted in the first known loss of life and the destruction of many buildings including the first masonry ones. Wood framed buildings fared better than masonry so that the latter medium of construction found little favour for several years. However, when brick building began again, possibly as the result of the 1855 earthquake and of numerous smaller ones some, at least, of the new brick buildings were built with an appreciation of seismic problems. In such buildings floors were reasonably well tied to walls while steel members such as angles, railway irons and flats were introduced into the walls.

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Such precautions together with sound foundations, reasonable compartmentation and modest fenestration did produce moderately resistant structures. The majority of buildings were built to lower standards with poor compartmentation, large window openings, floating floors and roofs and tall heavy pretentious parapets but because of damage over the years many of these parapets have been reduced in height and varying forms of buttressing, strapping and tying applied to the buildings.

Towards the turn of the century cement mortars began to replace lime and rolled steel joists replaced the longer timber beams, while cavity external walls replaced solid ones. Internal walls became less frequent resulting in a type of brick building very susceptible to damage. About the same time concrete encased structural steel framing and reinforced concrete were introduced and the use of these materials expanded rapidly in the twenties with the demand for bigger and taller structures. Little if any attention was given to earthquake resistance until the onset of the Murchison earthquake in 1929 (intensity M.M.11, magnitude 7.8)(1) and the Napier earthquake in 1931 (intensity M.M.11, magnitude 7.9)(2). The heavy death roll in Napier of about 1½% of the total population and a correspondingly heavy capital loss led to the preparation of a report recommending the adoption of a General Earthquake Building Bylaw in 1931 and to the publication of a Model Building Bylaw by the N.Z. Standards Institution in 1935 which has since been developed into its present form as N.Z. Standard Model Building Bylaw, N.Z.S.S. No.95.

Provisions of N.Z. Standard Model Building Bylaw, N.Z.S.S.No.95(3)

The Bylaw is not compulsorily applied to Local Authorities but it has in fact been adopted by Government Departments, all cities including Wellington and nearly all of the smaller local bodies of which there are about 300.

The Bylaw provided inter alia for uniform seismic coefficients ranging from 0.08 to 0.16g for different classes of building, for the integration of building components, for crosstying foundation members, for a reduced live loading scale and increased working stresses under seismic conditions and for limiting the height of masonry buildings.

Later amendments provide for alternative equivalent coefficients varying from zero at the base to twice the mean coefficient at the top of the building and also for special coefficients applicable to parapets, chimneys and towers.

The concrete bearing wall part of the Code has been revised and prescribes working stresses for both horizontal and vertical diaphragms. The masonry part of the Bylaw has been extended and revised to cover reinforced brick and block work and to require more rational design methods. The timber and steel parts are in course of revision.

Earthquake resistant design developed as the Bylaw took shape but designers for several years aimed at producing earthquake resistant frames without regard to infinitely stiffer but casually located walls, fragile masonry partitions and other sensitive features. More attention has been given to the bracing effect of floors and walls during the last decade.

WAIRARAPA EARTHQUAKE JUNE 24TH, 1942

General

The principal earthquake of this series, magnitude 7.3, intensity M.M.8, and probably the most severe in the southern part of the island since 1855, seriously affected Masterton, a small town about 15 miles from the epicentre but the shock was felt widely resulting in appreciable damage within a radius of at least 60 miles.

Masterton is situated in a broad valley on coarse deep compact gravel and has few buildings over two stories. Most brick buildings were damaged in varying degree although the majority are still in occupation. Damage to residential buildings was slight apart from chimneys. It is probable that the nature of the subsoils minimised damage as most buildings were old, the standard of construction rather below average and the extent of damage not much worse than in centres much further removed from the epicentre. Other small towns at varying distances were similarly affected but in Otaki which is 60 miles from the epicentre a number of brick residential buildings were damaged, some to an extent nearing complete collapse. The buildings most severely damaged were sited on flat land lying between high stabilised sand dunes near the coast. In this locality the sands of the rapidly advancing shoreline are of quite recent origin and as there was evidence of fissuring and surface flooding it appeared that the damage was caused by settlement of the sediments which are known to be deep. Further, the most severely damaged buildings showed clear signs of differential settlements of the order of one to two inches.

DAMAGE IN WELLINGTON CITY FROM THE WAIRARAPA EARTHQUAKE

About 900 private buildings in the "Brick Area" or principal fire zone of the City were examined for the Earthquake and War Damage Commission which began a system of compulsory earthquake insurance of all private buildings in 1945, and numerous other buildings were examined at different times.

Most of these buildings were from two to four stories although an appreciable proportion approached the City limit of 105 feet. About half were situated on the weathered rock fringing the harbour or on the uplifted natural sediments at the head of the harbour. The remainder were built on reclamations although normally the foundations were taken down to the "old sea beach" by way of piles or pads at depths of up to 20 feet. Recent borings suggest that the natural estuarine deposits of sands, silts and gravels which extend below the old sea beach to depths of at least 200 feet have experienced a considerable measure of precompression.

The buildings in the principal survey were classified as follows:-

Type 1. Frame, reinforced concrete or concrete encased structural steel; floors and wall panels, reinforced concrete.

Type 2. Frame and floors as Type 1, bearing walls concrete.

Type 3. Frame and floors as Type 1, brick infilled panels.

Type 4. Brick bearing walls, Concrete floors.

Type 5. Walls as type 4, wood or steel and wood floors.

Type 6. Wood frame, plastered walls.

Type 7. Wood frame and wall finish.

Apart from structural details other information elicited included, type of foundation and subsoil, nature of occupancy, extent of damage, also notable hazards such as parapets, unreinforced chimneys, towers, abnormal slenderness, extensive glazing assymetrical bracing systems and the like.

A summary showing extent of damage follows:-

Building Type	1	2	3	4	5	6	7
No. Examined	131	49	15	29	522	27	123
No. Damaged	61	17	5	11	269	6	7
% Damaged	47	35	33	38	51	22	6

Types 1 to 3 were separated as it was anticipated that their behaviour would differ. Unfortunately absence of plans resulted in a measure of uncertainty in classification so that the lower percentages of damage for types 2 and 3 have no real significance.

Types 4 and 5 were considered separately for similar reasons. The lower percentage of damage for type 4 buildings was belived to be due to better integration and sufficient precompression in the walls to avoid failure in this earthquake of moderate intensity. However, there were sufficient signs of strain at lower levels in some of the buildings to raise serious doubts of their behaviour in a heavy earthquake. One similar building did collapse with disastrous results in the Napier quake.

Damage in type 6 buildings was mainly to plaster-work and in wood buildings generally, to chimneys.

It was evident early in the survey that buildings founded on the reclamation had suffered more than those located on natural ground. For instance, there was no serious damage in buildings west of Lambton Quay and Lower Willis Street which follow the old shore line. Similarly there was little damage for a block seawards off the old truncated spur at the junction of the same streets. Although most artificial fillings settled about 3 inches and one recent fill up to 16 inches, in filling depths of up to 20 feet, there was little evidence of damage to buildings resulting from differential settlement but a few tilted one or two inches towards streets or lightly loaded adjoining sites.

A few parapets were thrown down and many others damaged but earlier attentions to the more hazardous minimised damage.

Brick buildings were damaged for a variety of reasons but floating floors and roofs requiring the higher levels of walls to cantilever at least a storey were a common cause of damage. The walls of theatres and similar structures which were overlong or over-tall with indifferent lateral support bulged normally to their surfaces. Weaknesses induced by extensive glass facades in one, two or three walls without compensating bracing features resulted in obvious torsional strain while in some cases simple constructional errors such as cold mortar joints, unbonded corners or wall intersections, unbonded additions and the weakening effects of alterations or "modernising" treatments were the principal cause of damage. At least two substantial brick buildings were severely damaged by the impact of taller and heavier neighbours. A number of well designed buildings suffered no discernible damage, but recent demolitions have shown that some buildings exhibiting little damage owed that freedom to built-in reinforcement; in one case a complete substantial steel frame.

Most of the older concrete buildings had been noticeably strained by shrinkage and temperature movements prior to the earthquake and had low percentages of wall panel reinforcement. Old cracks in these buildings were widened and extended and new cracks formed while masonry partitions, fragile finishes and contents sometimes suffered. Figure 3 represents a typical building of this type. Two or three buildings with little bracing lost most of their glass. Almost invariably damage was most evident to the narrow elevations regardless of orientation. Broad buildings were seldom damaged on either elevation so that there is little to suggest that increased stiffness and shorter periods induced greater accelerations. Damage was also evident at the junctions of wings in most U shaped and L shaped buildings.

There was some slight evidence of damage in buildings designed to be earthquake resistant sufficient to suggest that a bigger or more prolonged shock would have caused significant damage, at least, to walls, partitions and finishes. Similarly there was sufficient evidence of extravagent movements at the higher levels of the taller buildings of rather slender dimensions to suggest that a similar experience in a big earthquake would be bound to alarm the occupants and damage most of the contents.

DAMAGE TO RESIDENTIAL HOUSING

The typical New Zealand house is wood framed with wooden floors and external boarding, concrete piles, gypsum-plaster internal linings and corrugated iron, corrugated cement-asbestos or tiled roofs. Alternative wall coverings are brick veneer, stucco and cement-asbestos sheets. Chimneys may be either concrete, brick or pumice block with the shafts reinforced. Older buildings usually have wooden piles, wooden linings and unreinforced chimneys.

Earthquake damage is normally relatively slight even in the event of heavy shocks and comprises damaged chimneys, cracked plaster or veneers, cracked drainage pipe connections and occasionally collapsed pile foundations.

In Wellington the City Engineer estimated that 20,000 chimneys were damaged and at least 5,000 buildings or 25% of the total. It is evident that although individual earthquake losses were moderate the total was high.

As a matter of interest, winds in Wellington which have been recorded up to 112 m.p.h. cause little damage to buildings.

FUTURE SURVEYS

In future surveys an endeavour will be made to separate buildings according to the principal causes of damage, namely:

(a) insufficient wall anchorage at roof level.

(b) foundation failure

and (c) superstructure failure uncomplicated by differential settlement.

Foundation failure is often difficult to distinguish with certainty as suitable reference marks are not usually available. It is advisable in new construction where some settlement is likely to establish bench marks and check levels until settlement is completed and again following earthquakes.

MULTISTORIED GOVERNMENT BUILDING

The Ministry of Works is interested in almost all public building in New Zealand and is responsible for the design and construction of a variety of structures for most Government Departments of an annual value of about £5,000,000, and of State flats and housing of similar value. The Department has been continuously engaged in building for about 80 years. A description of a number of multi-storied buildings designed by officers of the Department follows:

GENERAL: Almost all post war multi-storey departmental buildings have been built in reinforced concrete to suit the national economy as steel is imported and costly whereas concrete aggregates and cement are freely available. Concrete also is the least expensive of the available fire resistive materials and seems likely to behave more favourably in earthquakes than alternatives such as pumice, vermiculite, perlite or asbestos concretes which are usually applied in the form of precast blocks or sheets.

Departmental Concrete specifications require a minimum compressive strength of 3000 p.s.i., a minimum cement content of 560 pounds per cubic yard for concrete mixed in situ and 514 for plant mix with maximum water-cement ratios of 0.60 and 0.66 respectively. Most concrete in the principal centres is mixed in central plants where ingredients are controlled for grading and are weight batched.

Concrete is almost invariably placed with high frequency immersion type vibrators. Generally wooden forms are used although sometimes patent metal forms have been used for part of the work. Where "fair-faced" concrete finish is required it is usual to use resin bonded plywood shuttering. Fairfaced that is "off the boxing" finish has much engineering merit as it does facilitate repair of cracking which may develop subsequently from any cause.

Live loadings for floors are very similar to those used elsewhere and due allowances are made for floor finishing, false ceilings and partitions. Provision is also made in design for a lateral force corresponding to 0.10g applied uniformly or for an equivalent force determined from a triangular loading system.

The principal working stresses used are:-

Compression due to flexure	1200 p.s.i.
Compression due to axial load	900 p.s.i.
Diagonal tension due to shear	90 p.s.i.
Tension in reinforcement	18000 p.s.i.

Working stresses up to 33⅓% in excess of the foregoing are permitted for combinations of wind or seismic and other stresses.

Except for Wellington Departmental Buildings plain round mild steel reinforcing rods in accordance with B.S.S.785 have been used but in that case mild steel deformed bars complying with A.S.T.M, A 305 were adopted to permit the use of larger bars with better bond characteristics. Deformed bars will be more frequently used in future work. (a)

As far as possible light weight roofs framed in structural steel have been used to lessen costs and to reduce the effects of seismic loading. Similarly light weight partitions have been adopted whereever possible.

An endeavour has been made to ensure reasonable stiffness and small inter-storey deflections to minimise damage especially secondary damage to windows, partitions and finishes. Shear walls or deep membered frames have generally been used as they may be adapted to satisfy functional requirements and are relatively economical.

Some examples of typical buildings follow:-

WELLINGTON DENTAL SCHOOL - Fig. 4a: Prior to 1937 and following the Napier earthquake all of the bigger Departmental Buildings were designed with concrete encased earthquake resistant steel frames, reinforced concrete floors and reinforced concrete walls arbitrarily reinforced with ½% of steel in both directions. As New Zealand imports steel costs are relatively high. Sections are also difficult to fabricate for the bigger buildings as sufficiently heavy sections are not rolled. An endeavour was made in this design to economise by using reinforced concrete throughout and by using walls for bracing purpose. With these ends in view the Dental School was designed with transverse shear walls and deep membered reinforced concrete longitudinal walls having spandrel beams of wall thickness extending from window sill level to window head in depth and with vertical wall sections extending for the full available width between openings.

This building experienced the 1942 earthquake but was undamaged except for possibly very slight strain at the horizontal construction joints in the transverse walls in the lower storey.

(a) Deformed Bars will be used for all work from July 1961.

AUCKLAND DEPARTMENTAL BUILDING - Fig. 5: A number of schools, hospital and institutional buildings were built in two to four stories with shear walls but none of these presented much difficulty as the functions of the buildings lent themselves to a free use of shear walls in combination with either reasonably centrally positioned longitudinal shear walls or stiff external deep membered frames.

The design of an office building 306 feet in length, 57 feet in width and 112 feet in height was commenced about 1950. Functional requirements permitted the use of shear walls at the ends in combination with two almost centrally placed transverse walls and one longitudinal wall. As the minimum foundation depth was about 20 feet and as one face of the building was alongside an old tall brick building on shallow foundations caissons were used instead of piles or pads. To obtain flexibility in pad diameter to suit loading variations 8 and 10 feet barrels were used with provision for undercutting beyond the barrels from 6 inches to 3 feet permitting a range in pad diameter from 8 feet to 16 feet. Costs compared favourably with those for footings or piles if such methods of founding had been practicable.

End reactions from the shear walls were quite heavy and these were balanced by way of the ground floor slab with the lateral forces assumed to act at caissons centres in proportion to the caisson loadings. The ground floor slab with its marginal beams was designed as a deep horizontal beam to serve that purpose.

Provision was made in the early design stages for 20% of the seismic loading to be taken by the framing members. Subsequent computations indicated that one centrally placed frame would be required to take a loading of almost this magnitude due to floor slab deflection between the most widely separated shear walls.

As caisson loading intensities were reasonably uniform and depths constant, a uniform value was assumed for rotation at ground level.

The central transverse walls were stiffer than the end walls above 2nd floor levels but over the lower stories the reverse was the case necessitating a heavy shear transfer from the centre to ends. The second floor slab was designed accordingly with some adjustment in the floors immediately above and below.

Difficulties were experienced during design as the result of the vertical continuity of door openings through the transverse shear walls. Steps have been taken since to avoid or minimise this trouble by staggering openings or otherwise avoiding such openings arrangements.

WELLINGTON DEPARTMENTAL BUILDING - Fig. 6:[4] This building is 215 feet long and 83 feet wide with 10 stories above ground and two stories largely below and is braced with two towers located in the central of 3 transverse bays and the second and fifth of the six longitudinal bays.

The towers which measure 32 feet overall in each direction are shaped like conventional steel stanchions normally buttressed at the base. Webb and flange thicknesses vary from 36 inches each at the base to 10 inches and 8 inches respectively at the top. The towers which are

165 feet high each support a vertical load of about 3750 tons at ground level. The corresponding seismic load of 750 tons is applied about 70 feet above ground floor level so that the eccentricity at that level is approximately 14 feet. The computed deflection at the top under seismic loading conditions is 1.5 inches. Consideration was given to the effect of oblique loading, and adjustments in reinforcement made as required.

The building is rafted on a slab 4 feet thick at a depth averaging about 20 feet on a previously excavated site in subsoils consisting of sands, silts, clays and gravels and composites. Unconfined compressive strength of the subsoil varied from 15 p.s.i. to 96 p.s.i. but the lower values were limited to small lenses at some depth. Consolidation tests resulted in consolidation indices of 0.02 to 0.09 for loadings in the contemplated range.

This type of design avoided most of the difficulties experienced with longer shear walls and permitted a minimum of conflict with functional requirements. There are no openings in either flanges or webs of the towers while lift wells, stair wells, lavatories and the like are conveniently located in the spaces between flanges and webs.

The towers support a relatively large proportion of the vertical loading and have none of the disabilities of external shear walls which are often lightly loaded vertically and required to support heavy lateral loads thus requiring some attention to avoid the effects of partial uplift on the external faces of the building.

MULTI-STOREY FLATS AUCKLAND AND WELLINGTON - Fig. 7.

These buildings about 192 feet long 33 feet wide and 100 feet high are quite slender having a ratio of 3 : 1 at ground level and a rather greater ratio above 1st floor.

The flats above 1st floor are two storied enabling intermediate floors to be framed in wood as are all partitions within the flats, an arrangement which reduces the weight of the building by 20% while maintaining the fire compartmentation of each flat.

Openings in the longitudinal walls are staggered and the reinforcement arranged in diagonal bands about 4 feet wide with supplementary reinforcement placed in the usual positions around the openings. If openings had been aligned vertically the depth available would not have permitted effective horizontal reinforcement. As a matter of interest these walls although long have not yet shown signs of shrinkage cracking.

The Auckland building was founded on a stepped raft at average depth of about 20 ft on the sound Waitemata sandstones and mudstones. The Wellington building extends over a deep filled gulley and is supported on piles cast in situ varying from 16 to 45 feet in depth sunk to weathered greywacke.

The Wellington building has been instrumented with accelerometers at three levels also strain gauges at the lower level. (b)

(b) Period from formulae $T = 0.05\ H/D = 0.90$ secs. Period recorded from wind gusts and small earthquakes 1/3 sec.

DUNEDIN DENTAL SCHOOL - Fig. 8: This 5 storied building is constructed with two transverse 28 foot bays spaced longitudinally at 14 feet to satisfy the rather exacting functional requirements. Originally it was intended to use both transverse and longitudinal shear walls but it was found that the adoption of a deep membered central longitudinal frame simplified foundation construction very considerably. Transverse beams were tapered from outside to centre to concentrate load on the central frame, an arrangement which lightened the exterior frames and stiffened the central frame. Beams in the central wall were of maximum depth that is from door head to floor slab at the lowest floor level with reducing depths at higher levels. Columns in the central frame were designed with their principal axes in the longitudinal direction.

The building was founded on bulb piles at a depth of 20 feet on a heavy consolidated layer of conglomerate about 25 feet in depth overlying estuarine sands and silts.

PRINTING WORKS, WELLINGTON - Fig. 4b:

This building has been designed with 30 foot square bays, transverse shear walls and longitudinal frame bracing. It has five working floors and a basement. In general upper storeys will be machine loaded with some provision for storage and the basement will be used for heavy bulk storage. A survey was made of existing plant to arrive at an appropriate design loading and it was determined that lower live loads were permissible with large panels and that even allowing for heavier future equipment a uniform live loading of 250 p.s.f. was adequate. The storage basement was designed for 600 p.s.f.

The storey height varies from 14 feet to 17 feet 6 inches, floor to floor, to allow for air conditioning and operating equipment (fork-lift trucks) so that a shallow beam system was used to minimise column height. The beams in each direction at first floor level consist of three stems each 18" in width by 24" in depth at 3 feet 3 inch centres with the interspaces between intersecting beams filled at the column head and may be likened to a broad shallow beam 9 feet 6 inches wide by 24 inches deep reduced in width to 4 feet 6 inches for the central two-thirds of its length. Slab thickness is 6 inches. This arrangement results in comparatively low dead and live loading and low storey heights which all tend to reduce inter-storey moments and deflections.

In view of the nature of the buildings relatively high inter-storey deflections were permissible as only windows and stairs require appreciable freedom.

BUILDINGS IN COURSE OF DESIGN - Fig. 9:

Plans of four other buildings are shown with balanced minimal bracing systems. Two have central bracing systems, one a central system and end walls and the other two end walls and a central wall located to suit the particular functional requirements. As far as possible wall openings have been staggered to avoid consecutive weaknesses.

The longitudinal walls of the Science Block, Fig. 10, will be developed with deep spandrel beams to develop a deep frame action.

Services of which there are many rise vertically between columns against the blank wall spaces and cross under the floors in alternate slab panels.

CURRENT BUILDING DESIGN:

Contemporary building design favours tight planning for reasons of economy, rather severe geometrical shapes, open fenestration and the use of a wide range of subsidiary materials. Most of these considerations assist in arriving at simple engineering structures capable of rational analysis and fine design. The Bylaw tendency is to reduce floor live loadings and increase working stresses for most building materials. Structural members are consequently smaller and interstorey deflections greater in comparison with the older buildings. Damping factors must also be much lower so that it would appear unwise to reduce seismic coefficients until more modern buildings have been subjected to full scale shocks.

Officers of the Department of Scientific and Industrial Research (5) have already developed an analogue for computing the response of varying types of building frames when subjected to strong motion earthquake and are in process of developing another instrument to deal with shear walled structures. They have also installed a number of strong motion instruments throughout the country so that shortly it may be possible to compute responses for a wide range of structures using New Zealand strong motion records.

Precast concrete units, prestressed concrete and shell structures are gradually gaining acceptance, but as yet are barely competitive with the older forms of construction.

SOME UNRESOLVED DESIGN PROBLEMS:

There is little doubt that subsoil behaviour during earthquakes has a profound influence on building behaviour. A limited amount of preloading has been done on silty sites and subsoil replacement or compaction in others while equipment is now available to compact loose sandy soils by vitro-flotation. A few rafts have been overcompensated that is excavation is balanced against estimated actual dead and live loading plus a proportion of the seismic loading. However, much remains to be done as it is felt that the concern of designers of buildings on poor soils is largely one of foundation integrity.

Although there are many methods for computing stresses in deep membered frames the subject is still an obscure one and an endeavour is being made to carry out some model tests at least for the more common types of frames in attempt to arrive at simple rational method of design.

CONCLUSION:

In all buildings designed by Ministry of Works consideration has been given to minimising structural damage in addition to safeguarding life.

Mass has been reduced to a minimum consistent with the availability of materials and the centre of mass maintained at its lowest possible level.

Simple bracing systems with either shear walls or deep-membered frames in combination with soft frames have been used in preference to combinations of dissimilar stiff frames to avoid uncertainties in shear distribution.

Wall bracing systems have been sufficiently well balanced to avoid serious torsional stresses arising from structural unbalance and generally one system has been furnished with a sufficient lever arm to absorb accidental torsional effects brought about by variations in loading, variations in subsoil conditions and other factors beyond control with only modest changes in shear distribution. Internal systems of bracing have been preferred as the possibility of uplift is lessened and there is less interference with the normal functions of buildings.

Buildings or sections of buildings have been separated to permit deflection free from disturbance from neighbouring structures.

Buildings with rigid frames and no shear walls have been used, but rarely, as comparatively few buildings are free from occasional stiff walls, panel infillings and other fragile components required for other reasons and likely to be damaged by interstorey deflections.

Finally, in all buildings the principal consideration has been to find the simplest structure consistent with the functions of the building and its services as even in that form it is likely to be more complex than most engineering structures and less amenable to rational design.

The Author wishes to thank Mr F.M. Hanson, Commissioner of Works, for the opportunity to present this paper, and Mr H.L. Hume, Chief Civil Engineer, and Mr B.W. Spooner, Chief Designing Engineer, for their interest and numerous other Departmental Officers for their assistance.

(1) F.W. Furkert, "The effects of earthquakes on Engineering Structures". Proc. Inst. C.E., 236 (1932–1933).

(2) Report of the Hawkes Bay Earthquake, Bull. No. 43, N.Z. Dept. of Scientific and Industrial Research.

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(5) M.J. Murphy, G.N. Bycroft, L.W. Harrison, "Electric Analogue for Shear Stresses in a multi-storey building". Proc. World Conference Earthquake Eng.

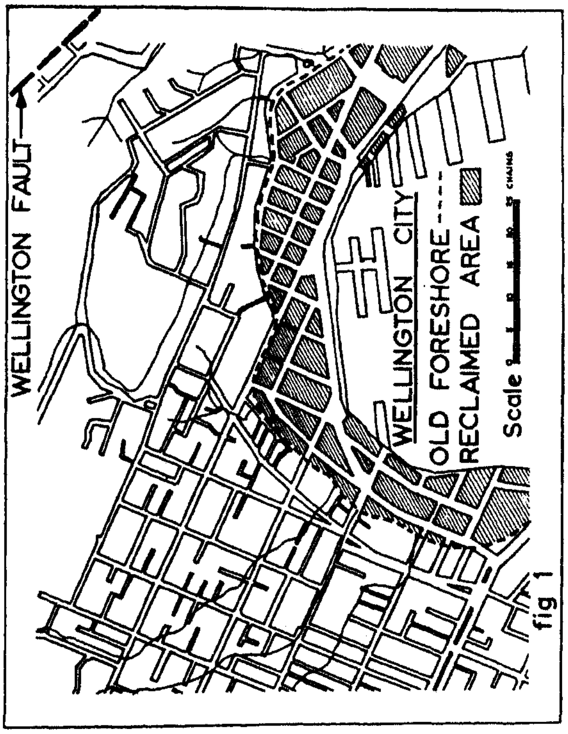

fig 1

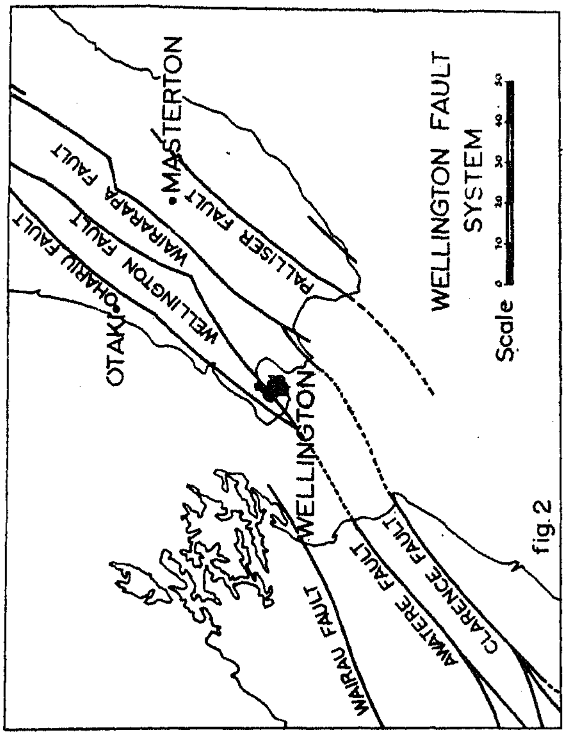

fig.2

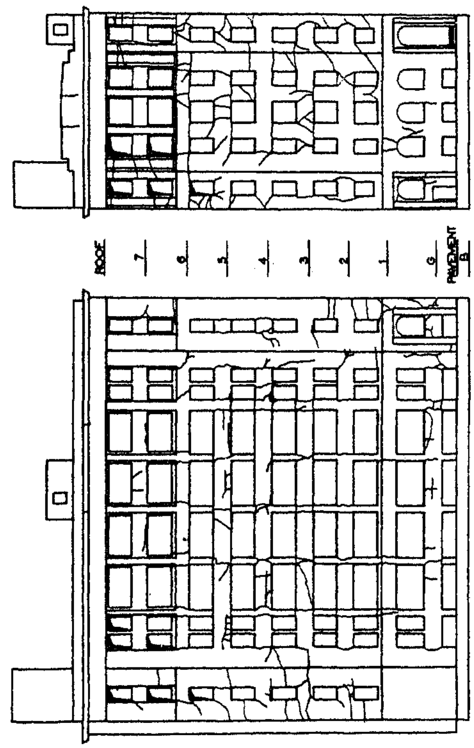

fig.3.a TYPICAL DAMAGED TYPE J. BUILDING

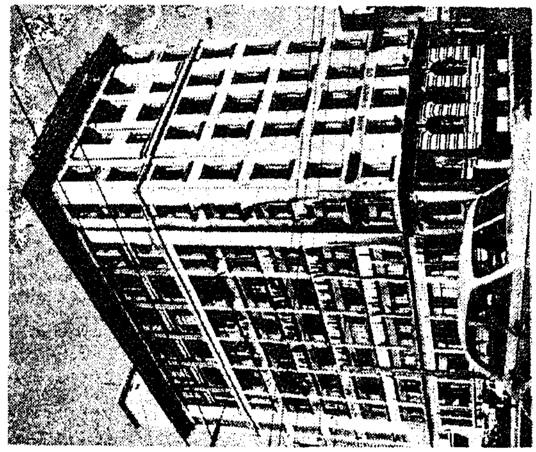

fig 3 b

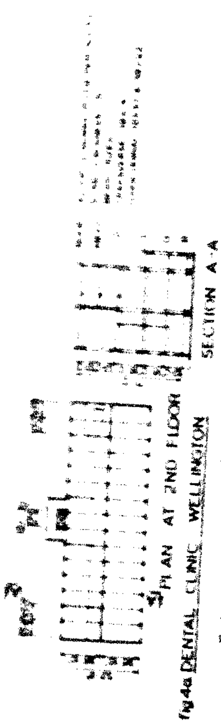

fig 4a DENTAL CLINIC WELLINGTON

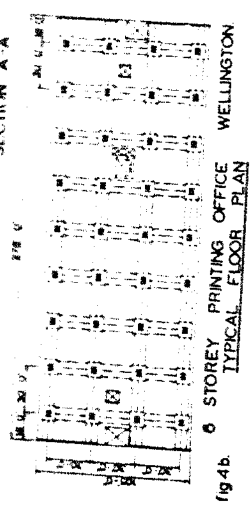
fig 4b. 8 STOREY PRINTING OFFICE WELLINGTON
TYPICAL FLOOR PLAN

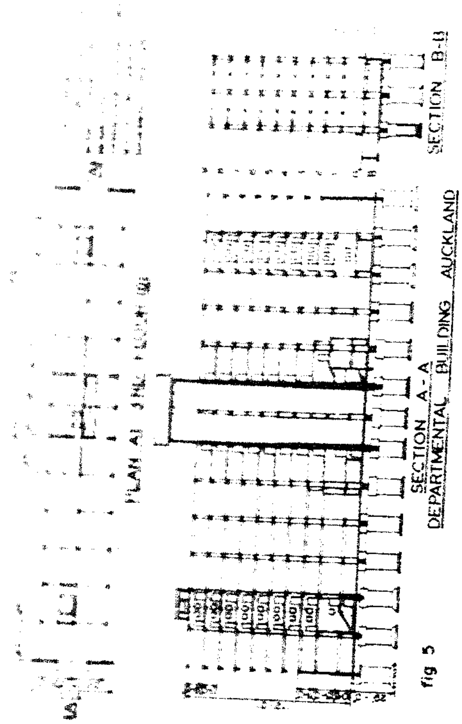

fig 5 DEPARTMENTAL BUILDING AUCKLAND

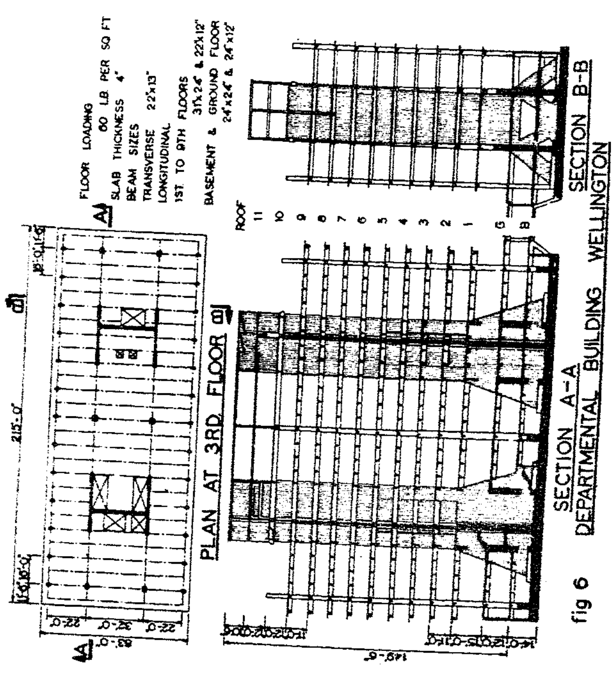

fig 6 DEPARTMENTAL BUILDING WELLINGTON

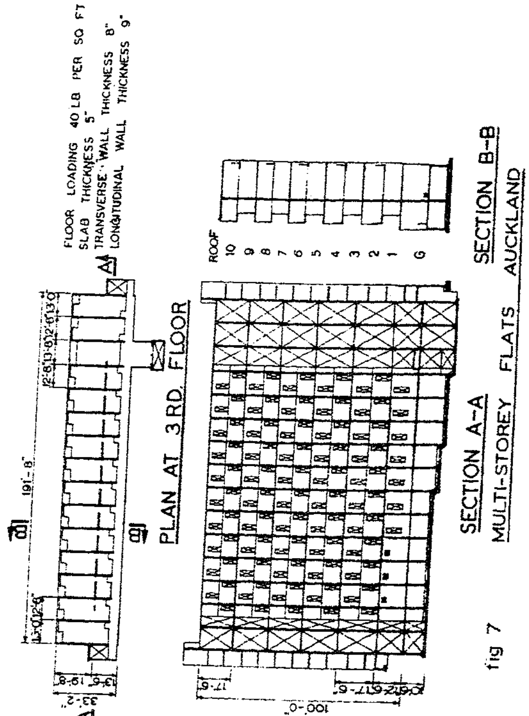

fig 7 MULTI-STOREY FLATS AUCKLAND

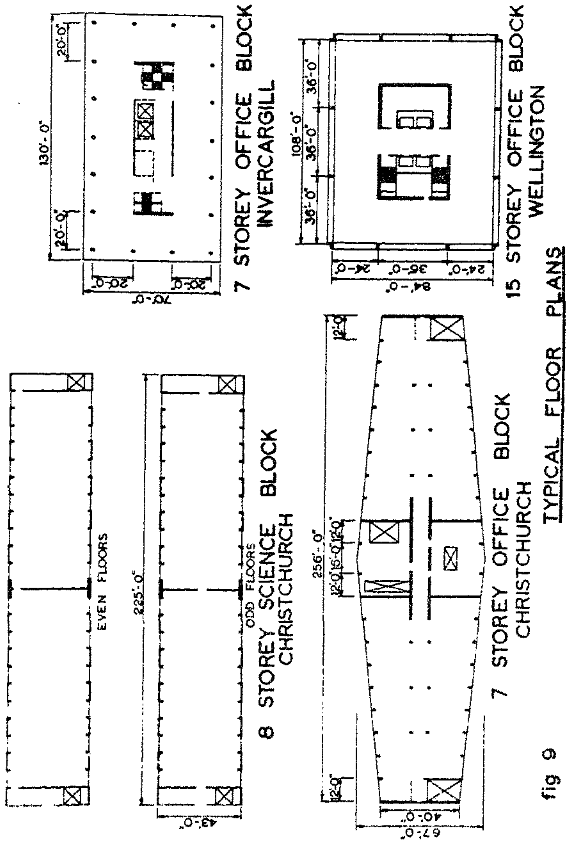

fig 9

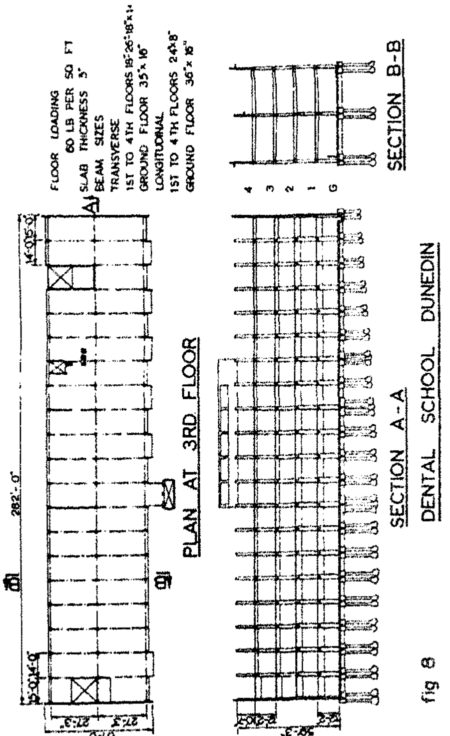

fig 8

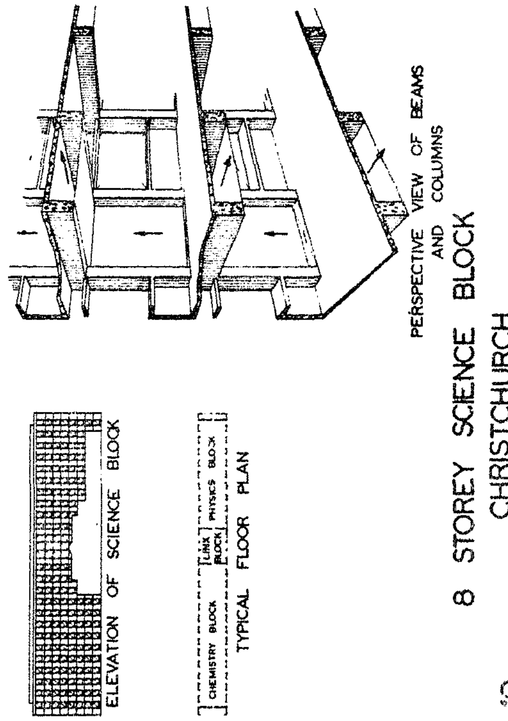

fig 10