NEW ZEALAND PARLIAMENT BUILDINGS : BEEHIVE

by

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1.0 SYNOPSIS

Aseismic analysis and design of this circular shear core building now being erected are described. Special attention was given to achieving ductility by suitable choice of material, geometry and detailing. As an added safety feature the shear core system was designed so that it will continue to function even if, under extreme cycling overloads, local damage should cause changes in its initial force resisting mechanism. Gravity forces are carried by the shear core, and perimeter columns which are vertical to 4th floor and then rake inwards to the roof. A prestressed concrete ring beam counterbalances the resulting horizontal thrust at 4th floor.

2.0 INTRODUCTION

The building is to provide accommodation and other facilities for the Prime Minister and his Cabinet. Because it is an addition to the existing Parliament Buildings in Wellington the structure is unavoidably placed on a site which is located within ½ mile of an active seismic fault, and even closer to less active ones. A major earthquake is a high probability during the life of the structure.

3.0 DESIGN CODES

Preliminary design commenced in 1964. The bulk of the design effort took place in the period 1968-69 in accordance with the C.O.P. for the structural design of public buildings, PW 81/10/1:1968. The reinforced concrete requirements of this code were very similar to those in the current N.Z. Standard Draft 3101P:1970, "R.C. Design". The seismic code loads were those of NZS 1900 Ch. 8 1965.

4.0 THE BUILDING

4.1 Choice of Structure. Refer to fig. 1. Architectural, functional and aesthetic considerations determined the shape of the building but the circular form was also eminently well suited for development into an efficient earthquake resisting structure. National policy at the time precluded the use of imported structural steel so that the choice of structure lay between a reinforced concrete shear core or reinforced concrete frame. A shear core offered a number of advantages:

i) Greater stability in the event of major ground displacements.

ii) Limitation of non-structural damage using less expensive and simpler detailing. Also many of the concrete walls required for service ducts, lift shafts and fire compartments were able to double as structural elements.

iii) A framed structure would have required radial as well as perimeter

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seismic members to avoid unacceptable member torsions and this would have introduced the complexity of concurrent beam yielding.[1]

vi) Greater stiffness with smaller, and hence to the occupants less alarming, movements in the frequent moderate earthquakes.

Real or apparent disadvantages of a shear core are:

i) The shorter period of the shear core required about 25% higher code loadings than in a frame. However, the likely difference in damping would probably lead to near identical real response.[2] Also, a design based on near peak response of the presumed site spectrum is less vulnerable to spectrum uncertainties than one taking advantage of a somewhat longer period.

ii) Uncertainties in coupled shear wall performance at the time. Those associated with the alternative R.C. ductile frame have since proved to be comparable (e.g. joint shear design).

5.0 THE SITE

The site is geologically young and has been subjected to 20 events MMVI or higher, including two of MMX (Richter $7\frac{1}{2}$ or 8) in the last 132 years.

Borings to a depth of 140 feet showed the soils under the building to be predominantly weathered and decomposed greywacke alluvium, consisting of very random layers of dense weathered gravelly materials, stiff to hard silts and silty clays of low plasticity.[10] Settlement is not a problem due to over-consolidation[5] and removal of 3.8ksf of soil. The adopted allowable bearing pressure of 6.7ksf for load factored long term loads and 10ksf transient loads provides a large factor of safety against failure.

6.0 FOUNDATION

Following consideration of many alternatives a flat slab 155' in dia. and 9' thick was adopted. Thus the raft used the whole weight of the building to resist overturning from the shear core, minimised foundation uplifting and allowed full use of the subbasement outside the core. The thickness was largely determined by the limiting shear stress ($4\phi\sqrt{f'c}$) of the concrete in the slab adjacent to the core, but nominal shear steel was added in case cracking of the slab became extensive. Two layers of intermediate steel were placed in both horizontal directions to control shrinkage cracking. Final maximum soil pressures due to load factored loading were 4.7ksf long-term and 6.8ksf transient.

7.0 SHEAR CORE

7.1 General. The shear core consists of 6 coupled channel shaped segments. Flanges on the segments were used for a number of reasons. They:

i) Improve the ductility of the segments.

ii) Increase the moment of inertia and ease reinforcement placing.

iii) Allow coupling spandrels to be made wider than segment webs thus lowering shear stresses.

iv) Are able to resist those components of core radial frame action bending, resulting from the curved plan shape, which are not located in the plane of the floor diaphragms.

v) Are able to resist, but also generate some, secondary torsions due to non-alignment (in plan view) of the torsion centres of segments and coupling beams.

vi) Serve both as primary structure, and as support for stairs and slabs, service ducts and fire compartments.

The reserve capacity of a coupled shear wall structure can be improved at relatively little expense by considering the probable failure mechanism and adding reinforcement where necessary. For segment design a reduced beam stiffness was considered as this ensures that segments will be able to function when beams deteriorate. Although for a given applied load a significant increase in segment moments and shears takes place the reduction in response practically offsets this increase.[6] This does not necessarily mean that beam sizes should have been reduced in the first instance as stiffer members assist in damage control and increase comfort to occupants in small earthquakes.

In designing the coupling beams, unlike the segments, an initial assumption of low beam stiffness is not beneficial. Low assumed stiffness results in low beam design moments and consequently low ultimate capacities. Energy dissipation of a coupled shear wall on a significant scale does not take place until the column segments yield.[7] When beam capacities are low the system operates more as individual cantilevers resulting in much larger deflections for a given external load than when effective coupling is maintained. This in turn would produce early damage in the coupling beams should their stiffness be significantly greater than assumed.

The ability of the system to provide individual cantilever performance following failure of coupling beams while a desirable redundancy feature should be delayed as long as possible. The combined effect of lengthening structure period and increase in damping would halve the response but a significant increase in segment ductility demand would still result from long earthquake motions. Against this, removal of most axial compressive loads (induced by the coupling beams) improves ductility and removal of induced axial tensions improves segmental shear strength.

7.2 Shear Core Analysis. Preliminary sizing was based on an adaption of a method by Burns.[8] The initial analysis of the core was by an adapted plane frame Calfram computer programme. Late in the design a space frame analysis using a STRUDL programme became available which allowed the two centres of gravity of the column segments to be maintained with minimum alterations to the spandrel beam properties. A check was made to ensure that there was no significant distortion from the circular shape at each floor level.

The stiffness of members in a coupled shear wall system is subject to constant change with varying direction and intensity of earthquake loading. Members change from homogenous elastic, to cracked and finally to the inelastic state. Axial forces vary in magnitude and from tension to compression in combination with varying intensities of flexural and shear stresses. Bi-axial loading complicates the conditions. As with all practical design the assumptions that are made must allow for uncertainties. These were principally the degree of softening of coupling beams, and reduced stiffness and area of segments subject to high axial tensions. To obtain a reasonable degree of correlation with the model test referred to below the initial analysis made allowance only for column shortening, shear distortions in deep beams and distortions at member junctions by extending coupling beams to form "Muto beams".[9] (Clear span plus $\frac{1}{4}$ beam depth at each end). For column segments full floor to floor height was adopted.

The model test was carried out by Ministry of Works Central Laboratory on a 1/30 full size cast epoxy resin model.[11] This work was particularly important in view of the adaptions required to the early computer programmes. In fact the model tests, did draw attention to the presence of hoop stresses different from those in an unperforated tube and these were later confirmed more accurately by the "STRUDL" programme. Only one segment was fully strain gauged, with checking gauges on others. Approximately 380 gauges were used, but the symmetry of the model allowed the loading to be applied in some 12 directions to give the maximum number of readings and check readings. For practical purposes model and computer results showed very good correlation bearing in mind of course the common assumptions made. (Fig. 4). Additional instrumentation of the model would have been required to get full correlation of hoop stresses at the lower levels, particularly in the solid basement walls. Hoop stresses are significantly affected by the redistribution required between segments subjected to bi-axial bending but prevented from deflecting out of line at various levels by coupling beams and diaphragm. Our practice of taking all tensile components of shear by reinforcement (particularly in potential hinge positions) meant that adequate steel was present to take hoop stresses because maximum shear and hoop stresses do not occur concurrently. As mentioned earlier, computer runs were also made with reduced stiffnesses in the coupling beams between the core segments because they are subject to the highest rotational ductility demand, minor cracking tends to rapidly alter their stiffness, they tend to be the least ductile members because of their proportioning, earthquake damage records indicate earliest deterioration in these members and because of inadequate large scale test information at the time on their behaviour. As expected, with reducing stiffness a marked increase in the column segment moment occurs, however, as previously mentioned this is balanced by the effect of increasing period and damping.[6] The pessimistic assumption of spandrel beam deterioration resulted in some additional reinforcement being placed in the segments but provided a valuable second line of defence.

7.3 Core Detailing. (Fig. 2, 3). Was based on the principle that members should not fail in shear or concrete compression. Flexural capacities were therefore assessed for zero under-capacity and probable maximum reinforcement yield stress, while normal under-capacity factors were used for shear. In accordance with test information available at the time[15]

$10\phi\sqrt{f'c}$ was set as the maximum for members where shear is continuously introduced at the boundaries. This figure was reached only in the highest stressed coupling beam (even though its width had been increased to 3') but lower shear stresses occurred in others, particularly those at higher levels where ductility demand is greatest. Shear stresses in segments were significantly lower, a condition favourable to ductility. The concrete was not considered to contribute to shear strength because of the possibility that it would deteriorate under seismic loads. To slow the possible deterioration of compression zones, to prevent buckling of reinforcement under repeated loadings in cracked compression areas and to resist radial components, ties were used. (Fig. 3).

Reinforcement of the segments was designed using specially prepared inter-action diagrams[6] using principles similar to those applied to conventional columns. Bi-axial bending necessitated preparation of envelope diagrams, under-capacity factors varying from 0.9 to 0.7. Flexural reinforcement had to be provided to meet the requirements of a number of load cases and directions and the final capacity in some directions always exceeded the analysis moment. This in turn required an increase in shear reinforcement to match section flexural capacity. The analysis shear was therefore increased by the ratio of capacity to analysis moment. To determine section capacity the intersection of the function of axial load versus moment for increasing earthquake effect was determined from the interaction diagrams.[6] This had to be done for a number of load cases. Flexural over-capacity averaged 50% in the lower stories increasing towards the top of the building. Segmental hinges must therefore form in the lower stories and in these regions shear reinforcement was provided for all tensile components of shear at moment capacity.

8.0 FLOOR AND RING BEAMS

The majority of floor slabs were precast with in-situ junctions over supporting beams and between themselves. To ease the usual problem of placing slab and beam reinforcement the top slab steel was formed into flat loops which alternate as shown in fig. 2. To confirm the envisaged transfer of stresses by a horizontal truss system full scale tests[12] were carried out, and these proved that provided the joint was wider than 10", the yield stress in the reinforcement could be developed. The 4th floor ring beam is prestressed against the horizontal component of the axial force from raking columns above, and is reinforced for all other loads. Total horizontal force is about 700k and prestress derived from three cables about 725k total capacity. The beam will be stressed after the 4th floor but before any raking columns are erected.

9.0 CONCLUSIONS

The authors believe that they have achieved an efficient earthquake resisting system that should sustain little damage in a severe earthquake and which has ample redundancy against collapse. As always actual earthquake performance must be used as a reference point for design and examination of recent earthquake damage directly by the authors or as reported indicated that in the vast majority of cases shear wall earthquake performance even without special ductility consideration had been surprisingly good. In the few instances where this was not the case it had been due to major design, detailing or construction shortcomings. Member sizes and details reproduced here make an interesting comparison with those reported from damaged shear wall buildings in Alaska 1964 earthquake.[14]

10.0 ACKNOWLEDGEMENTS

The senior author was responsible for the structural concept, the design philosophy and standards employed. Ballintine headed the design team with Williams and Chapman responsible for detailed analysis and design. The authors wish to thank the Commissioner of Works for permission to publish this paper, Dr N. Priestley for the model analysis and the many others involved in the successful completion of this project.

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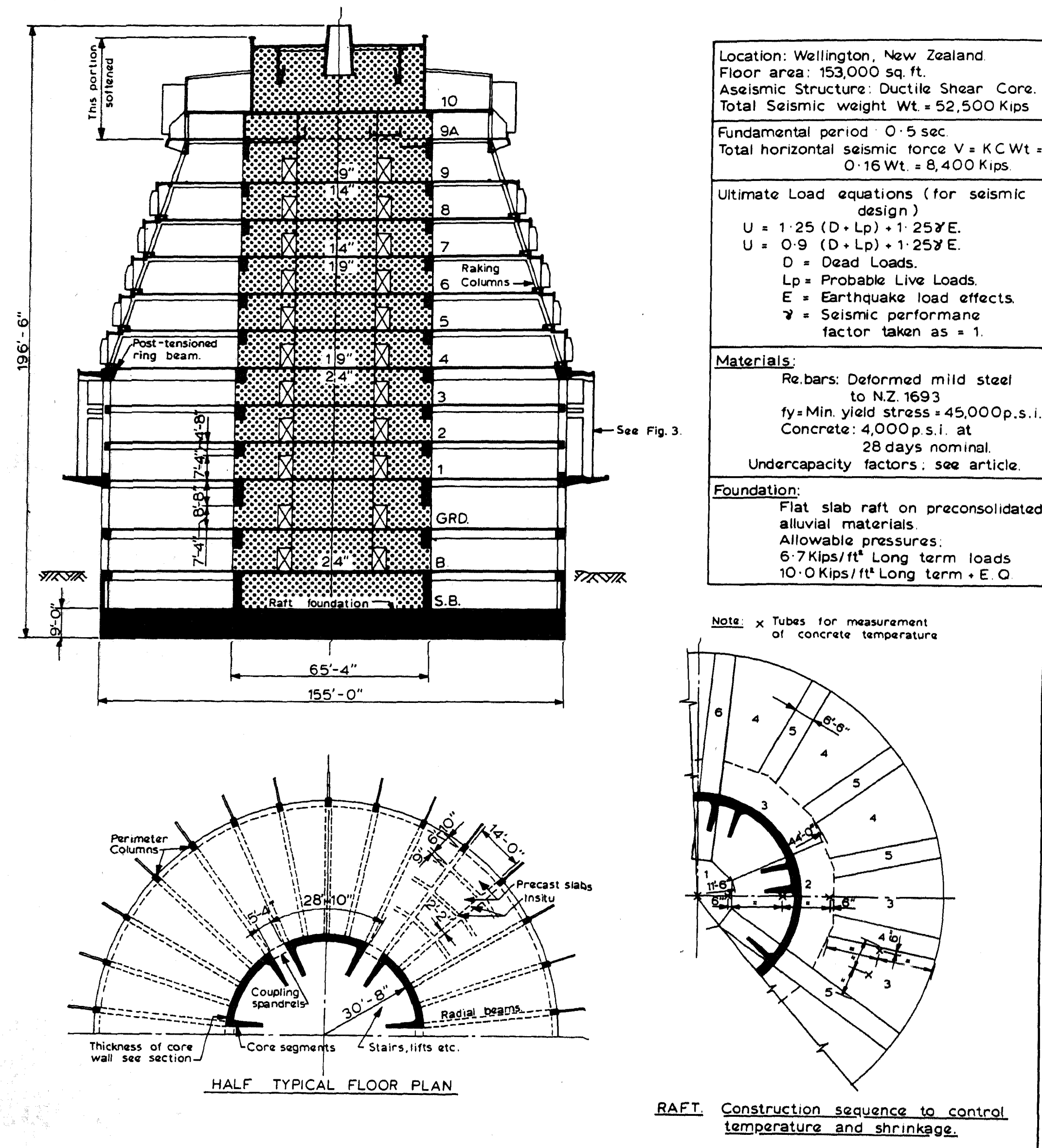
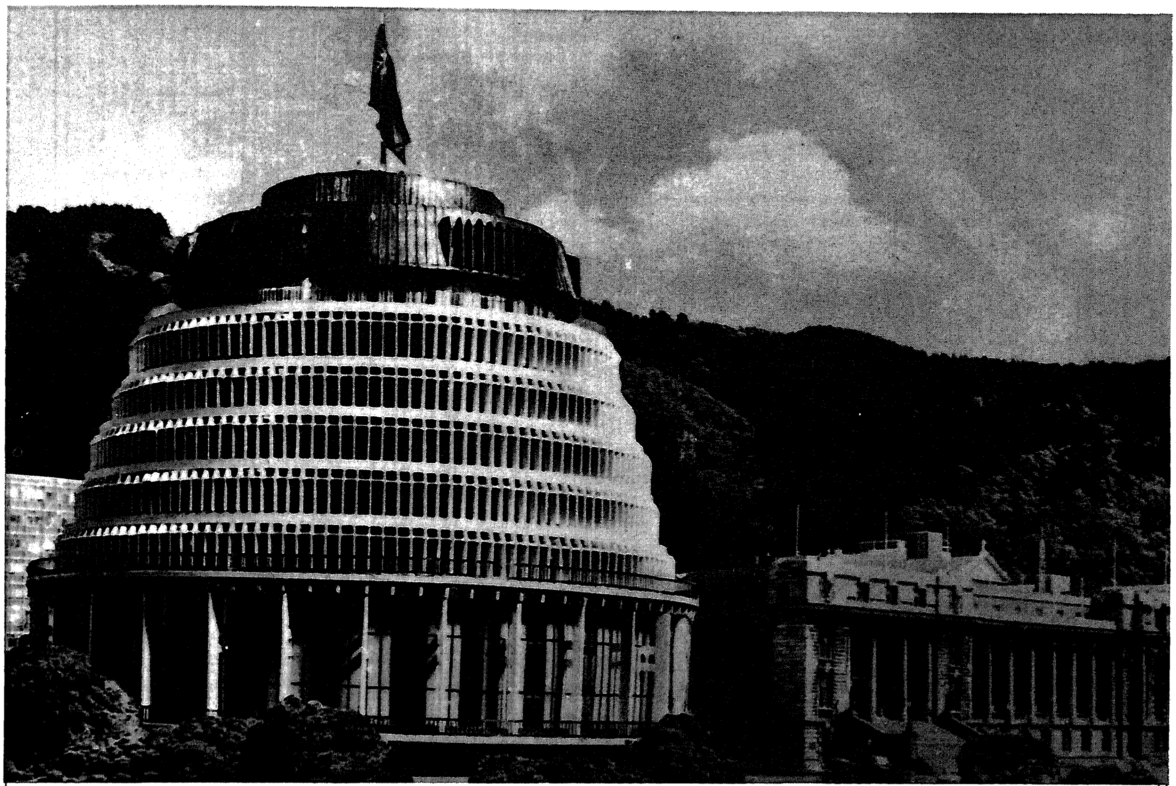
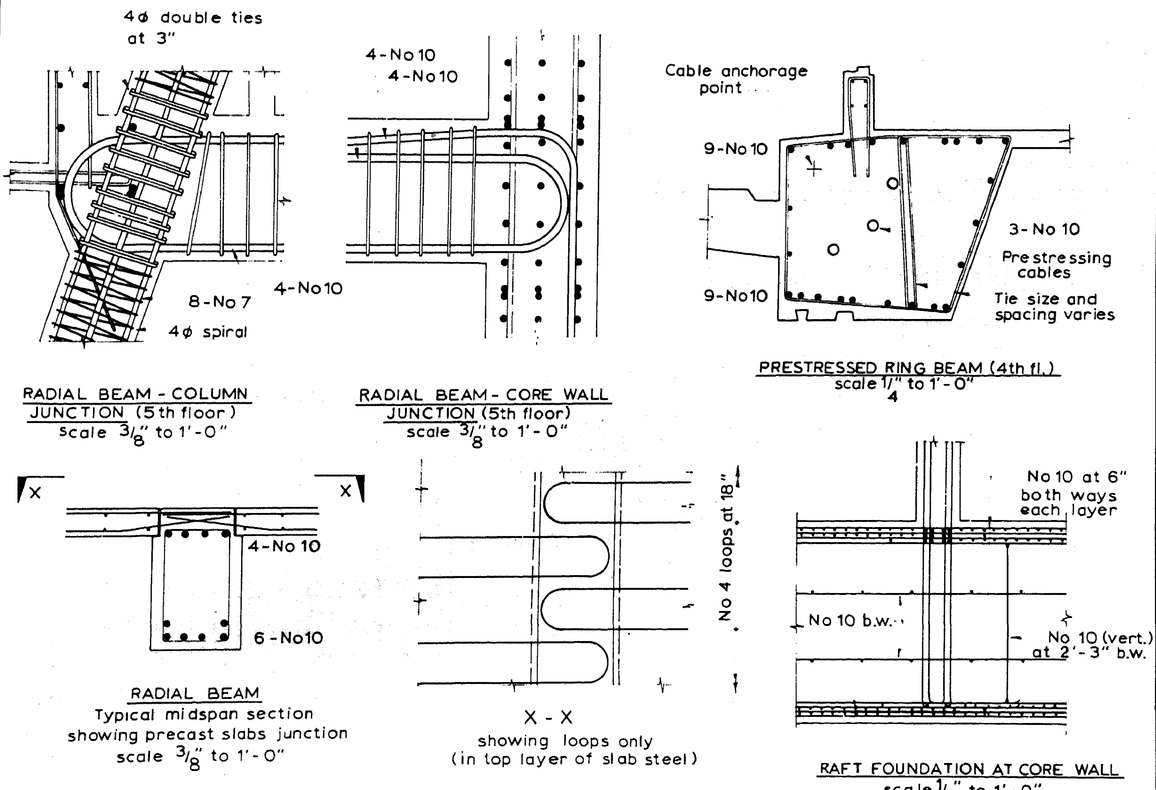


FIG. 1. ADDITIONS TO PARLIAMENT BUILDINGS "BEEHIVE" TOWER BLOCK





RADIAL BEAM - COLUMN JUNCTION (5th floor)
Scale 3/8" to 1'-0"

RADIAL BEAM - CORE WALL JUNCTION (5th floor)
scale 3/8" to 1'-0"

PRESTRESSED RING BEAM (4th fl.)
scale 1/4" to 1'-0"

RADIAL BEAM
Typical midspan section showing precast slabs junction
scale 3/8" to 1'-0"

X - X
showing loops only
(in top layer of slab steel)

RAFT FOUNDATION AT CORE WALL
scale 1/8" to 1'-0"

FIG. 2

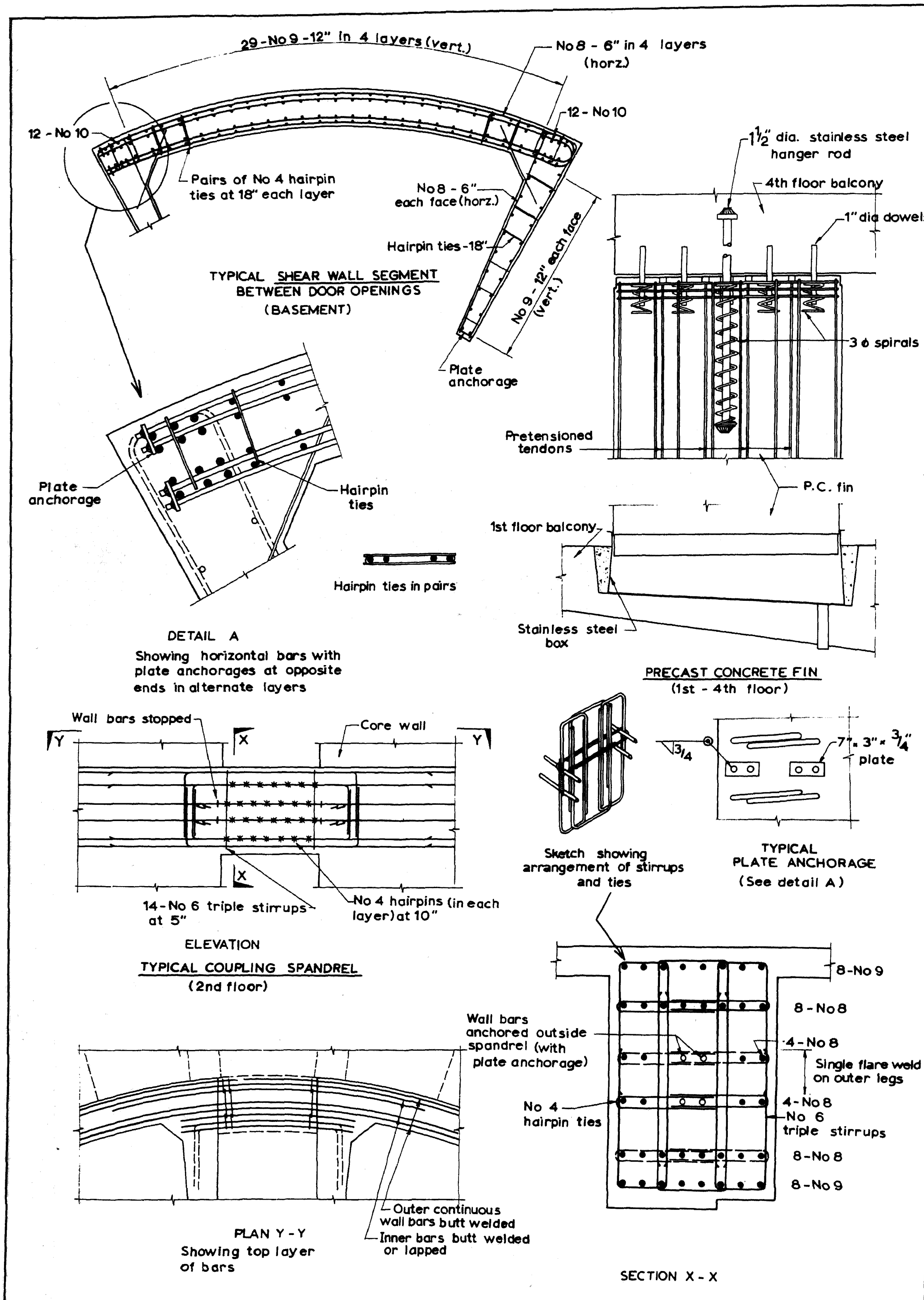
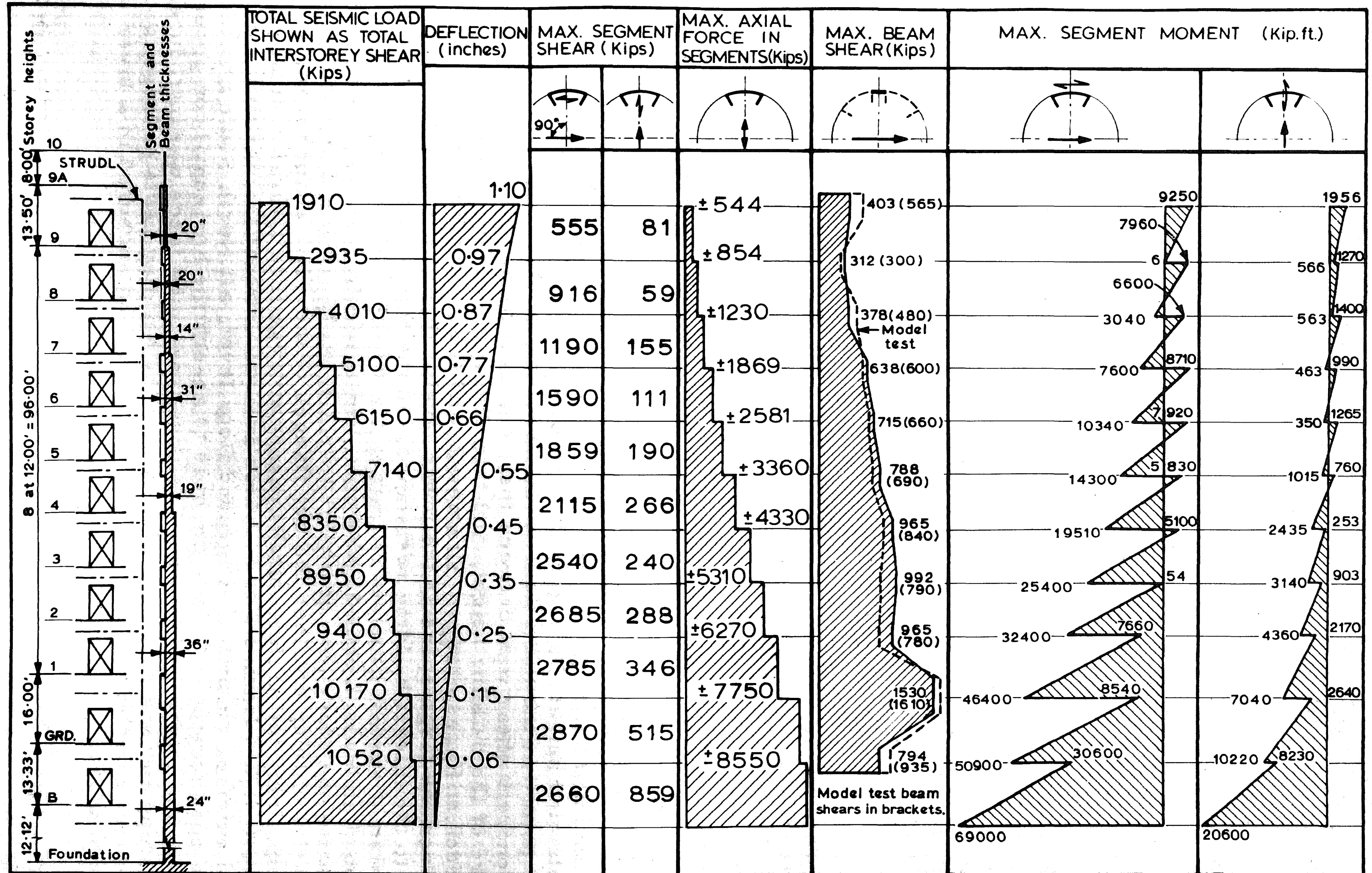

29-No 9 -12" in 4 layers (vert.)
No 8 - 6" in 4 layers (horz.)
12 - No 10
12 - No 10
Pairs of No 4 hairpin ties at 18" each layer
No 8 - 6" each face (horz.)
Hairpin ties -18"
No 9 - 12" each face (vert.)
Plate anchorage
TYPICAL SHEAR WALL SEGMENT BETWEEN DOOR OPENINGS (BASEMENT)
Plate anchorage
Hairpin ties
Hairpin ties in pairs
DETAIL A
Showing horizontal bars with plate anchorages at opposite ends in alternate layers
$1\frac{1}{2}$" dia. stainless steel hanger rod
4th floor balcony
1" dia dowels
3 ϕ spirals
Pretensioned tendons
P.C. fin
1st floor balcony
Stainless steel box
PRECAST CONCRETE FIN (1st - 4th floor)
Wall bars stopped
Core wall
Y
X
Y
X
14-No 6 triple stirrups at 5"
No 4 hairpins (in each layer) at 10"
ELEVATION
TYPICAL COUPLING SPANDREL (2nd floor)
3/4
7" x 3" x 3/4" plate
Sketch showing arrangement of stirrups and ties
TYPICAL PLATE ANCHORAGE (See detail A)
8-No 9
8-No 8
Wall bars anchored outside spandrel (with plate anchorage)
4-No 8
Single flare weld on outer legs
No 4 hairpin ties
4-No 8
No 6 triple stirrups
8-No 8
8-No 9
Outer continuous wall bars butt welded
Inner bars butt welded or lapped
PLAN Y - Y
Showing top layer of bars
SECTION X - X

FIG. 3



NOTES: Results include 1·25 Load factor. Values are maxima and do not necessarily occur concurrently.

Fig. 4. SHEAR CORE ANALYSIS