THE DEVELOPMENT OF THE DESIGN OF THE ANZ HEAD OFFICE BUILDING, LAMBTON QUAY, WELLINGTON

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ABSTRACT

The irregular shape of the site on which this multi-storey building is to be located has given rise to an interesting architectural solution which in turn has led to more complex than normal methods of analysis and detailed design being required. This paper describes the development of the structural solution and the procedures set up to handle the design. Special mention is made of some of the difficulties encountered in following codified analysis methods.

1.0 INTRODUCTION

The building, which is to be the new head office of the ANZ Banking Group Ltd., in Wellington, is a reinforced concrete structure of twenty levels comprised of four levels of podium structure (including two basement levels below ground level) and sixteen levels of tower block structure. The site is bounded by Lambton Quay, Featherston Street and Grey Street (Figure 1). Both the tower block and podium structure, therefore, are basically triangular in plan. This shape dictated not only the basic structure form, but also the analyses and design procedures needed.

2.0 STRUCTURE

2.1 Selection of Tower Floor Plan

The basic tower floor shape developed by the Architects and principal consultants, Peddle Thorp & Maidens, is a tidy solution to the restrictions placed on them by the triangular site. A number of alternative structural forms were investigated to suit this shape. Beams that might be necessary either to develop frame action or to support the floor slab were subject to a constraint on their depths. This arose from both the architectural need to seek the minimum floor to floor height so that the maximum number of storeys could be provided within the restricted building height and the requirements of air conditioning ducting.

A conventional fully framed system was studied along with a scheme using shear walls in the re-entrant corners of the floor plan. Both of these systems left the central area free of columns but the one using shear walls was soon found to be unacceptable because of the consequent loss of window area.

The scheme finally adopted arose from considering a perimeter frame with a small number of internal columns. At the request of the client, these internal columns were condensed to one large central column with radial haunched beams (Figure 2). The client was prepared to meet the additional expense this entailed because it could be offset by the more easily subdivided floor plan it gave for letting purposes.

The use of prestressing in the flat slabs was investigated as a means of reducing the slab thickness but this was deleted both for economic reasons and because of the difficulty of anchoring cables in the vincinity of the kite-shaped columns.

2.2 Tower Framing

The building comprises four levels of podium structure with sixteen tower floors as shown in Figure 3. The podium and tower block structures together form an integral structure with the tower block being basically a moment resisting frame structure and the podium block a moment resisting frame and shear wall structure. The principal decision in development of the tower framing was whether the tower structure should be separated from the podium structure. The advantages of this separation are:

(a) A simplification of the frame for analysis.
(b) The probability of a more satisfactory seismic performance through provision of a symmetrical tower.
(c) No high floor diaphragm shears where the tower joins the podium.
(d) A longer fundamental period of vibration.

The principle disadvantage of separation is the finishing of the gaps in the floor slabs.

It was decided that because this was to be a prestige building separation gaps should be avoided and the tower and podium framework were combined. The resulting complexities in the analysis are discussed later.

3.0 SELECTION OF DESIGN STANDARDS

3.1 Introduction

Because of the ever changing nature of the latest thinking on the finer details of design, a procedure is followed in which current practice is reviewed, design standards selected, client approval sought and then the procedure is frozen. In this way it is possible to maintain a consistent design

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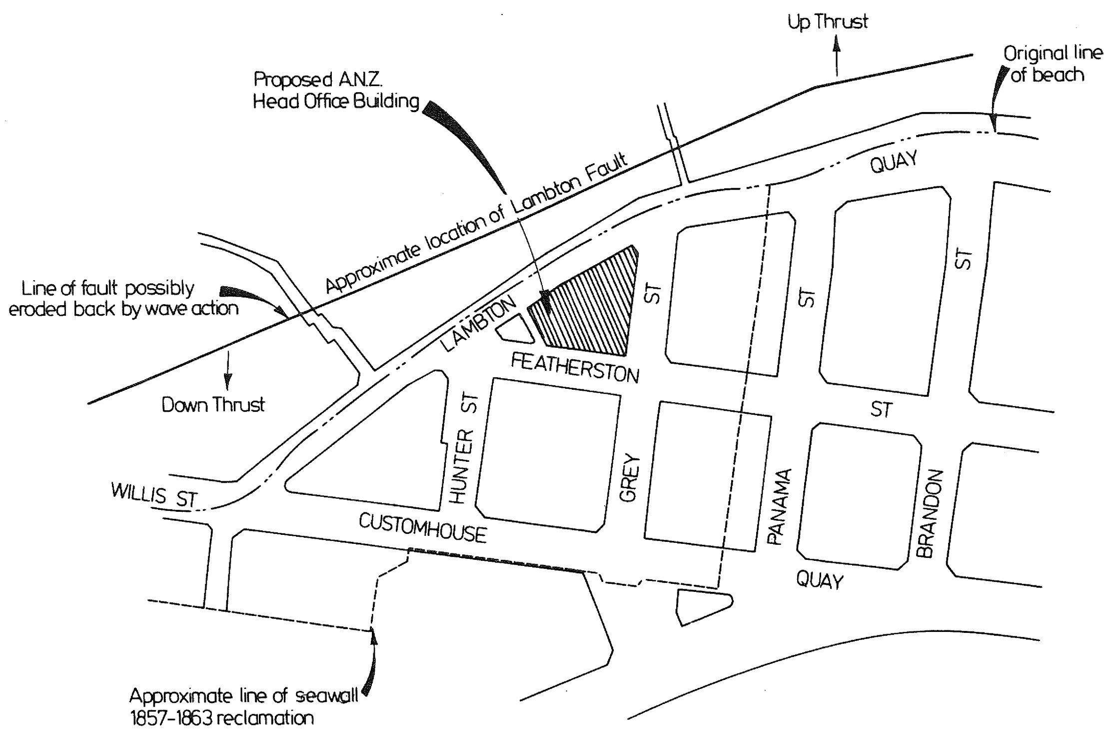

Figure 1 : SITE PLAN

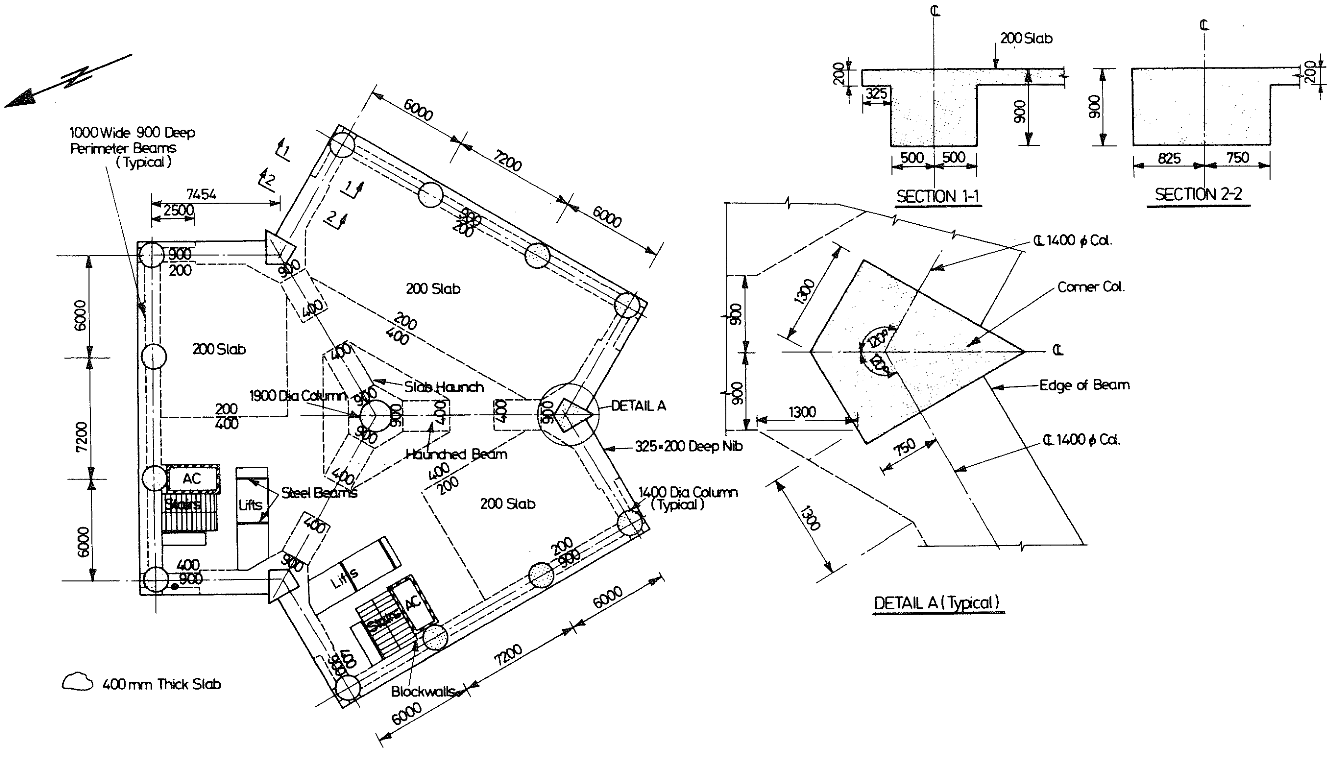

Figure 2 : TYPICAL FLOOR PLAN
ANZ BANKING GROUP LTD.
LAMBTON QUAY

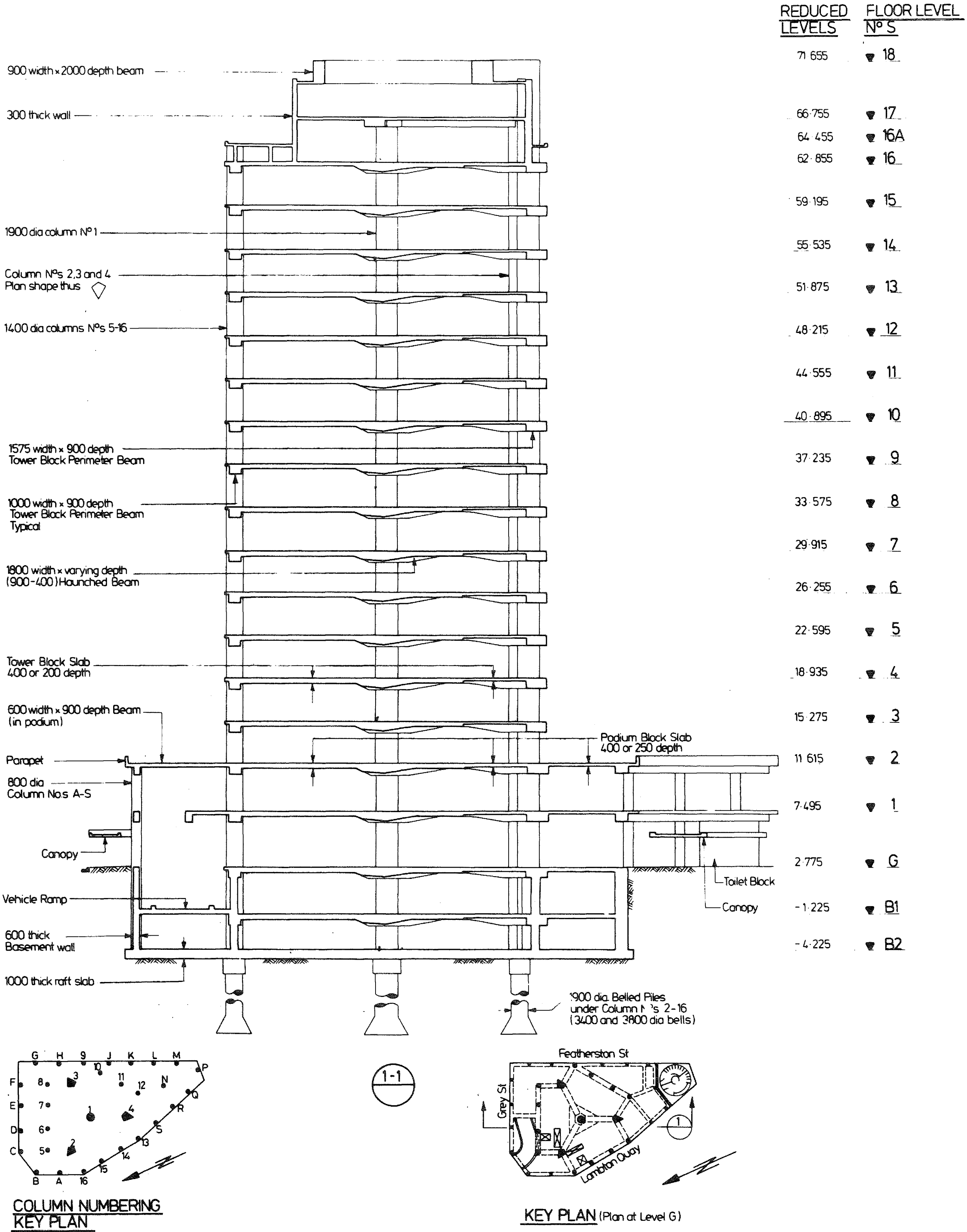

Figure 3 : SECTIONAL ELEVATION NORTH-SOUTH

specification available to all the members of the design team over the period of the design phase. If, for whatever reason, design standards changes need to be considered during the course of the job, they may then be submitted to the Client for cost/benefit evaluation with respect to the original specification.

If it is wondered what sort of design standard indecision may lead to unexpected cost later in the design then one only needs to consider the lateral load considerations for passenger lifts. When this matter was being considered three pertinent document were available. These were the Power Lift Rules of the Marine Division of the N.Z. Ministry of Transport, NZS 4203:1976 and a draft internal specification of the N.Z. Ministry of Works & Development. Because of the possibility that the structure, at some future stage, would come under the N.Z. M.W.D.'s scrutiny in approving it as suitable for a government tenancy, the more stringent of the three documents, that of the N.Z. M.W.D., was adopted.

A fair proportion of the design specification is made up of comments on standard office practice and includes information on the allocation of responsibilities.

3.2 Design Codes

Current New Zealand Standards were offered for client acceptance for most aspects - including loading, steelwork and concrete blockwork design.

The N.Z. Code of Practice for General Structural Design and Design Loadings for Buildings (NZS 4203:1976)(1) was followed even though the Wellington City Corporation had not at that stage formally adopted it. For reinforced concrete design and detailing ACI 318-71(2) was offered instead of the N.Z. equivalent. Before submission to the client, discussion of the proposal took place with the City Corporation and their written approval was obtained.

3.3 Detailed Design Standards

The mere adoption of current loading codes and design specification does not supply sufficient guidelines to the design team for the production of satisfactory details. A number of supplementary design methods and standards were therefore also adopted. This process was simplified where possible by endeavouring to adopt only one definitive paper for the aspect under review. This tends to overcome any incompatibilities that may arise through the mixing of sections from different researchers.

An important section of the specification was setting down of guidelines for the capacity design of the frame of the structure. This was broken down into sections on:

(a) Frame Analysis
(b) Moment Re-distribution
(c) Beam flexural and shear reinforcement design
(d) Beam overstrength moment calculations
(e) Column design axial loads and moments
(f) Column moment and shear reinforcement design
(g) Beam-column joint-shear design
(h) Pile and foundation design.

These guidelines were prepared by the B.C.H. & F.'s Standards & Research Group (Wellington) and were formulated on the basis of past experience, knowledge of the firm's computing facilities and an appraisal of current papers.

4.0 THE ANALYSIS

The purpose of the analysis was to :

(a) Provide an equivalent lateral static load distribution as required by the N.Z. Code NZS 4203:1976(1)
(b) Provide the resultant member reactions due to dead, live and earthquake loads.

4.1 The Computer Model and Physical Problems

Program EASE2, available on the CDC CYBERNET system in Melbourne, was used to set up the three-dimensional model of the structure. The triangular shape of the structure, and the ability of the program to handle both the tower and podium interactively together, were the deciding factors in the decision to carry out a full three-dimensional analysis of the structure. If the structure had been rectangular in plan with regular framing system parallel to the sides, the two-dimensional computer programs developed within the firm would have been used to deal with each principal direction independently.

Finite joint size and axial deformations were modelled. Shear walls were modelled as cross-braced panels and floor diaphragms were represented by beams with appropriate properties to simulate the slabs they replaced.

Foundations (both perimeter walls of the podium and piles) were modelled using equivalent springs in both the vertical and the horizontal directions.

The values of stiffness for these springs were calculated from the soil properties obtained from site investigations carried out by sinking test boreholes. These values were averaged to take into account the fact that they had to represent both increasing and decreasing load in an elastic fashion. A displacement limit was placed on their use and the subsequent analyses showed that this limit was not violated.

One disadvantage of the particular computer program used was the need to specify separately the rigid zones at the ends of every member entering a joint. Only in this way could the effect of stiffness of the joint size be modelled.

For the dead and live load analyses the column areas were increased by a factor of 100 to eliminate differential displacements of beam ends under these long term loadings.

The representation of the structure required approximately 2000 nodes (or joints) and 1410 members. Approximately 4000 computer cards were required to describe the problem.

4.2 Floor Slab

A preliminary estimate of the floor

thickness was made using the yield line theory and setting up equations which could be easily solved by hand. A more sophisticated analysis was then carried out using finite element techniques for both the dead load and live load case.

Symmetry allowed half the slab to be analysed and use was made of the shell finite element available in the EASE2 program. Both the perimeter beams and the radial (haunched) beams supporting the slab were modelled as an integral part of the slab. The finite element grillage was selected, from the beginning, to be finely divided so that the need for additional analyses to determine sensitivity would be unlikely to arise. (Figure 4)

4.3 Lateral Load Analysis

NZS 4203:1976 [(1)] allows the base shear coefficient for the seismic static lateral loading to be factored by 0.9 if a dynamic modal analysis is carried out. Clause 3.4.6.1(d) requires any major buildings with unusual structural features to have their seismic lateral loading distribution determined by dynamic analysis.

Modal analysis was used to determine this distribution in two principal directions. A base shear coefficient of 0.054 was derived for the structure which had first nature periods of 1.4 and 1.2 seconds in the two directions. It was of some concern to find that the lateral load distribution calculated by this technique was subject to alteration because of the provisions of Clause 3.5.2.5 which restrict the designer to an inter-storey shear of not less than 80% of that value computed by the equivalent static forces method. The irony of it is that use of the equivalent static forces method is expressly prohibited by the same code for determining the seismic lateral load distribution of a structure such as this because it is considered not a suitable method for an irregular structure. Consultations with the DSIR and MWD then followed. It is understood that this particular clause was included at a time when the majority of high rise development did not exceed six of ten storeys. In structures of this height the effect of nature modes other than the first are not as predominant as in more slender buildings. However, the advice from the DSIR was that such a provision is still necessary to guard against an unconservative design level of inter-storey shear being used when there is a possibility that the post-elastic behaviour of the structure may exhibit more first mode behaviour than that of the elastic behaviour. Figures 5 and 6 show the seismic lateral load and interstorey shear distributions for one direction.

The designers of the structure were concerned that the strengthening of the middle-height region of the structure might lead to a structure ill-proportioned to develop plastic hinges in the most favourable manner.

For major office projects such as this one it is now also necessary to satisfy the NZ MWD that the structure has been designed to a standard (and by a method) approved by them for this will be part of the criteria used by them if some government department seeks their approval to rent space some time in the future.

The lateral load distribution finally used for the structure in question was produced by a "by eye" modification to the values calculated by the modal analysis in such a way that the 80% stipulation was no longer violated in the mid-height region.

The need for this type of approach with such manual intervention precludes the possibility of a one-shot automatic dynamic analysis that is available in some of the package programs.

4.4 Torsion

The analysis procedure makes it difficult to locate specifically the centre of stiffness of each storey or the torsional response of the frame in any meaningful way. The code requirements were interpreted as requiring the seismic lateral load to be applied at eccentricities of one-tenth of the overall width of the frame with respect of the centre of gravity of each floor.

5.0 DESIGN

5.1 Introduction

It is the aim of this section to explain how the design of the building was produced to the requirements of the design specification (discussed in Section 3) and how difficult aspects were resolved.

5.2 Preliminary Analysis

A full nine months before the final design was attempted work was carried out on the proposed structure of that time to assist with member sizing and floor system selection. At this early stage an effort was made to ensure that the resulting structure would be easy to build. Generous member sizes were fought for at this point and large-scale preliminary drawings of beam column reinforcement intersections were drawn to check that the expected percentages of steel would not cause major construction difficulties. It is interesting to note that the most economical structure solution was not considered to be either the best overall economic or architectural solution.

Although a limited three dimensional computer analysis was carried out at this point, an extremely simple hand calculation proved to be quite accurate and valuable. Assuming all the beam hinges (of identical capacity) to have formed at one particular level, the sum of the resolved plastic moment components in one directions was equated by virtual work principles with an estimation of the ultimate inter-storey shears acting at that level. Hence a typical section moment capacity could be obtained.

5.3 Moment Redistribution

With the final results of the three-dimensional computer analysis now available it was obvious that corner or end columns of the bents on each side of the triangular frame were attracting the expected large moments from the beams. To enable the realistic design of these columns, moment redistribution was carried out in accordance with Paulay's recent paper[3]. This involves a re-shuffling of the proportion of the inter-storey shear carried by a set of columns amongst themselves and matches, to some extent, what actually happens as the structure reaches that level of lateral loading at which some beam sections have begun to yield. The re-distribution procedures is best summed up by the two rules formulated by Paulay:

1. In any span of a beam the change of moment due to the redistribution process should not exceed at any point 30% of the absolute maximum moment derived for that span from elastic analyses for <u>any</u> combination of earthquake and factored gravity load.
2. Moment redistribution between columns should not change the maximum value of the combined end moments in any column, derived from elastic analyses for any of the load combinations referred to in 1. by more than ±15%. This limitation is satisfied if the redistribution of shear forces between columns is limited to ±15% of the shear force acting on the column in question.

These rules were applied so as to reduce the column-face beam moment at the outer columns at the expense of sections at internal columns. Practical considerations force a slight departure from the described method. As there were seventeen different load cases to be considered this procedure was carried out on two composite bending moment diagrams made up from the maximum positive and negative moments occurring at each end of the beams.

5.4 Beam Design

In the tower part of the structure the re-distributed beam moments were rationalised between similar bents on each of the three sides of the structure so that identical steel was specified for each. Where possible this rationalisation was also extended vertically to adjacent levels but this was only possible in a small number of places because of the designer's desire to proportion the capacity of the beams throughout the height of the frame in a manner consistent with the results of the modal analysis. Subsequent checks have shown that a favourable order of beam yielding should occur in that, at any one level, the perimeter beams will yield before the re-entrant beams. The haunched beams in the slab to the central column have attracted surprisingly little earthquake moment.

Detailing of beams was in accordance with a NZ MWD paper by R. L. Williams[4]. Minimum steel prevailed in only the top two levels and every effort was made to assist in the easy assembly of reinforcing cages with staggered butt laps of adjacent bundled bars to give equivalent staggered laps where possible. Although the calculated points of cut-off for redundant bars tended to occur within the plastic hinge zones, sensible fabrication and minimum steel requirements dictated that they were placed further towards the middle of the spans.

5.5 Column Design

Design axial loads were calculated by the method described by Paulay[6] which apportions the number of levels above that being investigated considered to be contributing beam overstrength moments to the column.

The elastic bending moments for columns were such that no point of contraflexure appears in several lower storeys levels. However, the moments were small and the minimum steel requirements of 1% covered all the load cases. The minimum steel requirements of 1 percent governed for all the columns except for the corner columns (Column Nos. 5, 8, 9, 12, 13 & 16 Figure 3) where biaxial beam hinging was considered. In this case the maximum steel percentage was 2.1% at lower storey levels (due to high axial tension) and reduced to a minimum of 1% at upper storey levels. Minimum steel criterion was also the governing case for the kite-shaped columns since there were no axial tension effects due to three beams hinging simultaneously. While originally designed as circular columns of equal area the kite-shaped columns were checked against axial load/ultimate moment charts produced by computer.

It was considered that the dynamic modal analysis had already provided any dynamic magnification to column moments as proposed by Paulay[6] so the "Xmas tree" effect did not need to be considered further.

Confinement for base-hinging was provided at points of fixity for the individual columns. The section ductility available for the centre column at basement level was checked from the original MWD Bridge Pier Ductility charts as being 5 - considered satisfactory as these charts are known to be conservative.

5.6 Beam-Column Joint Design

ACI paper No. 73-28[5] was used as a guide to the design of the joints. The interior columns were under axial compression under critical load conditions and a part of the joint shear was therefore assumed to be carried by the concrete. As a result, it was found that the transverse reinforcement requirements in the beam-column joints for these interior columns at lower storeys were governed by confinement rather than joint shear.

The beam-column joints for the corner columns were analysed by a truss model assuming a potential failure plane from corner to corner and considering bi-axial hinging of the beams. The horizontal shear was assumed to be carried entirely by transverse spirals since the column is subjected to axial tension under the critical load condition[5], and the vertical joint shear is assumed to be carried by the longitudinal column bars.

Joint geometry allowed the transverse reinforcement to be a maximum of the order of double R20 at 90mm pitch.

5.7 Pile Design

Piles were put under each column, penetrating down to weathered greywacke at a depth of about 5m below ground level. The piles under corner columns were designed to penetrate deeper as these columns were subjected to axial tension under critical load conditions. Nominal reinforcement was provided and the pile section capacity was checked to ensure that the plastic hinge formed at the column base under design earthquake loads.

6.0 CONCLUSION

This paper has described the development of the design of a multi-storey reinforced concrete building of unusual shape. The way in which the proposed structural systems have been modified to meet economic and architectural requirements has been outlined.

Emphasis has been placed on the advantages of establishing a design specification with the client right at the beginning of the design process in order to avoid later decisions leading to unexpected cost increases.

The dynamic analysis that was considered to be required because of the unusual form of the structure is described and the difficulties encountered in following the requirements of NZS 4203:1976 concerning the lateral load distribution are explained in detail.

It is shown that there is need for further research to be carried out into the codified procedures of spectral modal analysis. A rationalisation of the various rules of seismic provisions for passenger lifts is also needed.

7.0 REFERENCES

1. NZS 4203 "The NZ Code of Practice for General Structural Design and Design Loadings for Buildings".
2. ACI Standard 318-71 "Building Code Requirements for Reforced Concrete".
3. Paulay T., "Moment Redistribution in Continuous Beams of Earthquake Resistant Multi-Storey Reinforced Concrete Frames", Bulletin of the N.Z. National Society for Earthquake Engineering, Vol. 9, No. 4 December 1976.
4. Williams, R. L., "A Procedure for the Analysis and Design of Ductile Reinforced Concrete Frame Buildings of Moderate Height".
5. "Recommendations for Design of Beam Column Joints in Monolithic Reinforced Concrete Structures", Journal of the ACI, July, 1976, paper 73-28.
6. Paulay, T., "Columns - Evaluation of Actions", Bulletin of the N.Z. National Society for Earthquake Engineering, Vol. 10, No. 2, June, 1977.

This paper was presented at the South Pacific Regional Conference on Earthquake Engineering held in Wellington on 8, 9 and 10 May, 1979.

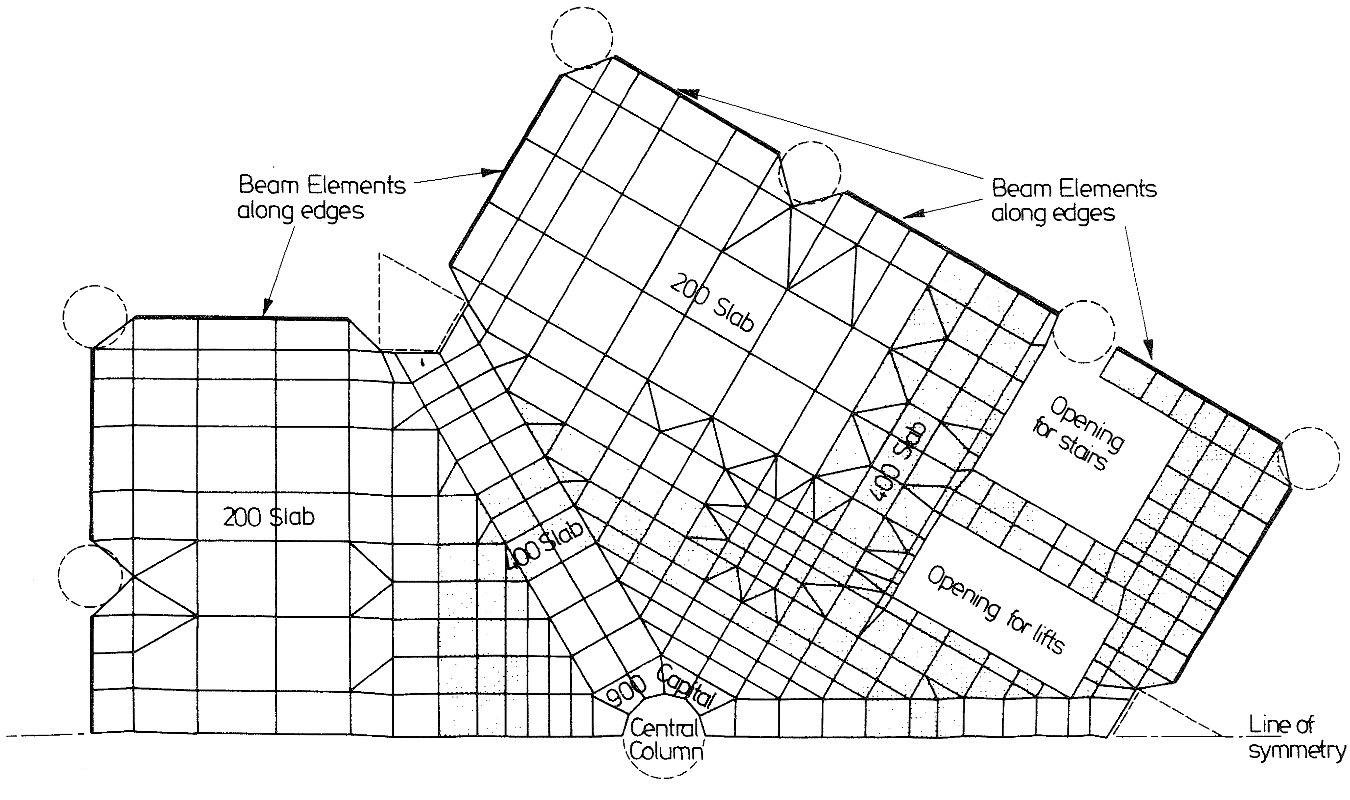

Figure 4: FINITE ELEMENT MESH OF TYPICAL TOWER SLAB

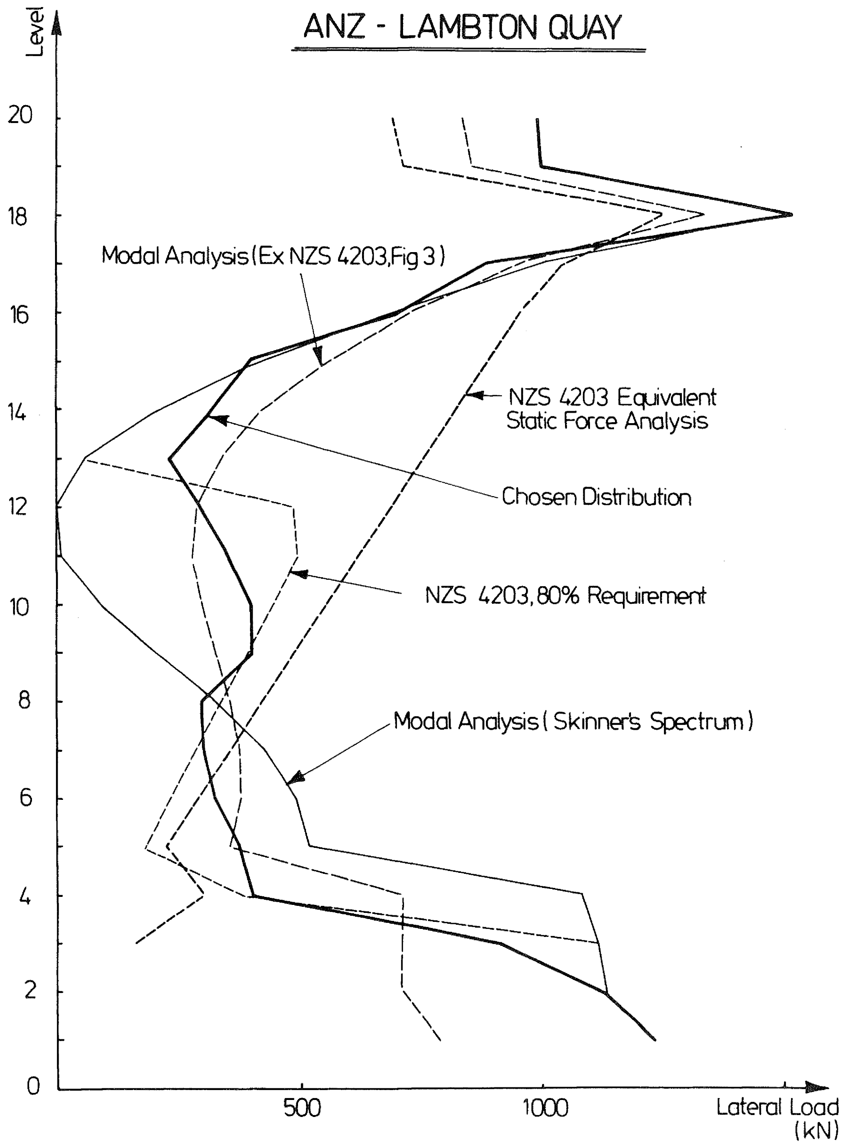

Figure 5: SEISMIC LATERAL LOAD DISTRIBUTION

Parallel to Featherston Street

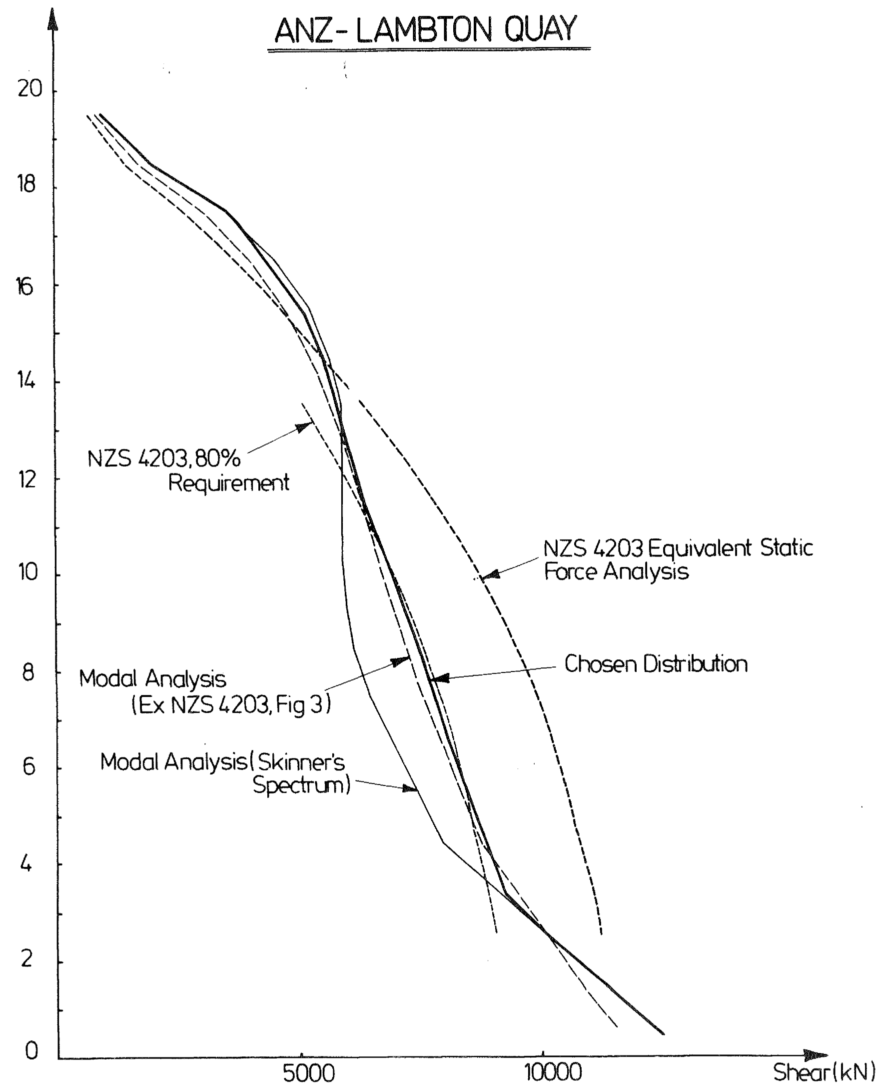

Figure 6: INTER-STOREY SHEAR DISTRIBUTION

Parallel to Featherston Street