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BUILDING RESEARCH ASSOCIATION OF NEW ZEALAND

BRANZ STUDY REPORT

DESIGN AND COST IMPLICATIONS OF DZ 4203 : 1989

Smith Leuchars Ltd

BUILDING RESEARCH ASSOCIATION OF NEW ZEALAND

PREFACE

The Building Research Association of New Zealand (BRANZ) commissioned this report to ascertain the impact of the proposed changes contained in the draft loading code DZ4203:1989 on the building industry.

ACKNOWLEDGEMENTS

The assistance of BRANZ in preparing this report is acknowledged. The assistance of the following consulting engineering firms who carried out the design comparisons of the individual buildings which provided the basic data for the report is acknowledged.

Hadley & Robinson Ltd
KRTA Ltd
Morrison Cooper Ltd
Murray North Ltd
Smith Leuchars Ltd
Spencer Holmes Miller Partners Ltd
Works Consultancy Services
Works Technical Services.

This report is intended for Code Writers, architects, structural engineers and designers.

DESIGN AND COST IMPLICATIONS OF DZ 4203 : 1989

BRANZ Study Report SR24

Smith Leuchars Ltd

REFERENCE

Smith Leuchars Ltd, 1989. Design and Cost Implications of DZ4203:1989. Building Research Association of New Zealand, BRANZ Study Report SR24, Judgeford.

KEYWORDS

From Construction Industry Thesaurus - BRANZ edition: Buildings; Code of practice; Comparing; Cost; Low rise; Multi-storey buildings; Limit state analysis; Loads; Performance concepts; New Zealand; Standards; Structural design.

ABSTRACT

The current loadings code (NZS4203:1984) is being rewritten to reflect advances in research and knowledge in recent years. The latest proposed draft of the revised code (DZ4203:1989) was circulated for comment and a SANZ Review Committee reviewed these comments in June 1989.

In an attempt to ascertain the impact on the building industry of the proposed changes in the draft code a number of consultants were commissioned to redesign projects which have been designed in accordance with NZS4203:1984. The findings of these consultants have been collated to form the basis of this report.

The effect of DZ4203:1989 on the designs of different types of structures is discussed together with indications of the approximate cost differences between structures designed to DZ4203:1989 and NZS4203:1984, and rough orders of cost established.

This contract has been the collaborative efforts of eight consulting engineering practices working under the guidance of one of them acting as principal consultant. In all instances the draft code 2/DZ 4203 : 1989 was applied by each consultant to a structure which had previously been designed in accordance with current code requirements. While every effort has been made to ensure a uniform approach and understanding by all participants, (usually by answering queries as they arose), there has been no technical audit by BRANZ or the principal consultant of either the designs, the interpretation of the draft or current code requirements.

CONTENTS

INTRODUCTION

The draft loading code, General Structural Design and Design Loadings For Buildings (DZ4203:1989), represents a major revision of the current code with the same title (NZS4203:1984). Significant changes are proposed which recognise the increased knowledge which has become available in recent years.

However, it is recognised that building codes must strike a balance between allowing the application of new sophisticated methods in design, while ensuring that excessive complexities in code criteria do not lead to unduly expensive buildings, and to misinterpretation and consequently incorrect and unsafe designs.

To provide an assessment of the implications of the proposed changes, an evaluative study of the draft loading code DZ4203 was undertaken with the aim of satisfying the following objectives:

- To compare the design requirements of DZ4203:1989 with those of NZS4203:1984.
- To identify cost implications of the primary structural elements resulting from the implementation of the requirements of DZ4203:1989.
- To identify ambiguities and points of controversy within DZ4203:1989.

The findings of this study with respect to the first two objectives are presented in this report.

It should be noted that the study was limited to an investigation into the impact of DZ 4203 on the primary structural systems of buildings and did not attempt to examine the consequence of the draft code on non-structural secondary components such as architectural systems - partitions, ceilings, claddings (apart from separations to primary structural elements) canopies etc - building services equipment and the like.

METHODOLOGY

A number of engineering consultants were commissioned to redesign selected structures in accordance with the requirements of DZ4203 (see Table 1). These structures had previously been designed to NZS4203 and they were chosen to give a wide spread of structural type. Limit state supplements to the appropriate material codes were provided by the appropriate material code committees to give any necessary guidance on design parameters not readily available in current standards.

The structures were reanalysed in accordance with the requirements of the draft and the redesign of key structural elements was undertaken to a sufficient degree to allow comparison with the same elements designed in accordance with the existing loadings code

TABLE 1 STRUCTURES SELECTED FOR STUDY

BUILDING TYPE(S)	DESCRIPTION	CONSULTANT
1	Four Domestic Timber Framed Structures	Spencer Holmes Miller Partners Ltd
2	Two Glue Laminated Timber Framed Structures Odlins, Te Marua - glue laminated portal warehouse Renouf Tennis Stadium - timber glulam arched structure	Spencer Holmes Miller Partners Ltd
3	Five lightweight steel-framed farm buildings	Hadley and Robinson Ltd
4	Waiariki Polytechnic, Rotorua 4-storey steel framed building incorporating eccentrically braced frames	Murray-North Ltd
5	F.D.C. House, Auckland 5-storey steel framed office building incorporating eccentrically braced frames (EBF) above Level 1 with reinforced concrete shear walls to the lower level	KRTA Ltd
6	Electric Power Transmission Steel Lattice Tower, 20m high	Works Technical Services
7	30 Bed Barracks Block Unit 2-storey reinforced concrete building incorporating moment resisting H frames of limited ductility in its transverse direction and elastically responding shear walls with cantilever columns above in its longitudinal direction	Works Consultancy Services
8	Wakefield Centre, Wellington 5-storey reinforced concrete retail structure with West-East orientated ductile moment resisting frames and North-South orientated shear walls of limited ductility	Smith Leuchars Ltd (Wellington)
9	Wilson Neill House, Dunedin 10-storey office building incorporating ductile reinforced concrete moment resisting frames	Hadley and Robinson Ltd
10	Unisys House, Wellington 13-storey octagonal shaped office building incorporating ductile reinforced concrete moment resisting frames	Morrison Cooper Ltd
11	28-storey Building, Auckland Office building incorporating ductile reinforced concrete moment resisting frames above level 4 and reinforced concrete shear wall core below level 4	Smith Leuchars Ltd (Auckland)

NZS4203. The effect of the proposed code revisions on serviceability requirements for both the principal structural systems and select secondary elements was also investigated.

Finally, the cost implications of the redesigned structural elements were assessed in broad terms with regard to both building costs and design costs to provide an overall assessment of the likely cost impact of the new draft code.

SUMMARY OF FINDINGS

A general overview of the findings of the twelve studies is presented below. For specific findings applicable to each structure analysed in the study, the reader is referred to the section, **DESIGN COMPARISONS OF BUILDINGS.**

Design Loadings

1. Gravity Loadings

For the structures analysed in this study, the minimum basic live loads given in DZ4203 were generally found to be in excess of those given in NZS4203, with the odd exception for particular floor usages (e.g., wardrooms to institutional buildings).

However, as a consequence of the lower load factors of the strength limit state combination 1.2D + 1.6L of DZ4203, the resulting factored loadings were typically found to be marginally lower from those of the equivalent NZS4203 load combination, $1.4D + 1.7L_R$.

For the other strength limit state combinations of DZ4203, the live load contribution is significantly less than that in the equivalent strength method combinations of NZS4203.

The live load contribution to be considered with the severe seismic limit state combinations of DZ4203 was also significantly lower than that in the equivalent combination of NZS4203.

The live load contribution to the serviceability limit state combinations of DZ4203 tended to be of a similar order as the corresponding loads under NZS4203, with the short term load factor offsetting the increased basic live load and a live load reduction factor of unity.

2. Wind Loadings

The derivation of the design wind loads using DZ4203 was found to be considerably more involved than using NZS4203.

The basic design wind speeds in DZ4203 for some regions e.g., Taupo and Auckland, have been significantly increased over those given in NZS4203 and this resulted in a very large increase in the design wind forces acting on the structures studied in those regions. For

Building Type 11 in Auckland, this increase was as much as 100 per cent.

Structures located within regions where orographic and topographic effects are significant were also subjected to increased wind forces. Increases in the basic wind forces derived from DZ4203 of the order of 82 per cent and 137 per cent due to orographic and topographic effects respectively are possible.

Elsewhere, the design wind forces derived from DZ4203 were typically of a lesser magnitude than those derived from NZS4203.

3. Seismic Loadings

Consideration of the severe seismic design loadings on the structures studied revealed the following trend when compared with NZS4203:

a) For "short" period structures redesigned to DZ4203, there was a dramatic increase in the magnitude of the design seismic forces acting on the structures (e.g., Building Type 2, Building Type 8 North-South direction).

b) For "long" period structures, an appreciable reduction in the design seismic forces was apparent (e.g., Building Types 9 to 11)

Structures which fell between these extremes exhibited a lesser variation in the level of seismic loading, and in some cases there were only marginal differences from corresponding loadings derived from NZS4203 (e.g., Building Types 4 and 7).

For the longer period, taller multi-storey buildings (e.g., Building Types 10 and 11) P-delta loading effects were found to be the more critical of the two severe seismic limit state load combinations over the lower levels of the structures.

The lumped seismic weights to be considered at each floor for derivation of the seismic shear were found to be marginally reduced when using DZ4203, due to a lesser live load contribution.

Structural System Types

1. Building Type 1: Four Domestic Timber Framed Structures

Wind loading tended to govern the designs, as expected. In general, the loads derived from DZ4203 and NZS4203 - and the Code of Practice for Light Timber Frame Buildings Not Requiring Specific Design (NZS3604:1984), from an approximate conversion of bracing units - were comparable. However, the suggested changes contained in the timber code supplement for the conversion of the timber design code parameters contained in the Code of Practice for Timber Design (NZS3603:1981) to limit state design format, resulted in the design approaches using the forces derived from each code varying considerable.

2. Building Type 2: Two Glue Laminated Structures

The wind loads derived from DZ4203 were typically similar to or less than those obtained using NZS4203. However, the seismic forces derived from DZ4203 were appreciably greater than those derived from NZS4203.

Few changes resulted to the portal frame structure when redesigned in the accordance with DZ4203 although a reduction in principal member sizes was possible for the arch stadium structure.

3. Building Type 3: Five Lightweight Steel-framed Farm Buildings

The design of these structures was governed by wind and snow loadings.

It was found that the redesign using DZ4203 resulted in little change to the existing designs except in the following situations:

a) Snow load dominated the design of lean-to type buildings, where significant reduction in loadings resulted which could be reflected in member sizes.

b) Partly enclosed buildings where the ratio of dominant wall openings is such that a markedly reduced internal pressure coefficient resulted with a corresponding reduction in loadings which could be reflected in member sizes.

c) In regions where the orographic, topographic and/or channelling multipliers are significant, the design wind loadings increased appreciably, necessitating increased member sizes.

4. Building Types 4 & 5: Medium Rise Eccentrically Braced Frame Structures (Waiariki Polytechnic Building, Rotorua; FDC House, Auckland)

The design of the eccentrically braced frames comprising the lateral load resisting systems of these two buildings using DZ4203 was governed by the severe seismic forces which were considerably greater than the wind loads.

The new seismic coefficient derived from DZ4203 for the FDC House structure was appreciably less than that used in the original design. However the designers noted that this is not necessarily a reflection of the change of codes but rather the conservative nature of the information available at the time of the original EBF design.

Despite the reduction in base shear for this structure under DZ4203, the severe seismic combination including P-delta effects resulted in marginally increased loadings over that of the original design, necessitating an increase in the sizing of the EBF members.

For the Waiariki Polytechnic building, the increase in seismic loadings over the original design using NZS4203 was of a similar order to that of the EDC House structure, but with the P-delta loading combination being the less critical of the two severe seismic load combinations.

The increase in size of the EBF members of both these structures was expected to have a flow-on effect on their foundations as a consequence of the need to design for the capacity actions of the superstructures.

With only marginal changes in the gravity loads, a reduction in the beam members sizes of the independent gravity frames was found possible in only the Waiariki Polytechnic structure. However, with the suggested changes to the DZ3404/A2 Draft Code for Design of Steel Structures design parameters for use with DZ4203, as provided in the steel code supplement, it was found necessary to increase the size of the columns to the steel gravity frames of the EDC House structure.

The wind loadings derived from DZ4203 for both buildings were significantly greater than those derived from NZS4203.

5. Building Type 6: Steel Lattice Tower

Whilst the use of DZ4203 and NZS4203 are not strictly applicable to the design of such special structures (refer Clause C1.1.1 of DZ4203 and Foreword to NZS4203), it was found that design wind forces derived from the draft code were significantly greater than the corresponding loads derived from NZS4203. The increased forces were due principally to the higher site wind speeds derived from DZ4203, with topographic effects being significant.

It was found necessary to increase the size of typical tower structural members to handle the increased loadings. Seismic loadings were not critical for this structure.

6. Building Type 7: 300-Bed Barracks Block

Seismic loads governed the lateral load design of this two storey structure in both the serviceability and the severe seismic limit states, with the corresponding wind loads only half the values of the seismic loads.

In the transverse direction, where the lateral loads are resisted by limited ductility reinforced concrete H-frames, the severe seismic loads calculated in accordance with DZ4203 were very similar to the design loads for the original design using NZS4203, and no net change to the original structure resulted.

Under the seismic serviceability limit state, racking deflections of the second storey columns exceeded the allowable limits specified in DZ4203 for buildings with inadequate separation of non-structural elements. However, this was considered acceptable for this structure.

In the longitudinal direction, where lateral loads are resisted by precast columns cantilevering above the first floor and in-situ walls at each end between ground and level 1, the severe seismic loads derived using DZ4203 were marginally less then the corresponding loads derived from NZS4203. This allowed a marginal net reduction in the reinforcement content in the walls between ground and first floor and the associated footings.

Little appreciable difference was found in the strength and serviceability loadings on the first floor slab derived from the two codes.

7. Building Type 8 (North-South):
Medium Rise Shear Wall Structure
(Wakefield Centre)

The severe seismic loadings derived from DZ4203 were appreciably greater then those derived from NZS4203, principally due to the short period of the structure and the revised shape of the design response spectra of DZ4203. Consequently, the shear wall sizes and flexural reinforcement contents were increased in accordance with the increased shears and overturning moments acting on these elements.

With such stiff structural elements of limited ductility, P-delta effects were insignificant and the seismic serviceability limit state forces were only a fraction of the governing severe seismic strength values. The wind loads derived from DZ4203 were significantly increased due to the closeness of Port Nicholson (of low terrain roughness value), but were non-critical for the design.

8. Building Types 8 (West-East), 9,10 & 11:
R.C. Ductile Moment Resisting Frame Structures
(Wakefield Centre West - East, Wellington;
Wilson Neill House, Dunedin;
Unisys House, Wellington;
28-storey building, Auckland)

The variation in the level of seismic forces resulting from the use of DZ4203 compared with NZS4203 for these structures is illustrated in Table 2.

Table 2 : Level of Seismic Forces

Structure	Ratio Vbase(DZ4203)/Vbase(NZS4203)	To (secs)
Wakefield Centre (5 level)		
West - East	123%	1.26
Wilson Neill House (11 levels)	44%	1.85
Unisys House (13 levels)		
Major Direction	75%	2.08
Minor Direction	58%	2.66
28 Storey Building, Auckland	28%	3.00

For the longer period structures, considerable reduction in seismic loading resulted, with the converse occurring for the stiffer lower rise, Wakefield Centre structure. This reflected the difference in shape of the response spectra between the current and draft loading codes.

a) Building Type 8 (West-East) (Wakefield Centre)

In the redesign of the Wakefield Centre, it was found that the seismic serviceability limit state governed the flexural design of the beam hinges. Gravity loading contributed significantly to the resulting load combination due to the rather long span of the beams. The column design was dictated by the requirements of capacity design.

The two severe seismic limit state combinations were found to produce load effects of near equal magnitude in the frames of the structure with the P-delta combination the more critical over the first level only.

With the increased seismic loads from DZ4203, the flexural reinforcement content was increased by 11 per cent and 50 per cent in the beams and columns respectively.

b) Building Type 9 (Wilson Neill House)

In the redesign of Wilson Neill House, gravity load considerations together with wind loading governed the flexural design of the beam hinges, with the requirements of capacity design dictating the column design.

The base shears and overturning moments derived from DZ4203 for wind loading were significantly greater in magnitude than the corresponding values for earthquake loading, but were still less than the design loadings derived from NZS4203. As a consequence, significant reductions were possible in the reinforcement to beams and columns.

c) Building Type 10 (Unisys House)

In the redesign of Unisys House, the flexural design of the beam hinges was governed by the seismic serviceability limit state combination.

A reduction in beam size and reinforcement content was possible due to the reduced seismic forces with consequent reductions in column reinforcement under the requirements of capacity design.

Of the two severe seismic limit state combinations, the one including P-delta effects was found to be critical over the lower levels of the structure.

Wind loadings derived from DZ4203 were found to be significantly less than those using NZS4203, and did not govern the design.

d) Building Type 11 (28 Storey Building, Auckland)

In the redesign of the 28 storey building in Auckland, critical beam actions were derived from a number of different load combinations, dependent upon location up the height of the building, illustrated as follows:

i) Top upper quarter of structure - severe seismic limit state (non P-delta combination).

ii) Middle half of structure - strength limit state wind.

iii) Bottom quarter of structure - severe seismic limit state (P-delta combination).

This resulted from a significant reduction in earthquake loadings over that derived using NZS4203, which governed the original design, and a doubling of the wind forces due to a marked increase in the design wind speeds in the Auckland region.

It was found that the redesign actions for the beams were 30 to 70 per cent lower than those of the original design for the building, dependent upon location up the height of the building, allowing a reduction in both beam and column member sizes.

Serviceability Requirements

For the structures of this study, it was generally found that the structural systems designed to NZS4203 adequately satisfied the serviceability deflection and drift requirements of the draft DZ4203. However, for one structure, Building Type 5 (FDC House, Auckland), precambering of gravity beams was considered necessary to satisfy beam deflection limits.

Lateral movements of the structures of this study under strength and severe seismic limit state load combinations fell well short of the proposed limits of the draft code, which appear to be more relaxed than those of the current code.

For buildings of reasonable high ductility demands, greater separation of secondary components is required than previously necessary using NZS4203. For such structures, it would appear that the maximum interstorey drifts which can be accommodated by secondary elements such as external cladding are going to dictate the trial sizings of structural elements.

Cost Implications

1. Building Costs

Estimates of the cost effects of redesigning the structures studied to DZ4203 are given in Table 3.

2. Design Costs

Estimates of the impact of DZ 4203, relative to NZS 4203, on design costs are given in Table 4.

It was recognised by most consultants involved in the study that DZ4203 will result in more work for the designer.
For low rise construction, the increase in work and the corresponding costs may be insignificant.

However, for medium to high rise construction, the increase in loadings and load combinations to be considered in the design procedure has increased substantially. This is reflected to a considerable extent in the designer's time input required for analysis using DZ4203.

Consultants appointed on a time charge basis are likely to be fully compensated for the extra design work. However, for consultants appointed on some form of percentage fee basis, full compensation appears unlikely with the possibility of a fee reduction as building costs are reduced, as evident for Building Types 9, 10 and 11.

Table 3: Building Cost Variations

Building Type	Structure Content	Cost Variations as Percentage (%) of Original Cost: Structural Content	Total Building
1	Four Domestic Timber Framed Buildings	-	-
2	Odlins, Te Marua, Glue Laminated Portal Warehouse	(-1)	N.A.
	Renouf Centre Tennis Stadium, Timber Glulam Arched Structure	(-4.4)	N.A.
3	Five Lightweight Steel-framed Farm Buildings	+50*	+25*
4	Waiariki Polytechnic, Rotorua	+0.1	+0.03
5	FDC House, Auckland	+4.4	+1.5
6	EPT Steel Lattice Tower	+16	+16
7	30 Bed Barracks Block	(-0.2)	(-0.02)
8	Wakefield Centre, Wellington	+1.8	+0.6
9	Wilson Neill House, Dunedin	(-7.5)	(-2.5)
10	Unisys House, Wellington	(-7.0)	(-0.5)
11	28 Storey Building, Auckland	(-6.0)	(-2.0)

* Significant cost increases expected only in orographic regions when the orographic lee multiplier is applicable. Elsewhere little variations in costs expected.

N.A. - Not Available

Table 4: Design Cost Variations

Building Type	Design Cost Variation as Percentage (%) of Original Design Fee
1	+20 (1)
2	+1 (1)
3	0
4	+5 (1)
5	+10 (1)
6	+10
7	0
8	+0.9
9	0
10	+8
11	+20

(1) Based on increase in design time/effort required using DZ 4203.

REFERENCES

Standards Association of New Zealand "Draft Code of Practice for General Structural Design and Design Loadings for Buildings" 2/DZ4203:1989.

Standards Association of New Zealand "Code of Practice for General Structural Design and Design Loadings for Buildings", NZS4203:1984.

Standards Association of New Zealand "Code of Practice for Light Timber Frame Buildings Not Requiring Specific Design", NZS3604;1984.

Standards Association of New Zealand "Code of Practice for Timber Design", NZS3603:1981.

Standards Association of New Zealand "Draft Code for Design of Steel Structures", DZ4203/A2 1988.

DESIGN COMPARISONS OF BUILDINGS

Type 1	:	Domestic Timber-framed Structures
Type 2	:	Glue Laminated Timber-framed Structures
Type 3	:	Lightweight Steel-framed Farm Structures
Type 4	:	Four-storey Steel-framed Building
Type 5	:	Five-storey Steel-framed Building
Type 6	:	Electric Power Transmission Steel Lattice Tower
Type 7	:	Two-storey Reinforced Concrete Building
Type 8	:	Five-storey Reinforced Concrete Retail Structure
Type 9	:	Ten-storey Office Building
Type 10	:	Thirteen-storey Office Building
Type 11	:	Twenty-eight-storey Office Building

BUILDING TYPE 1

Four Domestic Timber Framed Structures

Spencer Holmes Miller Partners Ltd
P.O. Box 588
Wellington

INTRODUCTION

The objective of this study has been to consider the design requirements of DZ 4203 : 1989 with those of NZS 4203 : 1984 with particular regard to the implications for the design of domestic structures using NZS 3604. Most domestic timber framed construction is carried out in accordance with NZS 3604; this standard determines the required lateral strength to resist wind and seismic loads in terms of bracing units (for superstructure) and braces (for subfloor structure). Bracing units are relative to both the strength and stiffness of the particular form of the load resisting element, with minimum requirement of 5 kN = 100 bracing units.

The specific relationship of resistance (bracing units) to load is complex, and cannot be specifically related to static, and/or seismic loading. In general NZS 3604 provided a more conservative result than through specific design using NZS 4203 and NZS 3603.

Where the original design was executed using NZS 3604 the lateral resistance has been transformed into an equivalent load at 5 kN = 100 bracing units; and where specific wind design using NZS 4203 was used the resulting loads have been increased by factor of 1.3 to provide an equivalent ultimate load capability requirement for comparison with DZ 4203 analysis. Seismic loads to NZS 4203 have been determined at 1.0E again for an equivalent ultimate load capacity requirement.

Codes used have been:

-	NZS	3603 :	1987	Timber Design
-	NZS	3604 :	1984	Light Timber Framed Construction
-	NZS	4203 :	1987	Loadings Code
-	NZS	4203 :	1989	Draft Loadings Code
-	PZ	3603 -		B. Walford Modifications of NZS 3603 for Limit State Design of 17 April 1989.

HOUSE No. 1

Description This is a 99 sq.m dwelling with a partial basement, located in Johnsonville, Wellington (Seismic Zone A, High Wind Area). Roof construction is regarded as light, external wall cladding is weatherboard. See also Fig. 1.

Analysis DZ 4203 NZS 4203 and NZS 3604

Wind Loading -	Simplified Approach	NZS 3604 Table II NZS 4203
Site Criteria -	Surburban Sheltered Bx = 0.6 Bt = 1.0 Bo = 1.0 Br = 0.85 (roof only)	GR = 3.0; S = 1 Cpe + 0.8, -0.5 (Walls)

Total Forces

X	direction 25.7 kN (ult)	37.3 kN	(3604 converted)
Y	direction 23.5 kN (ult)	24.9 kN	(3604 converted)

Conclusion

NZS 3604 bracing units, converted to forces, provide in this instance greater resistance than would be required by DZ 4203.

HOUSE No. 2

Description This is a two storey beach house/boat shed at Lake Taupo which required specific design but made use of conventional NZS 3604 light timber framed elements, see also Fig. 2.

Analysis	DZ 4203	NZS 4203 and NZS 3604
Wind Loading -	Detailed Procedure Category 2 V = 48 m/s	 V = 35 m/s GR = 2

Total Forces

X direction	58.6 kN	35.8 kN (1.3W)
Y direction	18.3 kN	10.5 kN (1.3W)
Seismic	Not governing	Not governing

Conclusion

The lateral forces resulting from DZ 4203 are significantly greater than determined from NZS 4203; the increase in design loads can be almost directly attributed to the increase in design wind velocity.

HOUSE No. 3

Description This is a two storey dwelling located on river flats of Lower Hutt incorporating a brick veneer heavy tile roof. The balance of the structure is timber framed on a concrete raft slab. See also Fig. 3a and 3b.

Analysis	DZ 4203		NZS 4203	
Wind Loading	- Detailed Procedure Category 3.		GR = 3	S = 1.0

Total Forces

X direction	69.7 kN	74 kN	(1.3W)
Y direction	22.2 kN	22 kN	(1.3W)
Seismic Loadings	38 kN	36 kN	

Conclusion

The derived loadings are comparable between the two codes, and thus the bracing systems would appear not to be effected. Use of NZS 3604 results in significantly higher design loads than NZS 4203 in this instance.

HOUSE No. 4

Description This is a 3 and 2 storey house (in parts) located on an exposed hill top at Whitby, North of Wellington. See also Fig. 4a and 4b.

Analysis	DZ 4203	NZS 4203	
Wind Loadings	- Simplified Approach	GR =	1

	Surburban Exposed B_x = 0.93 B_t = 1.9 All other parameters = 1.0		V = 50 & V = 46 m/s by direction	
North to South	151	kN	139	kN
East to West	120.9	kN	97.6	kN
Seismic Loadings	Non-governing		Non-governing	

Conclusion

The loading derived from DZ 4203 are considerably higher than those of NZS 4203, especially in the East - West direction. This anomaly arises because while directional wind speed may be used under NZS 4203 the technique can not be applied to simplified wind design to DZ 4203. Generally forces in the North - South direction are increased by 8 per cent, which would not be greatly significant to bracing. The 24 per cent increase in the East - West loading however would involve a significant amount of additional design effort and materials. Design using the detailed procedures of DZ 4203 will allow using directional wind speeds and a closer examination of other parameters which may result in reduced loadings - at an increased effort and possibly minimal change in materials.

SUMMARY

Four houses originally designed to NZS 4203/NZS 3604 have been subject to a re-analysis, to DZ 4203.

Because NZS 3604 uses the concept of bracing units it has been necessary to provide an approximate conversion of these to an equivalent NZS 4203 ultimate load.

DZ 4203 does not provide for application of a simplified approach for roofs at greater than 30 degrees. This could possibly result in more design work as the detailed procedure is more difficult to apply. Depending upon the structure more or less work will be required once the analysis has been completed.

In general the loads derived by each code are comparable.

Design technique using the forces resulting from each form of analysis, however, differs considerably.

Under NZS 4203 alternative design/NZS 3603 allowable material stresses for wind or earthquake are increased by 50 per cent(K = 1.5). Similarly allowable connector loads are increased by 50 per cent (K = 1.5).

DZ 4203 - PZ 3603 provides allowable stresses that are multiplied by 0.9 for wind and 1.0 for earthquake. However for the limit state conditions PZ 3603 allowable material stresses are increased by between 1.43 and 2.22 on NZS 3603 values while allowable nail loads are increased by a factor of 4.15. It would appear that DZ 4203 will result in lesser nailing requirements.

DESIGN EFFORT

DZ4203 requires approximately 20 per cent more effort than NZS4203 on structres at this scale. A comparison with the effort for design to NZS3604 is inappropriate.

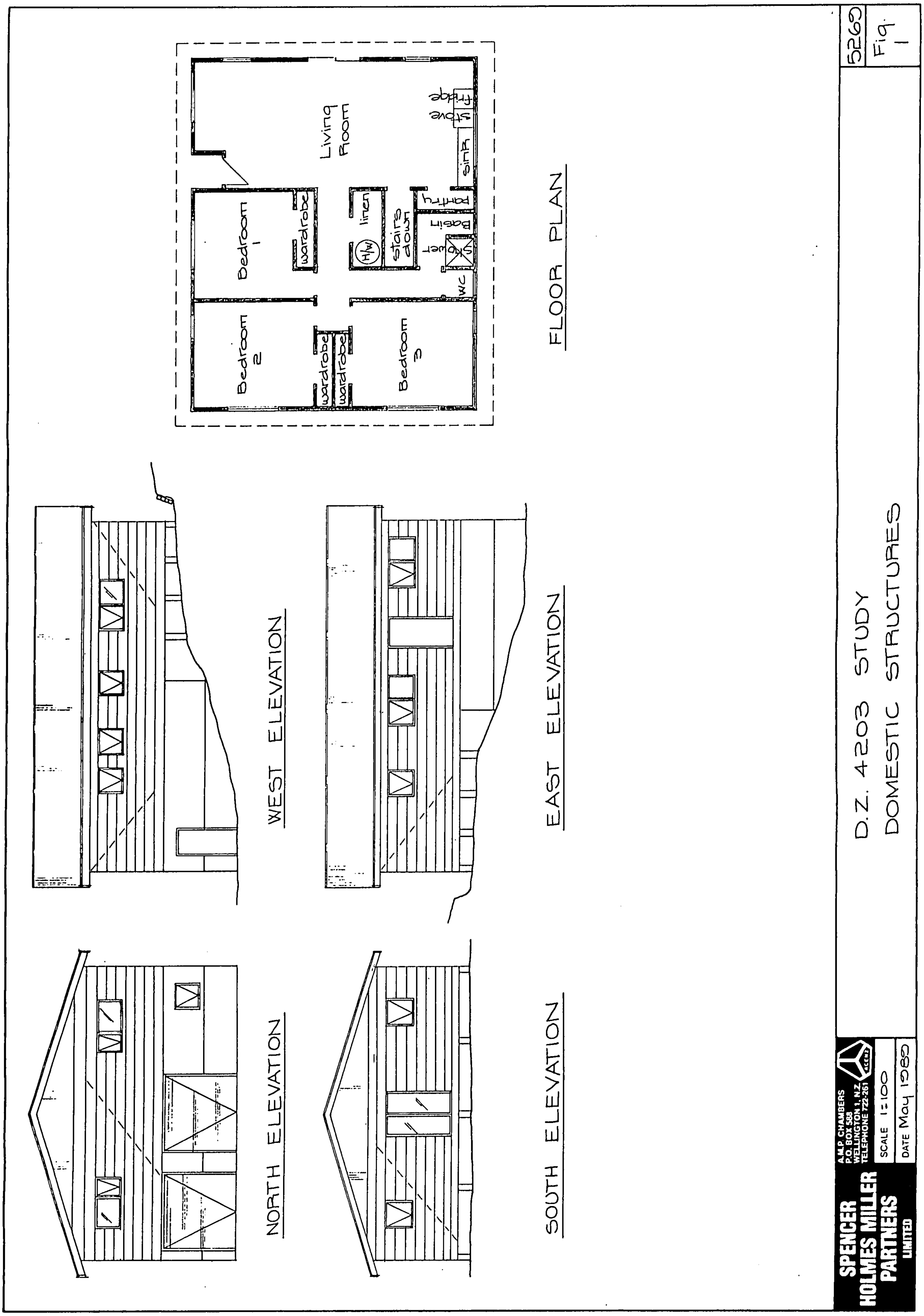

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Fig. 1
Living Room
stove
fridge
sink
pantry
Basin
Shower
WC
Bedroom 1
wardrobe
linen
H/W
stairs down
Bedroom 2
wardrobe
wardrobe
Bedroom 3
FLOOR PLAN
WEST ELEVATION
EAST ELEVATION
NORTH ELEVATION
SOUTH ELEVATION
D.Z. 4203 STUDY
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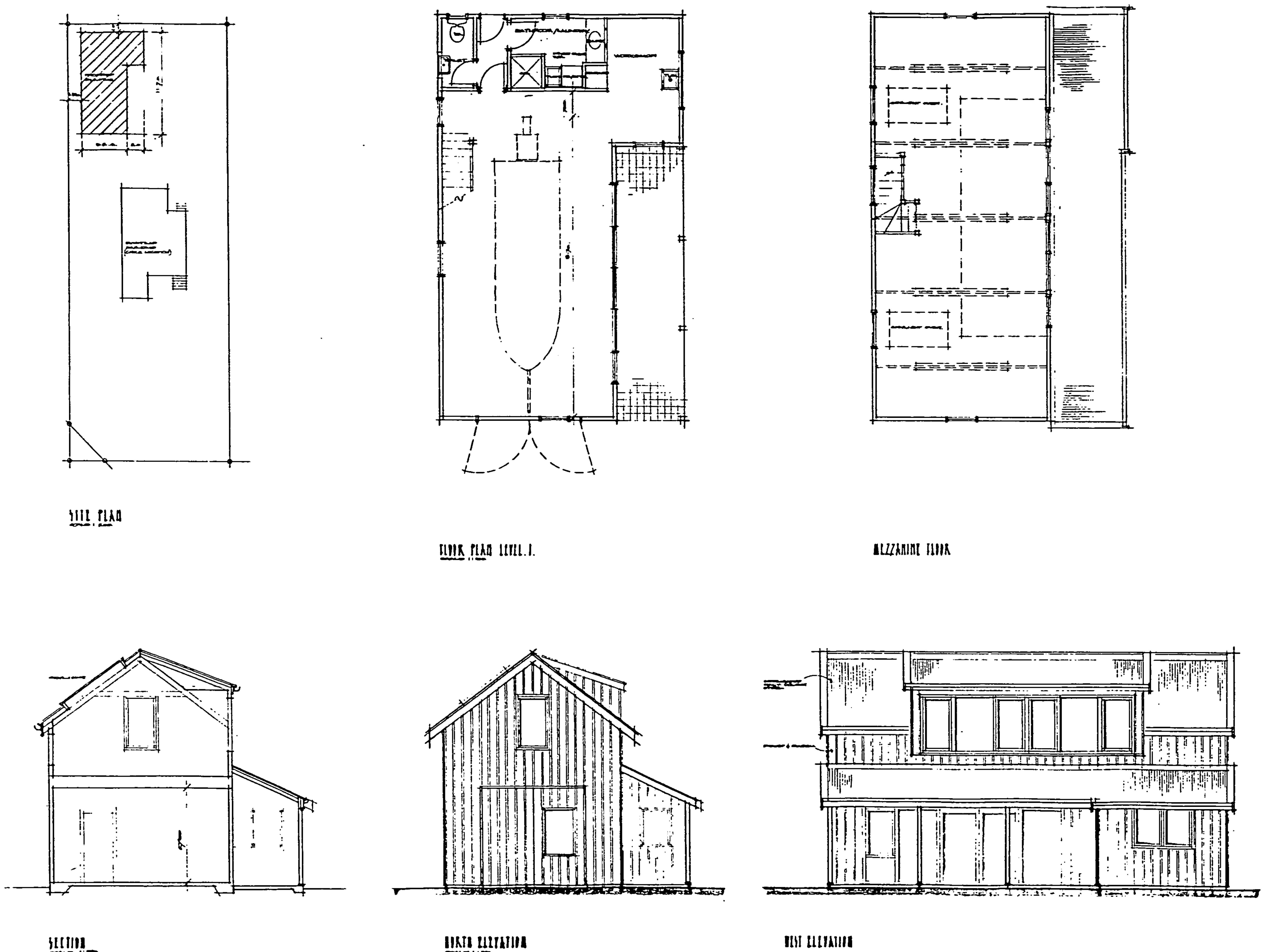
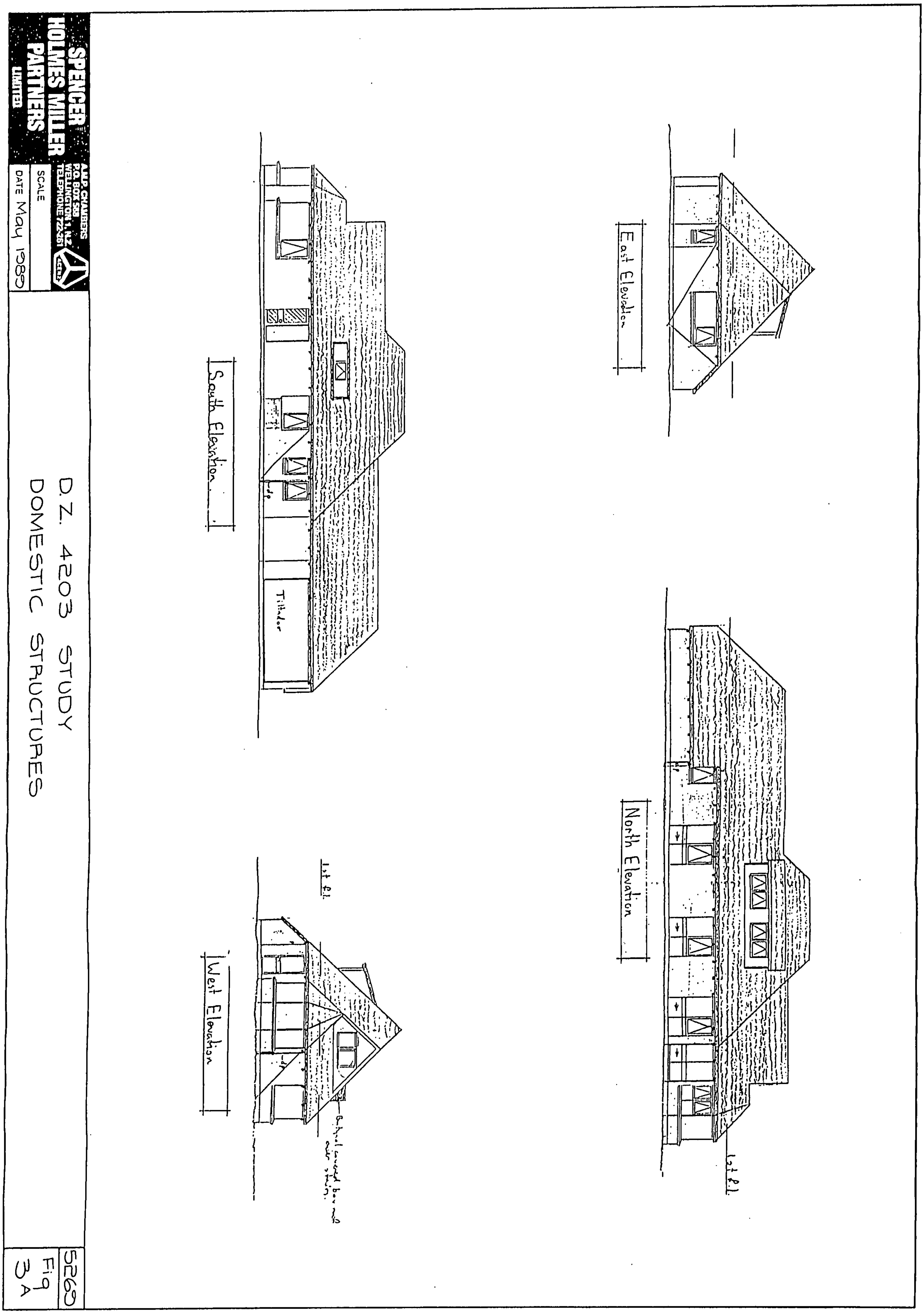
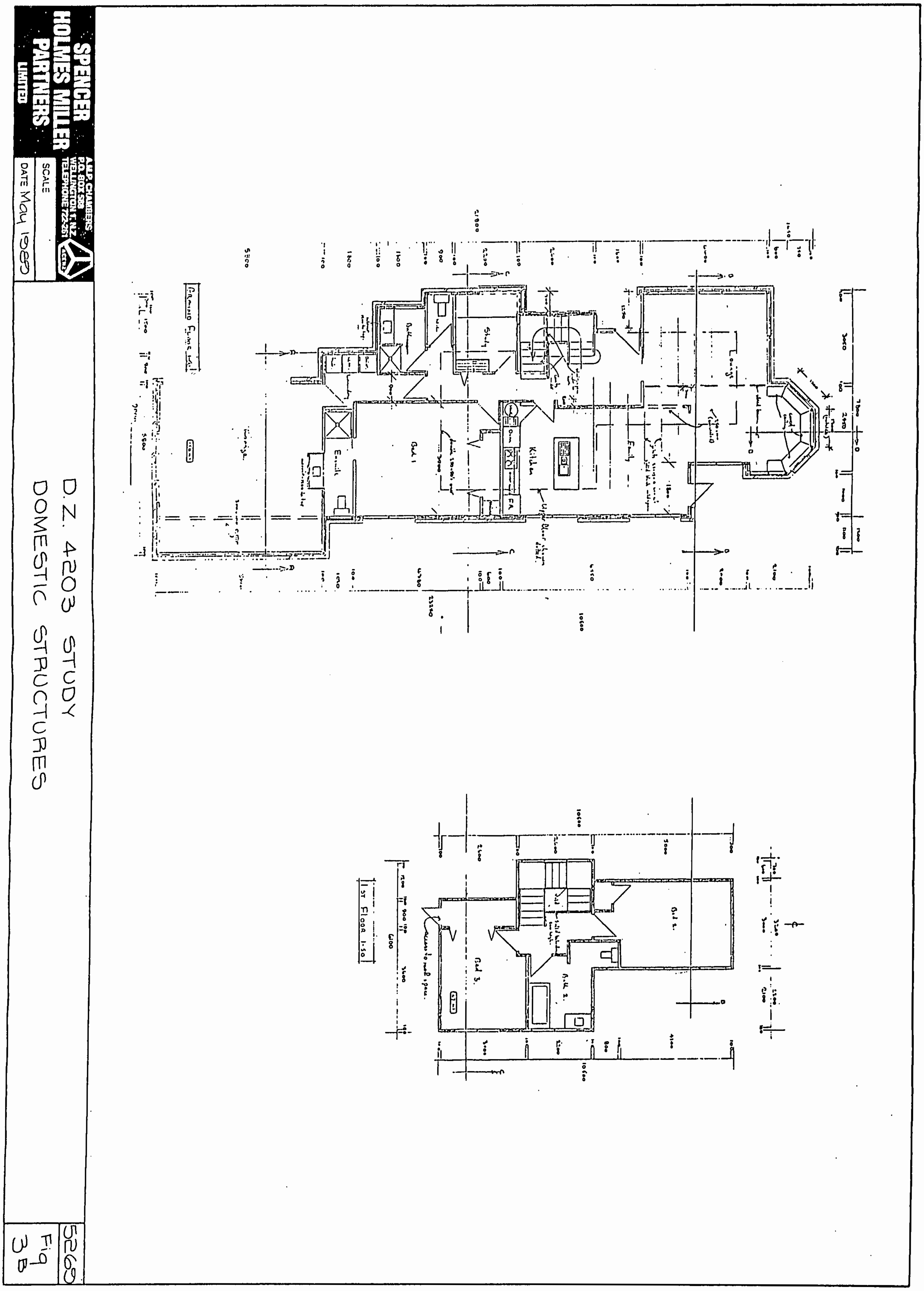
SCALE 1:100
DATE May 1982

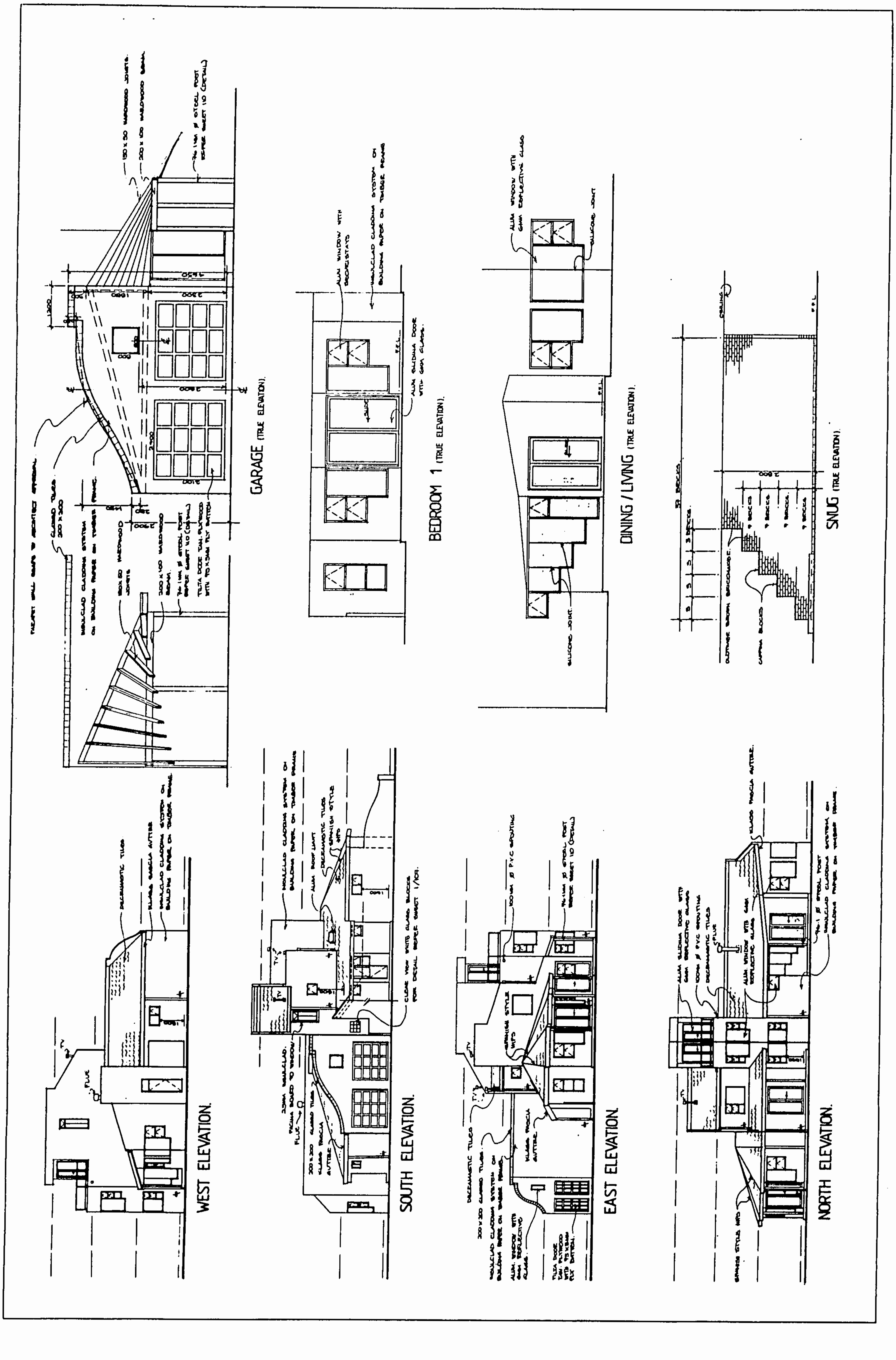


D.Z. 4203 STUDY – DOMESTIC STRUCTURES

Fig. 2

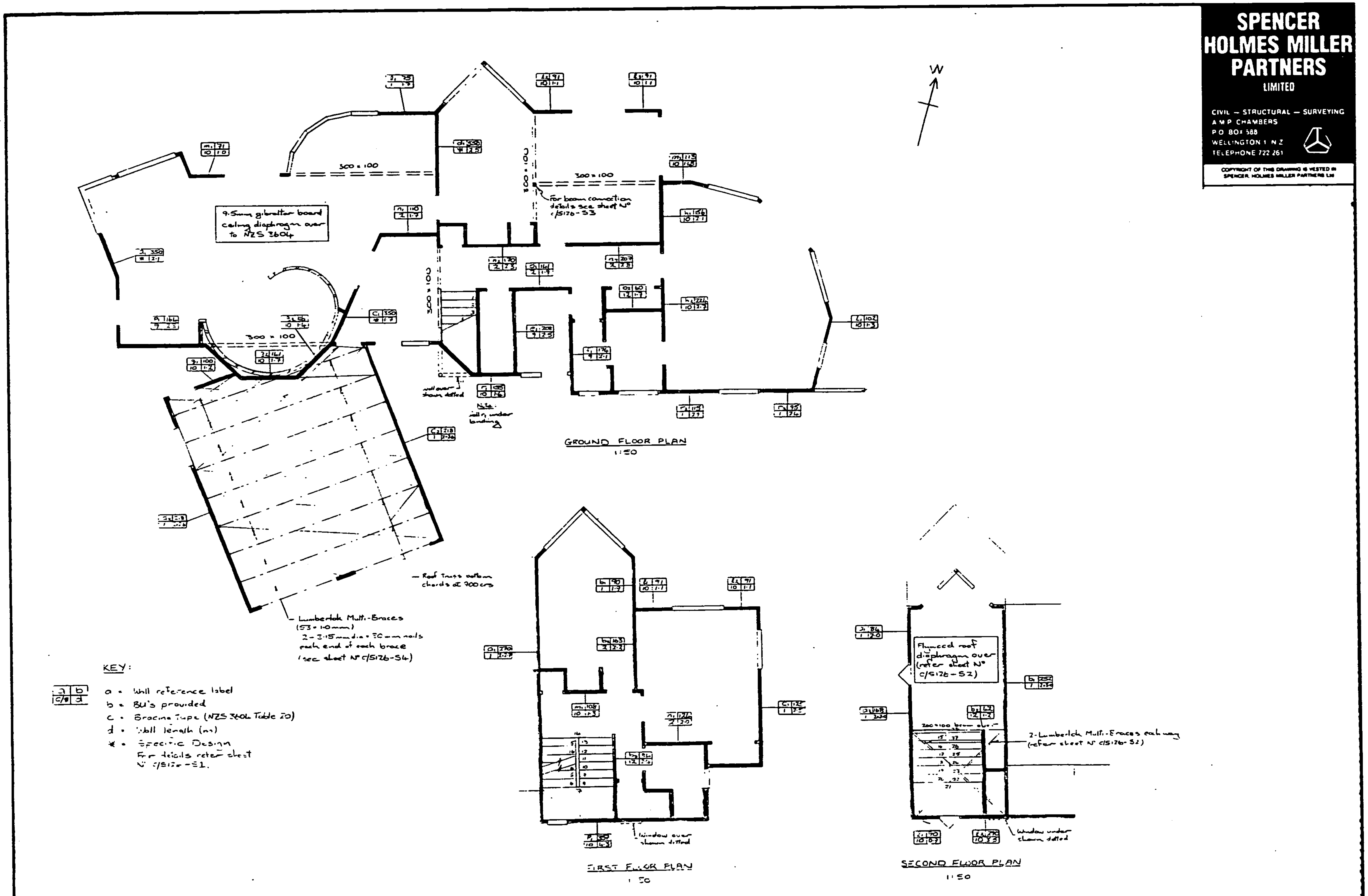

East Elevation
North Elevation
South Elevation
Tiltadoor
West Elevation
SPENCER HOLMES MILLER PARTNERS LIMITED
A.M.P. CHAMBERS
P.O. BOX 308
WELLINGTON 1, N.Z.
TELEPHONE 722-261
SCALE
DATE May 1989
D.Z. 4203 STUDY
DOMESTIC STRUCTURES
5260
Fig 3A


SPENCER HOLMES MILLER PARTNERS LIMITED
A.M.P. CHAMBERS P.O. BOX 528 WELLINGTON 1, N.Z. TELEPHONE 722-251
SCALE
DATE May 1989
D.Z. 4203 STUDY
DOMESTIC STRUCTURES
5260
Fig 3B
Ground Floor 1:50
1st Floor 1:50



D.Z. 4203 STUDY - DOMESTIC STRUCTURES

Fig.4A



D.Z. 4203 STUDY - DOMESTIC STRUCTURES

Fig. 4B

BUILDING TYPE 2

Two Glue Laminated Timber Framed Structures

(Odlins Ltd, Te Marua - glue laminated portal warehouse; Renouf Tennis Stadium - timber glulam arched structure)

Spencer Holmes Miller Partners Ltd
P.O. Box 588
Wellington

INTRODUCTION

The object of this exercise has been to execute the design of some principal elements of two timber framed structures using DZ 4203 and to consider design methods, efforts and cost implications in comparison with the original designs to DZ 4203 : 1984. Timber design for Limit State conditions has been undertaken using data provided by G.B. Walford, dated 17 April 1989 (Ref.2 Appendix A).

ODLINS WAREHOUSE

General

This is a double bay glue laminated portal warehouse located at Te Marua, North of Upper Hutt. The portal frames (refer Fig. 1) are at 6.5 m centres with a total warehouse area of 780 sq.m. Each portal bay is 15 metres giving a 30 m building width. Joints in the portals utilise steel plates and nails, and fixed bases are achieved using piled foundations. Timber purlins and girts and metal roofing form the cladding. The design parameters under both loading codes are tabulated in Table 1.

Analysis

Under DZ 4203 the gravity load dominates and at u = 1.0 seismic loading was not critical. The portal frame members and purlins remained unchanged under DZ 4203 but the nailing requirements are reduced considerably (see Fig. 2).

DZ 4203 simplified approach to wind loads were less then those obtained using NZS 4203 (see Table 1). We note that the simplified approach to wind loads to DZ 4203 does not give any guidance for wind pressures for pitch and trough roof structures, and so an approximation, which allowed for a variation in pressures across the roof, was used. Lateral deflection at the knee was found to be less than the recommended maximum deflection.

Costing

The costs for construction will only be affected by the reduction in the number of nails in the connections for this structure. An estimated saving of $2,350 would be anticipated by use of DZ 4203.

This structure unfortunately has not proceeded to construction, and definitive structural costs are not available. The cost saving on the overall structure has been estimated at just under 1 per cent, and the saving on the portal frames is estimated at 5 per cent.

RENOUF TENNIS STADIUM

General

This is a glue laminated arch stadium located at Central Park, Wellington. The building comprises 4 bays at approximately 16 m spanning 41 m, and is set very close to surrounding hills with only a relatively short length of open ground to one side. The stadium covers approximately 2600 sq.m and the centre three arches are doubled glue laminated members with pinned bases and two splices per member. Purlins are glue laminated, with lateral support at quarter points. See Figure 3 for the plan and cross-section of this building.

Analysis

For both NZS 4203 and DZ 4203 the structure was designed for elastic response (i.e., non-ductile performance) and because of the size of the structure DZ 4203 wind loads were derived using the Detailed procedure.

Deflection/serviceability criteria were not critical, particularly as the purlins receive arch support from the members provided for lateral stability.

For both the arches and the purlins a reduction in size of the member was possible; using DZ 4203, a saving of approximately 11 per cent of timber volume was achieved. for the purlins and the shoe detail the nailing to the end connections was considerably reduced. The reduction in size of the arch members resulted in a small increase in the nailing and dowels at the splice connections. A comparison of two design approaches is shown on Table 2 and details shown on Fig. 4.

Costing

We estimate the saving in timber for the arches and purlins for this project at approximately $24,000 with a further saving of $360 from the connections.

The stadium comprises part of a complex and it has been necessary to assess this section as part of the total work. It appears that the cost saving on the overall structural content of the stadium is approximately 4.4. per cent and the cost saving on the glue laminated content approximately 6.2 per cent.

GENERAL COMMENTS

We note there are no serviceability requirements for detailing with timber connections. Repeated live and wind loads at serviceability levels should not result in unsightly movement of joints. To this end it may be desirable to consider some suitable level of allowable stresses or loads to ensure that this does not occur. Using 50 per cent of maximum allowable loads on nails under serviceability loads would have had a serious effect on the arch structure and a minor effect on the portal frames.

The draft code provided for 8 per cent of the seismic load to be applied at the roof level for structures or towers. Because the form of construction of the arch is quite regular, and because the whole is in fact "the roof", no such loading was applied in this exercise.

CONCLUSION

The DZ 4203 will result in more work for the designer, and produces some saving in cost for timber structures. We note especially that the increases in allowable loads for nails in shear appears significantly greater than the adjustment for general stresses, and this point should be confirmed before proceeding further.

Consideration may need to be given to serviceability criteria for connections in timber structures.

The additional design effort relates largely to the additional load cases that need to be considered during analysis. The effort involved in the design of elements and joints should not vary significantly from previous practice.

It would appear that possibly a 1 per cent increase in the original design effort may be necessary. Thus an increase in the ratio of fees to building cost may be expected, which is however offset by the net saving.

These figures relate to the documents considered for this study. Changes to either the loadings or the materials codes could significantly effect the conclusions.

TABLE 1 - ODLINS WAREHOUSE - DESIGN LOADS

DESCRIPTION	NZS 4203	DZ 4203
Loadings		
Wind	GR 2.0	Simplified Approach
	S1 = 1.0	B x = 0.9
		B t = 1.0
		B c = 1.0
		B o = 1.0
		B r = 0.8
Roof and Wall Pressures (kPa)	a* +0.423	+0.383
	b* -0.845	-0.887
	c* -0.761	-0.536
	d* -0.507	-0.30
	e* -0.676	-0.536
	f* -0.845	-0.612
Seismic	Cd = CRSM	$C(T,\mu) = Co(T,\mu).R.Z.$
	C = 0.15	u = 1.0, T = 0.4
	R = 1.0	Z = 0.8
	SM = 2.4	Co = 0.8
	Cd = 0.36	C(T) = 0.64 (SLT)
		= 0.11 (SLTs)
Load Combination	Alternative Design	Strength Limit State
	1. D + L	1. 1.2D + 1.6L
	2. 0.7D + W	2. 0.9D + W
	3. D + 0.8E	3. D + E

TABLE 1 ODLINS WAREHOUSE - DESIGN LOADS (Continued)

DESCRIPTION	NZS 4203	DZ 4203
		Serviceability Limit State
		1. D + L 2. D + L + E 3. D + L + W
Recommended Deflection	△ eaves = 10mm (W) △ eaves = 20mm (E)	△ = 19.5mm
Actual Deflection	△ = 12mm (W) △ = 15mm (E)	△ = 8.0mm (W) △ = 17.0mm (W)
Moments (kNm)		
Knee	45.5 (D + L)	64 (1.2D + 1.6L)
Base	50.0 (0.7 D + W)	58 (0.9D + W) 53 (1.2D + 1.6L)
Apex	20.5 (D + L)	28.7 (1.2D + 1.6L)
Centre	52.6 (D + L)	74.0 (1.2D + 1.6L)
No. of nails/side/joint		
Knee	190	119
Base	190	119
Apex	79	64
Centre	237	135
Timber Portal	550 x 100 glulam	
Purlins	250 x 50 timber	

TABLE 2 - RENOUF TENNIS CENTRE - DESIGN LOADS

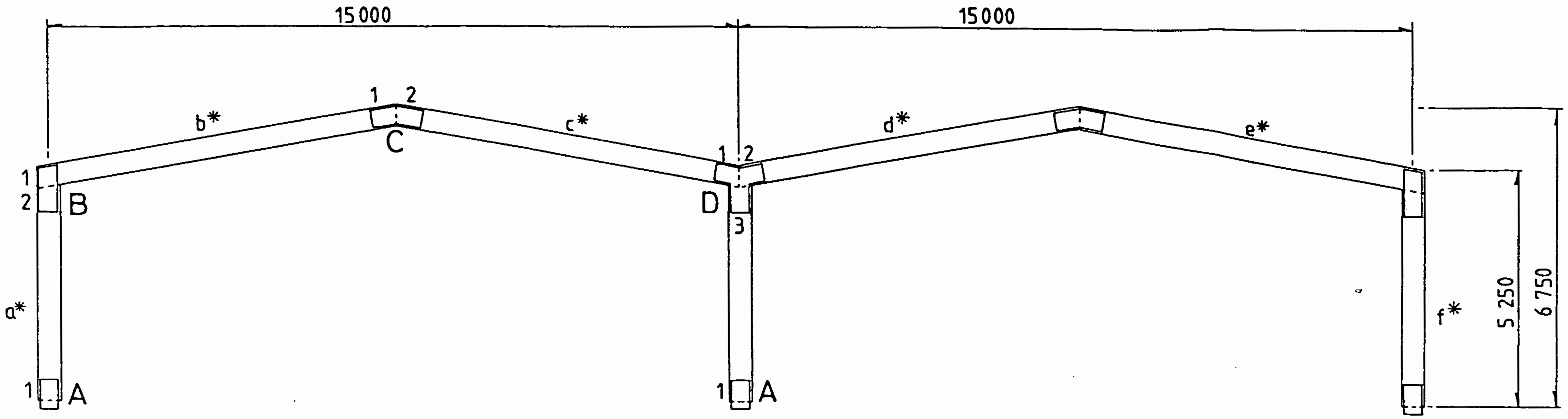
DESCRIPTION	NZS 4203	DZ 4203
Loadings		
Wind	h = 7 Class C GR 3 S1 = 1.0 S2 = 0.65 Vs = 32.5 m/s q = 0.647 kPa Cpi = + 0.3 K1 = 1.5	Category 3 Mzcat = 0.85 $M_{s\theta}$, $M_{t\theta}$, $M_{o\theta}$, $M_{c\theta}$, Me , Mi = 1.0 Area reduction = .8 q = 0.867 Cpi = + 0.2 K1 = 0.1 ϕ = 1.0 for glulam 0.9 for connections.
Seismic	Cd = CSMR = .15 x 2.4 x 1 = 0.36 K = 1.5	Class II, R = 1.2 μe = 1.0, T1 < 0.4 Co(T,μ) = 0.8 z = 0.8 C (T) = Z R Co = .8 x 1.2 x .8 = 0.768 K1 = 1.5 0 = 1.0
Deflections		No criteria

TABLE 2 - RENOUF TENNIS CENTRE - DESIGN LOADS (Continued)

DESCRIPTION	NZS 4203	DZ 4203
Load Combinations	Alternative Design 1. D + L 2. 0.7D + W 3. D + 0.8E	Strength Limit State 1. 1.2D & 1.6L 2. 0.9D & W 3. 1.4D Severe Seismic State 4. D & E Serviceability L.S. 5. D & Ls 6. D & Ws
Arches		
Worst Load Case General	D + L: Mt 326 kNm P 222 kN V 8 kN	D + E Mt 612 P 217 V 3
Splice	0.7 D + W Mt 161 P 124 V 43	.9 D + W Mt 322 P 156 V 59
Base Shoe	.7 D + W Mt 0 P -77.5 V 84	.9 D + W Mt 0 P -68 V 26

TABLE 2 - RENOUF TENNIS CENTRE - DESIGN LOADS (Continued)

DESCRIPTION	NZS 4203	DZ 4203
Purlins		
Load Combinations	Alternative Design 1. D + L 2. 0.7 D + W	Strength Limit State 1. 1.4D 2. 1.2 D + 1.6 Ls 3. 0.9 D + W
Worst Load Case		
Strengths	D + L Mt 23.11 kNm V/2 5.69 kN	1.2 D + 1.6 L Mt 54.22 kNm V/2 14.83 kN
Serviceability	L_R + Kcp D $\triangle$ < .006	$\triangle \leqslant$ .0033
	But eliminated by arch support at quarter points.	



TYPICAL ELEVATION OF PORTAL FRAME (5 off required)

All members 495 (11 laminations of 45 mm) × 94 Glulam.

Plate Type / Leg		Number of nails required NZS 4203:1984	DZ 4203: 1989
A	1	190	119
B	1	190	119
	2	190	119
C	1	79	64
	2	79	64
D	1	237	135
	2	237	135
	3	190	119

All nail plates to be 5mm thick (each side)

All nails to be 40 × 4mm dia. flat head nails, galvanised.

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A.M.P. CHAMBERS P.O. BOX 588 WELLINGTON 1, N.Z. TELEPHONE 722-261

SCALE 1:100

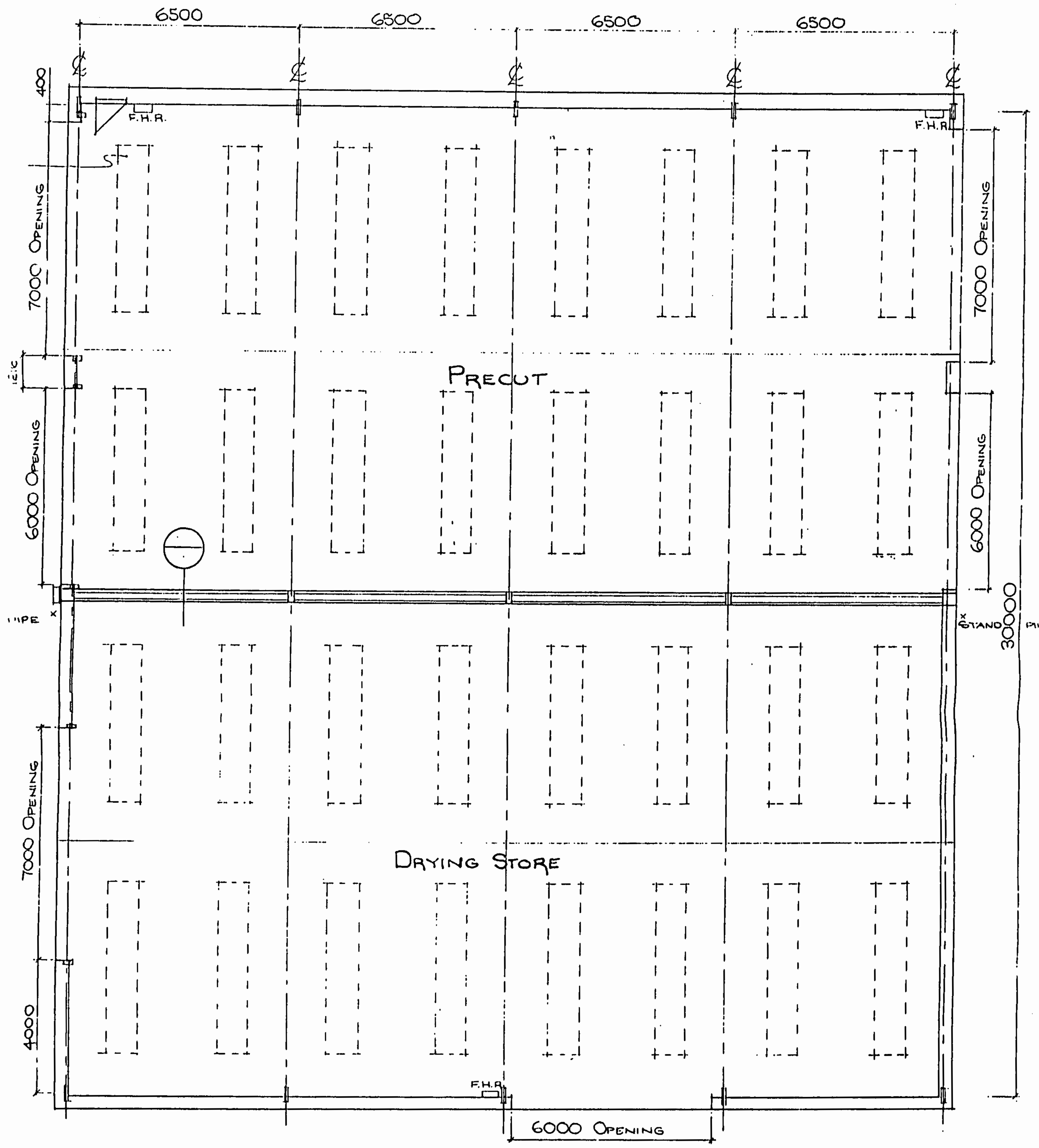
DATE May 1989

ODLINS LIMITED — TE MARUA

PORTAL FRAME ELEVATION

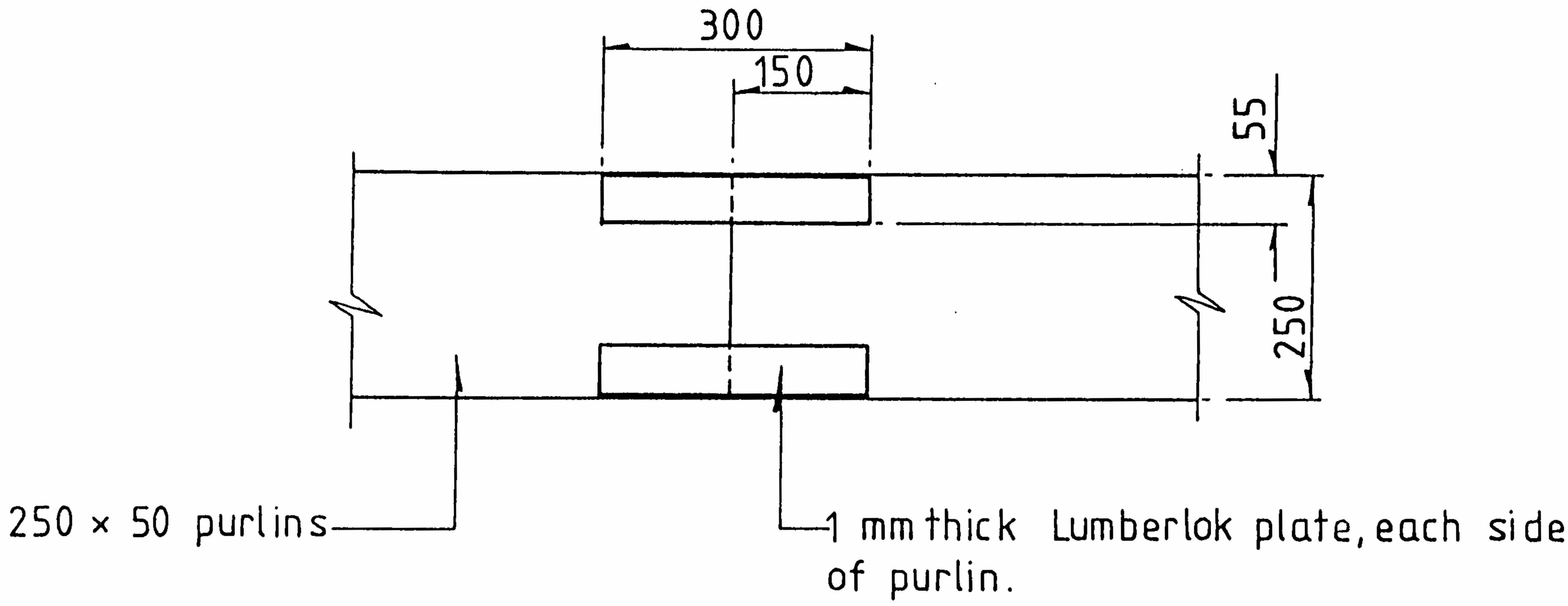
5269

Fig 1

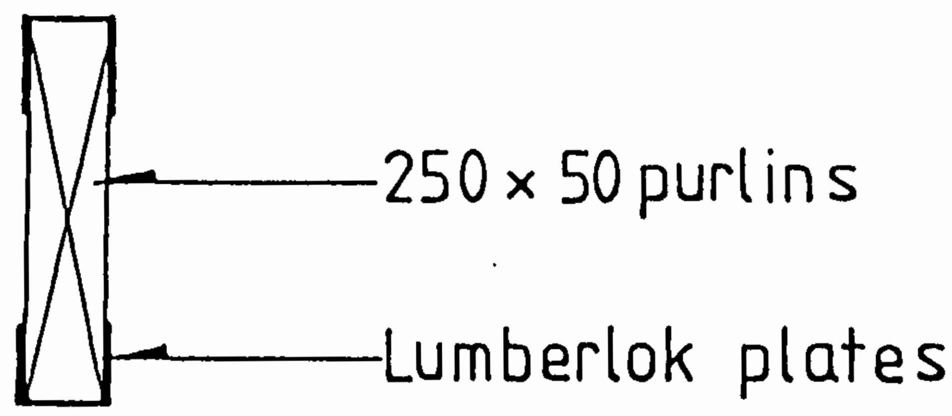


Floor Plan. Scale 1:100

Fig 1(a)



ELEVATION



SECTION

Number of nails / leg	
NZS 4203 1984	15
DZ 4203 1989	10

All nails to be 30 × 3·15mm dia. flat head nails, galvanised.

Total number of purlin splices required = 138

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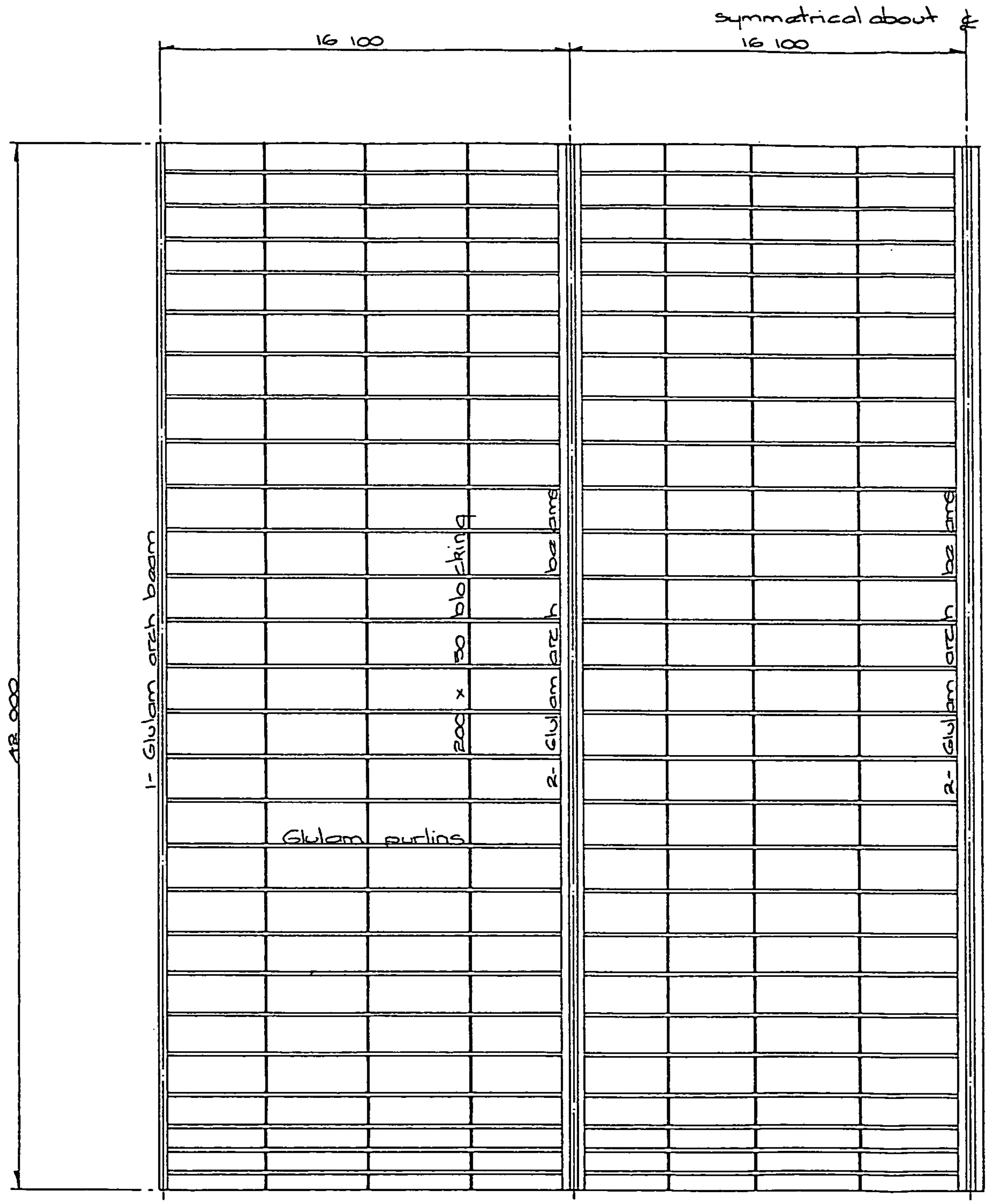
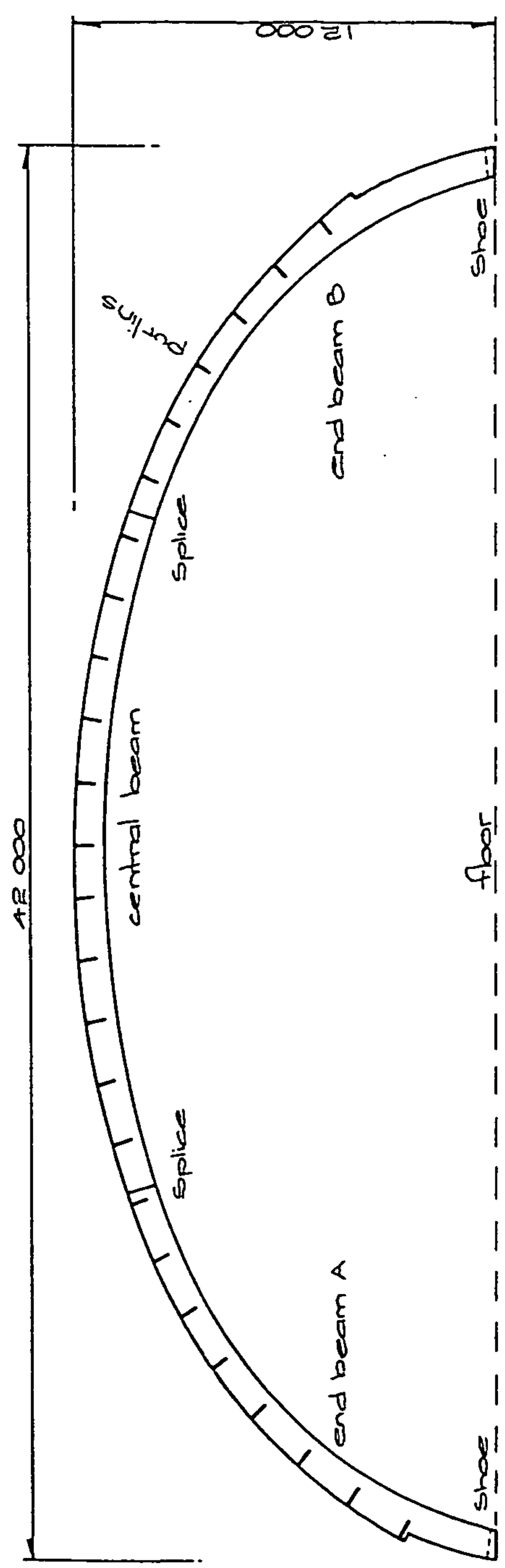
	SCALE 1: 10	PURLIN SPLICE DETAIL	Fig. 2
	DATE May 1989		



Roof Framing Plan 1:200



Typical Elevation of Arch Beam 1:200

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A.M.P. CHAMBERS
P.O. BOX 588
WELLINGTON 1, N.Z.
TELEPHONE 722-261

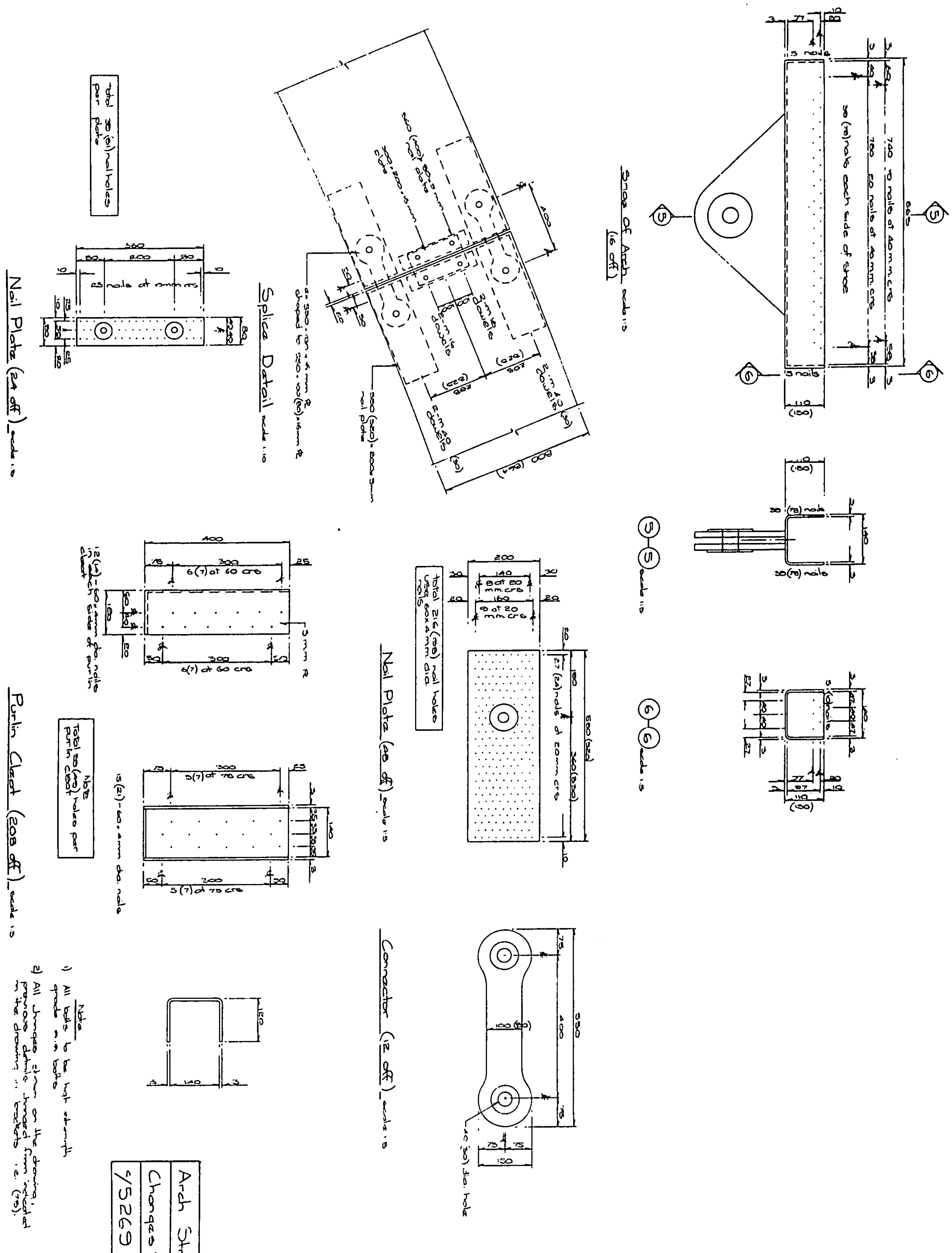
SCALE as shown

DATE May 1989

BUILDING 2 GLUE LAMINATED ARCH STADIUM

5269

Fig 3


Shoe of Arch (16 off) scale 1:5
Splice Detail scale 1:10
Nail Plate (24 off) scale 1:5
Nail Plate (48 off) scale 1:5
Purlin Cleat (208 off) scale 1:5
Connector (12 off) scale 1:10
Arch Structure
Changes in Connections
Fig 4

BUILDING TYPE 3

Five lightweight steel-framed farm buildings

Hadley and Robinson Ltd
PO Box 6068
DUNEDIN

INTRODUCTION

Our office has been designing farm buildings for a prominent South Island steel fabricator and building contractor for well over a decade. During this time, between one and two thousand farm buildings have been erected, mainly in the Otago/Southland region but also as far afield as the North Island.

Five typical farm buildings have been redesigned to 2/DZ 4203, for both snow and wind loads.

The buildings are supported on mass concrete pads with a belled-out base to aid resistance to wind uplift. In general, ground conditions are either firm clay or a compact silt/clay/gravel matrix.

The exact siting and positioning of the buildings is unknown. Geographical information received from the client is limited to

(i) Approximate location
(ii) Altitude
(iii) A rough indication of the exposure of the site

PRESENT FARM BUILDING DESIGN

This design office uses the strength method of design for the design of farm buildings.

Reduced load factors are used in lieu of the increased allowable stresses specified in the farm building code, NZS 1900 chpt 11.2. The relevant combinations of factored load used are:

$$U = 1.1\ D + 1.1\ S$$
$$U = 0.7\ D + 1.0\ W$$

Topographical effects are presently handled by arbitrarily setting the topographical factor, S1 to 1.0, 1.2 or 1.4 depending on the advice received on the exposure of the site - normal, high or extreme respectively. Although Appendix A of NZS4203 is not used, buildings on or near the edge of an escarpment are designed under the 'high' or 'extreme' exposure as appropriate.

Historically, the farm buildings designed in this office have performed to expectations. No collapses due to snow have been reported and few due to wind. Of the damage that has occurred, partial loss of cladding with, perhaps, an occasional purlin splitting longitudinally through the bolt hole, is the most common. Damage to structural steel frames has occurred only three or four times. These failures were due to erecting the building in an extremely exposed location, a location for which the building was not designed.

These buildings tended to be kitset buildings where the farmer himself erected the building, without informing the contractor of the true nature of the site, or where the farmer erected the building in a different, more exposed location on his farm, generally on top of a high hill (100m+).

DESIGN PROCEDURE FOR 2/DZ 4203

The only major influence 2/DZ 4203 had on the design procedure was the derivation of wind load.

Because geographical information for each site was not known it was assumed for this study that the back wall of the buildings faced south and the topographic and channelling multipliers were 1.00. In determining the orographic lee multiplier, the distance to the upwind range was guessed.

In general, accurate geographical information is difficult to obtain for most rural sites. As costs prohibit a site visit by the designer, the client is relied upon for all information.

The topographical multiplier in particular, will be difficult to evaluate as even a 2 or 3 metre high hill is likely to significantly influence the wind load.

COMPARISON OF DESIGN PARAMETERS

Table 1: Comparison of Design Parameters

FACTORED SNOW LOAD (kPa)

	NZS	DZ
Haybarn, Lee Flat	0.77	0.74
Covered Yard, Rongahere	0.52	0.50
Implement shed, Dacre	0.52	0.26
Clearspan Covered Yard, Cardrona	1.03	1.00
Woolshed, Lake Hayes	0.69	0.68

FACTORED LOCAL WIND PRESSURE (on roof) (kPa)

	NZS	DZ
Haybarn, Lee Flat *	1.02	2.08
Covered Yard, Rongahere	0.96	0.74
Implement Shed, Dacre	1.38	1.18
Clearspan Covered Yard, Cardrona *	0.87	1.94
Woolshed, Lake Hayes *	0.69	2.42

* In an orographic lee region

FACTORED AVERAGE WIND PRESSURE (on roof) (kPa)

	NZS	DZ
Haybarn, Lee Flat *	0.76	1.76
Covered Yard, Rongahere	0.71	0.44
Implement Shed, Dacre	0.84	1.03
Clearspan Covered Yard, Cardrona *	0.67	1.23
Woolshed, Lake Hayes *	0.46	0.86

* In an orographic lee region

The following are the major differences between NZS 4203 and 2/DZ 4203:

(a) A rather small snow coefficient for monoslope roofs in DZ 4203. Notice the reduced snow load for the implement shed.

(b) NZS 4203 has reduced wind speeds for inland locations, whereas 2/DZ 4203 increases the design wind speed due to the orographic lee multiplier. The net effect of adopting 2/DZ4203 is small around coastal Otago but increases the wind load 100 per cent in much of Central Otago. Topographic effects, which under 2/DZ4203 will apply to most farm buildings, will further increase the wind load up to 40 per cent, where the upwind slope is less than one in five. Where topographical effects are very important, the approach in Appendix A in NZS4203 is, in general, more onerous than 2/DZ4203.

(c) The internal pressure in 2/DZ 4203 is rather sensitive to the ratio of dominant openings, whereas in NZS 4203 it remains constant at +0.8. This effect significantly reduces the wind load on the covered yard and the clearspan covered yard, when designed to 2/DZ 4203.

COMPARISON OF CONSTRUCTION COSTS

Table 2: Comparison of Construction Costs

	RATIO OF MEMBER WEIGHTS (DZ/NZS)		
	Steel Frames	Purlins	Girts
Haybarn, Lee Flat	1.45	1.43	1.46
Covered Yard, Rongahere	1.00	1.00	0.67
Implement Shed, Dacre	1.00	1.00	0.93
Clearspan Covered Yard, Cardrona	1.00	1.43	1.44
Woolshed, Lake Hayes	1.16	1.43	1.60

Adoption of 2/DZ 4203 will not, in general, result in many cost savings. Savings will occur for the following two types of buildings:

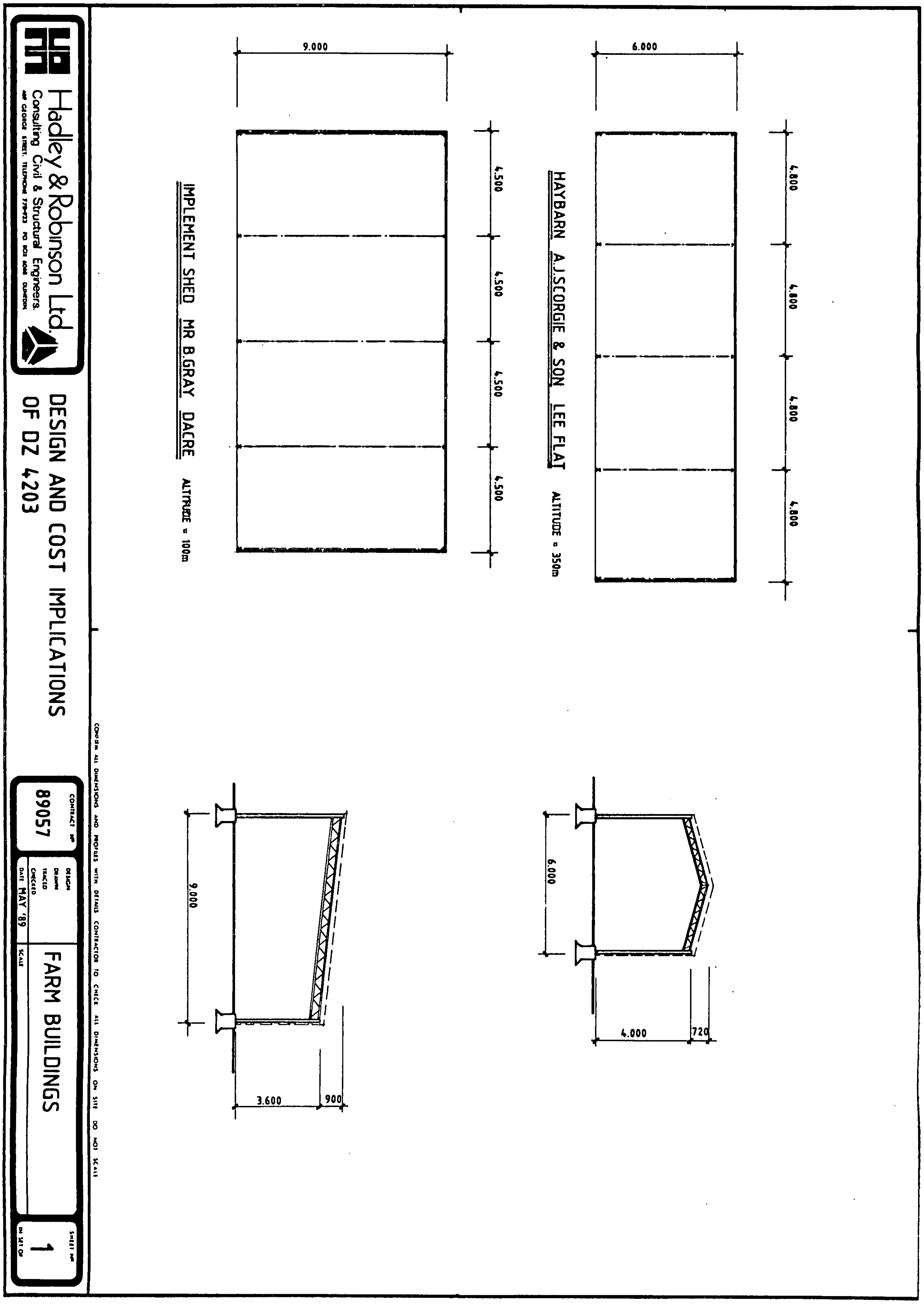
- Snow load dominated lean-to type buildings.
- Partly enclosed buildings, outside the orographic lee regions, where the ratio of dominant wall openings is such that a markedly reduced internal pressure coefficient results.

Significant cost increases can be expected in the orographic regions. The cost of the structure will reflect the increase in weights given in the above table as welding, handling, transport, erection costs etc. will all increase roughly in proporation to weight. Foundations also will increase as wind uplift usually controls their design. Cost increases can be expected up to 50 per cent of the structural components or approximately 25 per cent of the total building cost.

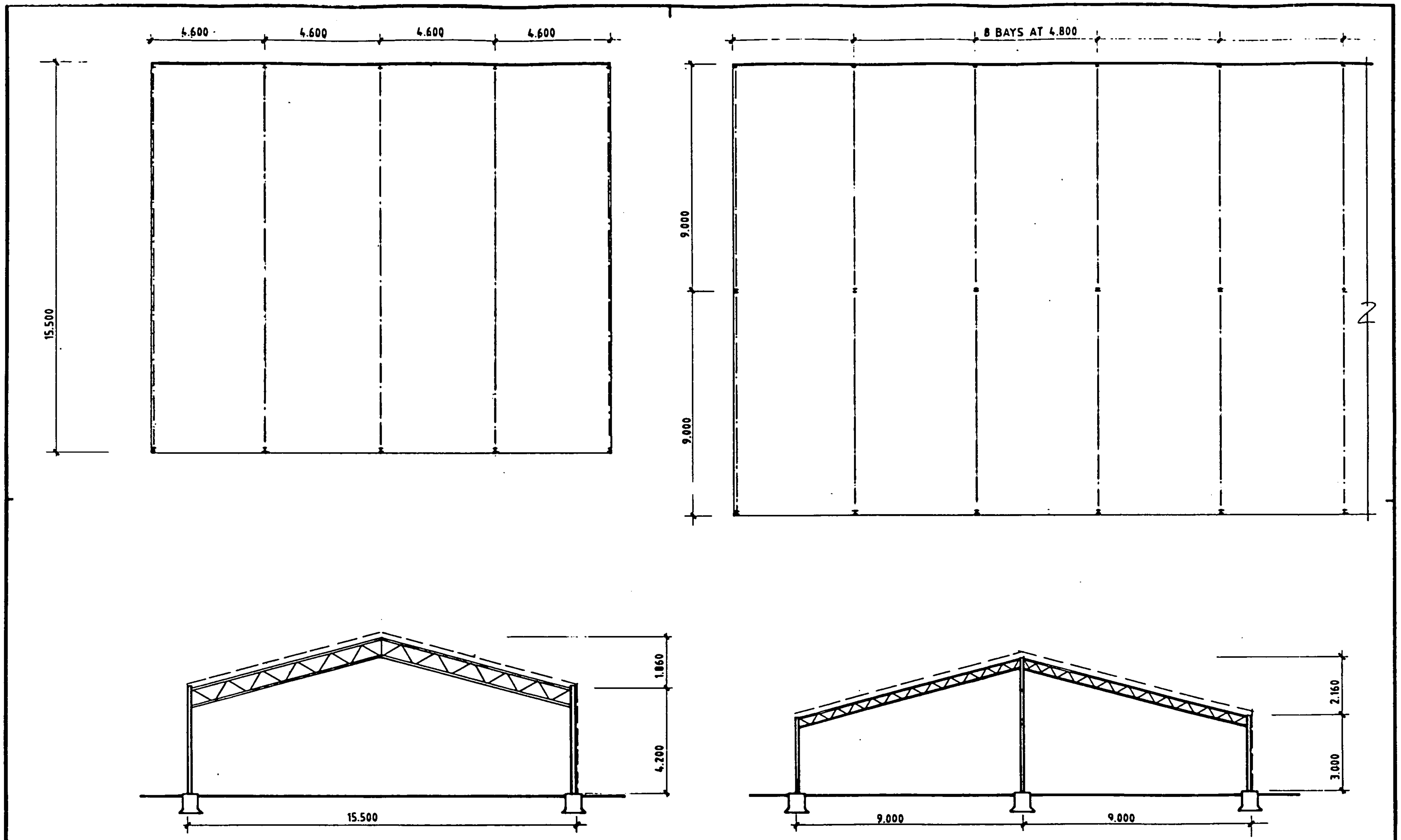
Surprisingly, no increase to the bolting requirements between the timber purlins or girts and the steel frames is required - one M12 bolt being sufficient.

DESIGN COSTS

Increase in design effort is limited to the collection of site data and the derivation of wind loads. If sufficient site data can be collected without a personal site visit, increases in design fees will be negligible. If, on the other hand, a site visit is required to properly assess the topographical, orographical and channelling multipliers then total design costs will more than double on average.



Hadley & Robinson Ltd.
Consulting Civil & Structural Engineers.
DESIGN AND COST IMPLICATIONS OF DZ 4203
FARM BUILDINGS
CONTRACT No
89057
DATE MAY '89
SHEET No
1
CONFIRM ALL DIMENSIONS AND PROFILES WITH DETAILS CONTRACTOR TO CHECK ALL DIMENSIONS ON SITE DO NOT SCALE
IMPLEMENT SHED MR B.GRAY DACRE
ALTITUDE = 100m
HAYBARN A.J.SCORGIE & SON LEE FLAT
ALTITUDE = 350m
9.000
4.500
4.500
4.500
4.500
6.000
4.800
4.800
4.800
4.800
9.000
3.600
900
6.000
4.000
720



CLEARSPAN COVERED YARD MR B.SCURR CARDRONA ALTITUDE = 500m

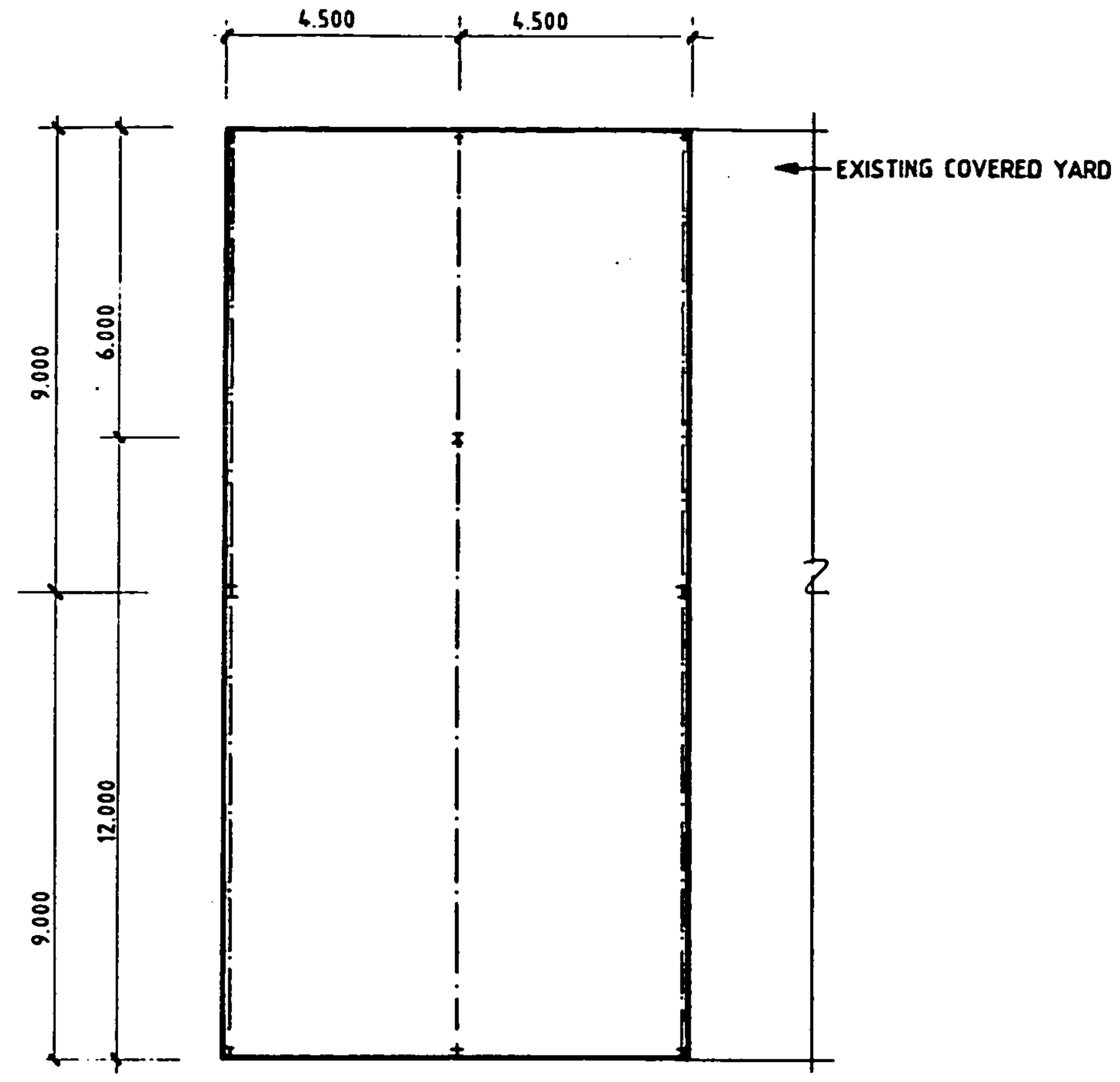
COVERED YARD C.R.ROSS & SON RONGAHERE ALTITUDE = 150m

CONFIRM ALL DIMENSIONS AND PROFILES WITH DETAILS CONTRACTOR TO CHECK ALL DIMENSIONS ON SITE DO NOT SCALE

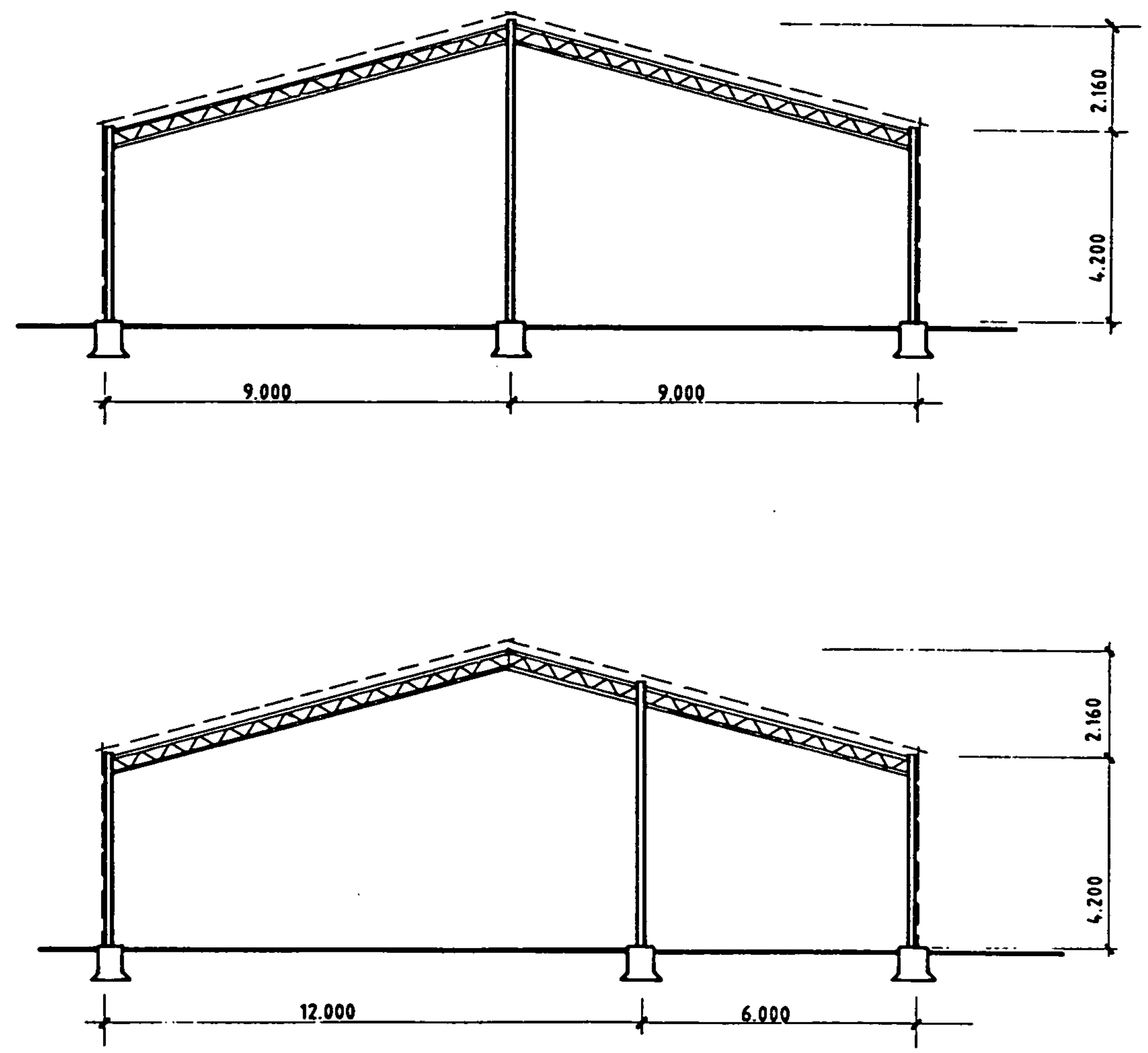
Hadley & Robinson Ltd.
Consulting Civil & Structural Engineers.

DESIGN AND COST IMPLICATIONS
OF DZ 4203

CONTRACT No 89057	DESIGN DRAWN TRACED CHECKED DATE MAY '89	FARM BUILDINGS SCALE	SHEET No 2 IN SET OF



WOOLSHED MR R.JONES LAKE HAYES ALTITUDE = 300m



CONFIRM ALL DIMENSIONS AND PROFILES WITH DETAILS CONTRACTOR TO CHECK ALL DIMENSIONS ON SITE DO NOT SCALE

Hadley & Robinson Ltd.
Consulting Civil & Structural Engineers.
449 GEORGE STREET. TELEPHONE 778-923 PO BOX 6068 DUNEDIN

DESIGN AND COST IMPLICATIONS OF DZ 4203

CONTRACT No 89057	DESIGN / DRAWN / TRACED / CHECKED / DATE MAY '89 / SCALE	FARM BUILDINGS	SHEET No 3 IN SET OF

BUILDING TYPE 4

Waiariki Polytechnic, Rotorua

Four-storey steel framed building incorporating eccentrically braced frames

Murray North Limited
PO Box 821
Auckland

BUILDING DESCRIPTION

The structure consists of four above-ground levels of approximately 35 x 27 m dimension and one basement level. The basement is of reinforced concrete, while the superstructure is of steel framed construction with composite metal trough floors. The lateral load resisting system comprises eccentrically braced frames, one on each building elevation, as shown in Figure 1. The EBFs are external to the facade and are horizontally framed back into the floor diaphragms.

In plan the structure is regular and symmetrical, a typical structural floor plan being depicted in Figure 2. The site is on the southern outskirts of Rotorua.

The underlying soils consist of many layers of volcanic ash of varying properties. The site has been preloaded and the raft foundation design allows for partial compensation of the structure weight.

SUMMARY OF DESIGN LOADS (Refer Table 1)

Gravity Loads

The minimum plant room live load given in the draft code is 50 per cent greater than the existing code.

Live load UDL for the ward areas in the draft are two-thirds of the existing code values. The strength limit state floor UDL is 1.44 times the existing code "alternative design" UDL, indicating a theoretical saving in floor beam weight of 10-20 per cent over the existing design. Other floor uses have the same live load values.

Seismic Loads

The same period was assumed as the original design, which did not take account of foundation flexibility. The nature of the structure is such that little change is expected if soil flexibility had been included, i.e., negligible translation and rotation of the massive basement box foundation.

An equivalent static lateral load analysis is applicable under both design regimes, giving a base shear coefficient of 0.13 from the existing code and 0.14 from the draft.

The live load acting during an earthquake is slightly less in the draft code (0.33 Lb compared with ψ Lb R). This gave a total seismic mass about 96 per cent of the existing code value for this particular structure.

Combining the above effects, the overall seismic lateral load is 4 per cent greater under the draft code, with a much larger force applied at the highest level since the requirement for 0.08V at the top did not apply under the existing code. The seismic overturning moment is 11 per cent greater than under the existing code.

The serviceability limit state seismic load was more severe than wind and governed the lateral movements.

Wind Loads

The much more complex wind sections in the draft code section 4.3 gave a strength limit state $q = 1.17$ kPa compared with an existing code value of 0.53 kPa. The total wind loads per floor are 4.21 kN/m and 2.88 kN/m respectively (46 per cent increase).

Wind loading was not a critical load case for the structure.

STRUCTURAL CHANGES

Lateral Load Resisting System

The member sizes in the eccentrically braced frames had to be increased by one or two member sizes, as detailed in the calculations.

This would also have a flow-on effect on the foundations which must be designed for the capacity actions of the steel superstructure, requiring approximately 13 per cent stronger foundation wall-beams.

The major constraint on achieving a more economical design was Clauses 12.11.2 (a) and (b) of Part 2 of NZS 3404:1988. The two lowest levels of active link had a similar shear demand, yet the code demanded that the lowest active link be heavier than all others. Significant shear redistribution to higher active links was also prevented.

The original design complied with the requirements of NZS 3404:1988.

Gravity Resisting System

Typical floor beam layouts, as illustrated in Figure 2, were redesigned. The strength limit state moment in the secondary beams (alphabetic grids) was 90 per cent of the value derived from the existing NZS 4203. However, the member size was governed by the unpropped construction case, for which the draft code with NZS 3404:1988 and the existing design have identical methods of calculation. The original design misinterpreted the HERA recommendations and calculated a 250UB31, when in fact a 200UB25 would have sufficed, a 20 per cent weight saving.

For primary beams (numeric grids) the draft code strength limit state moment is 44 per cent greater than the "alternative method" moment in the existing NZS 4203, or 94 per cent of the strength method of that document. The resulting reduction in member size is from a 530UB82 to 460UB67, an 18 per cent weight saving.

Serviceability limits were not critical in any case examined: typically deflections were half the suggested limits even with the reduced member sizes.

COST IMPLICATIONS OF DZ 4203

Building Cost

The approximately value of the whole building is $7 million.

Our estimate of the cost effects of the draft code are as follows:

Item		$ Increase	$ Reduction
Eccentrically braced frames			
Grids A & H + 2.9t) @ $3,500	=	20,300	
Grids 1 & 8 + 2.9t)			
Foundations			
+28 m^3 concrete @ $250	=	7,000	
+28 m^3 excavation @ $35	=	1,000	
Floor Gravity Beams			
-9.6t @ $2,700	=		-25,920
		28,300	-25,920
		-25,920	
Net Increase		$2,380	

This represents approximately 0.03 per cent of the total building cost, or say 0.1 per cent of the structure cost.

We conclude that use of DZ 4203 would have had negligible cost effects on this structure, although effects on the elements were significant but tended to cancel.

Design Cost

Our impression of the relative design effort required by the draft code compared with the existing code is that the draft would require about 5 per cent more work because of the more numerous line load levels, serviceability calculations and much more time consuming wind pressure derivation.

If the original design cost was 6 per cent of the structural cost, which in turn was 25 per cent of the total cost, the increased design effort would work out to be 0.08 per cent of the total building cost, or about $5,000.

TABLE 1 - COMPARISON OF KEY CODE DIFFERENCES AFFECTING WAIARIKI POLYTECHNIC BUILDING

ITEM	EXISTING NZ 4203 REFERENCE	VALUE	DRAFT DZ 4203 REFERENCE	VALUE
Plant Room Live Load	Table 2, item 3.17	5 kPa	Table 2.3.1, item 9.4	7.5 kPa
Ward Room Live Load	Table 2, item 4.2	3.0 kPa	Table 2.3.1, item 4.1	2.0 kPa
Seismic Weight Levels 2 - 6	Section 2.4	16334 kN	Section 1.6.4	15670 kN
Seismic Base Shear above Level 2	Section 3.4.2	2107 kN	Section 3.7.2	2194 kN (Severe LS)
Overturning moment at level 2		21493 kNm		23881 kNm
Wind Speed	Section 4.3.2	36 m/s	Section 4.3.2.2.1	50 m/s
Dynamic pressure	Section 4.4	0.53	Section 4.3.3.	1.17
load per floor, per metre (strength LS)		2.88kN		4.21 kN

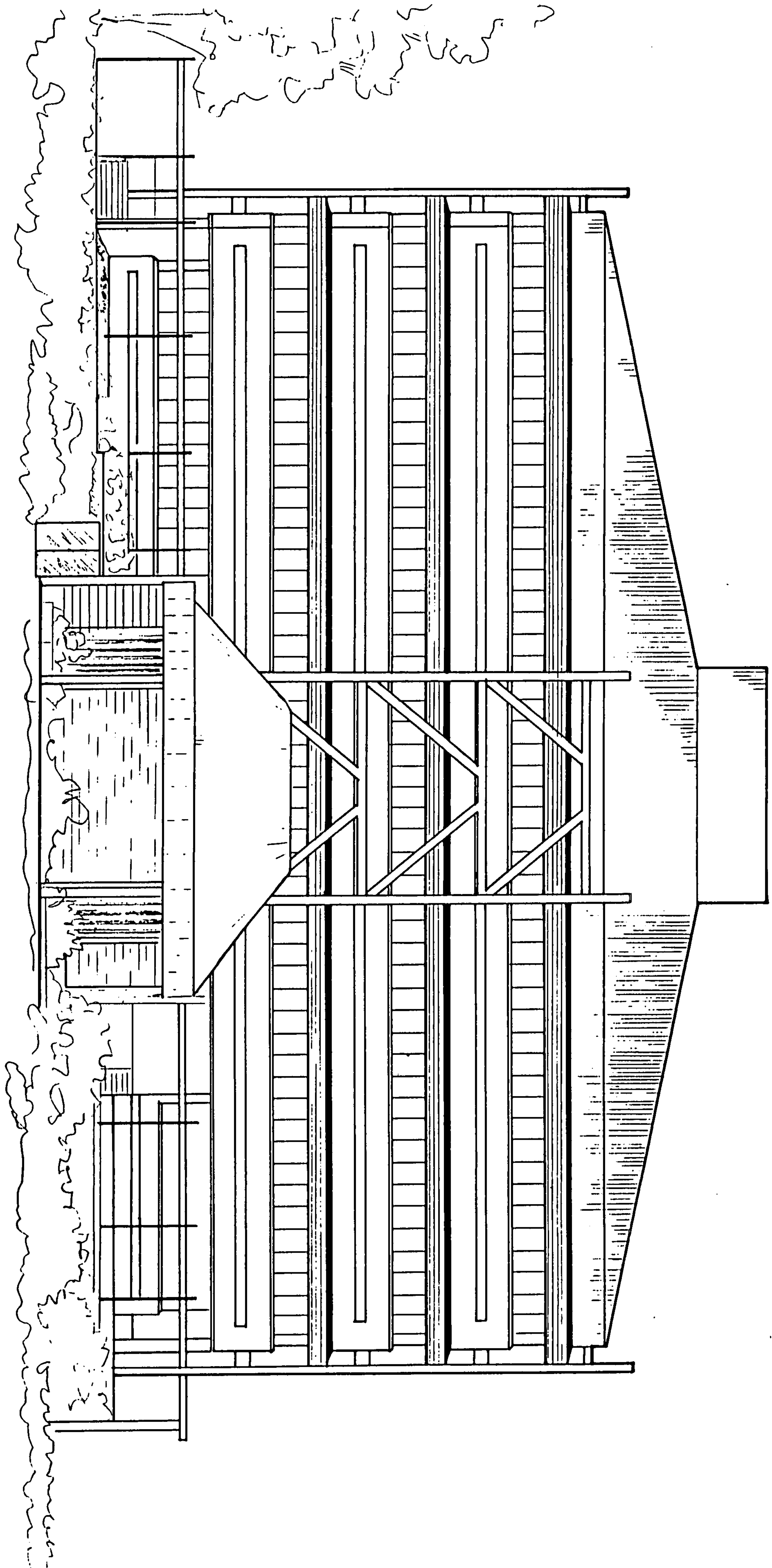


FIGURE 1 BUILDING ELEVATION

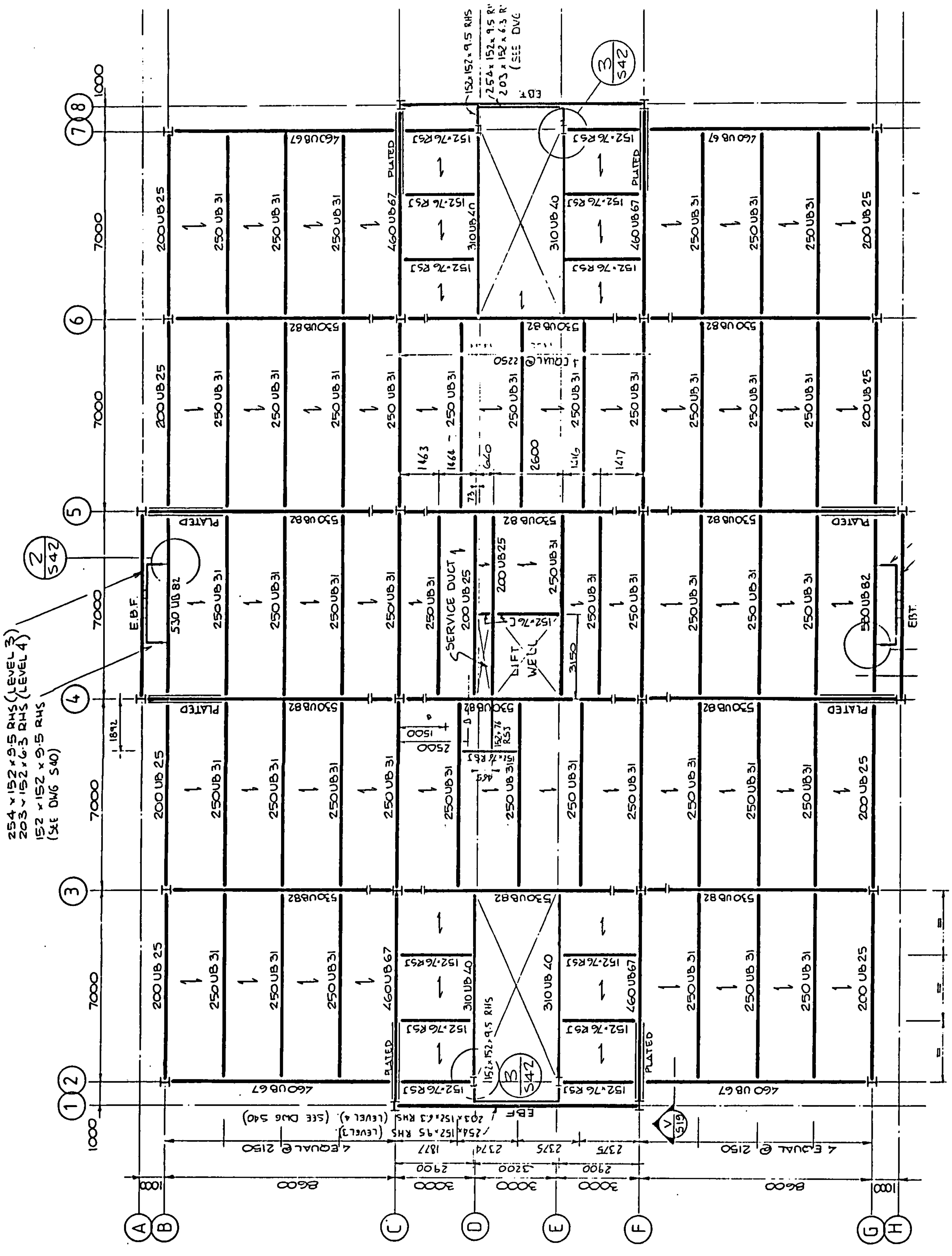


FIGURE 2 TYPICAL FLOOR PLAN

BUILDING TYPE 5

F.D.C. House, Auckland

Five-storey steel framed office building incorporating eccentrically braced frames above Level 1 with reinforced concrete shear walls to the lower level

KRTA Ltd
PO BOX 9806
AUCKLAND

BUILDING DESCRIPTION

The building to which this report applies is a four storey office building for Fletcher Development & Construction Limited located on Great South Road, Penrose. The building is approximately 50m x 30m in plan with a podium at Level 1 totalling approximately 1900 square metres. Ground and first floor levels are for car-parking while levels 2, 3 and 4 comprise general office space. The building is clad with a curtain wall glazing system.

The building framing system is steel and the building is founded primarily on rock. From ground to level 1, the structure consists of four concrete shearwalls and between levels 1 and 4 are EBF's. The two north/south frames are located in the external walls, while the east/west frames are located in the north wall and the core. The columns service gravity loads for the full height of the building, and are concrete encased below level 3. The roof (level 5) is supported from level 4 by cantilever columns. The flooring is 0.75mm Bondek flooring 120mm thick supported by composite steel gravity beams.

ANALYSIS AND DESIGN PROCEDURE

The steel eccentrically braced frame (EBF) was analysed assuming effective ground level at level 1, being the top of the shearwalls. This was due to the relatively stiff first level comprising shearwall construction. Allowance was made for the cantilever action of all level 4 columns.

The original design preceded the issue of DZ 3404 but was carried out in accordance with NZS 4203:1984 and the latest Hera recommendations available at the time. Also used in the design was the December 1985 Bulletin of the New Zealand National Society for Earthquake Engineering, Vol. 18 No. 4. The calculations assumed a material reduction factor of 1.0.

The latter redesign to DZ 4203 included the new recommended material reduction factors supplied by Hera. These reduction factors of 0.85 and 0.90 for columns and beams respectively were provided to us as part of our brief, in order to align DZ 3404 with the limit state design method of DZ 4203, for the purpose of this report. Generally the design procedure followed was similar to the original design but was based on the revised loads as defined by the new draft code.

Analysis

General

The designer's time input required for analysis by the new draft has increased substantially since more design factors are incorporated in the procedure. This was most readily evident when one considered that the volume of calculation to assess wind loading doubled using DZ 4203.

Gravity Loading

The reduction of load factors for dead and live loads to 1.2 and 1.6 respectively in DZ 4203 are offset by the introduction of material strength reduction factors.

In the case of beams the loadings are only marginally increased. However the strength reduction factor of 0.85 for columns causes an effective increase in the design loads for these elements.

Seismic Loading

The new seismic coefficient was reduced to 67 per cent of that required for an EBF by earlier considerations. This was not a reflection of the change of codes but rather, the conservative nature of the information available at the time of the original EBF design.

Clause 3.5.2.1 of DZ 4203 required that a building be less than 15m high, and have a period of less than 0.4 seconds, for the equivalent static method of analysis to be used. The building which was the subject of this report complied with the height restriction as level 1 is assumed to be ground level for analysis purposes. We understand from Richard Fenwick of the SANZ Loading Code Revision Committee, that this clause should read either less than 15m high, or a period less than 0.4 seconds, and thus we have used the equivalent static method of analysis, even though the building period exceeds 0.4 seconds. We note that the original design to NZS 4203 was by the equivalent static force method of analysis.

As the P-delta effects are a function of the structural deflection, the member sizes need to be fairly accurately assessed prior to initial analysis. As a minimum, three iterations were required to allow for P-delta effects.

Initially, the equivalent static force analysis was performed to obtain the initial deflection. Secondly, an analysis was required to find the effects of the induced loads due to the P-delta effect at the initial deflection. After completion of the second analysis, which has been used to obtain the final deflection due to seismic shear and P-delta loads, a third analysis was required which involved applying the resulting loads to the structure, thus giving member actions. This three step process replaced the single step process of NZS 4203. The extra iterations required 50 per cent extra time on that required for the seismic analysis under NZS 4203.

Even though the base shear has been reduced, the inclusion of P-delta induced loads has increased the loads on the EBF frames, as reflected by their increased size. The increase was approximately 11 per cent by steel tonnage (Refer **Cost Implications**).

Wind Loading

The base shear wind loads were more than double those derived from NZS 4203, primarily due to the increase in basic wind speeds. For the structure, which was the subject of this report, the seismic loading considerations outweighed those of wind loading, and was not considered critical. This would have a marked impact on the design of non-structural components such as cladding and glazing.

Serviceability

DZ 4203 defines new loading limit state to account for serviceability. This is in contrast to the use of K/SM factors utilised in NZS 4203 and the analysis time is thus increased slightly.

Structural Components

Components of the structure have been influenced differently by the revision of the code. The following are a few observations:

Beams

These were effectively unchanged apart from the effects of the code recommendations for deflections. The design was originally to the strength method as these were composite beams. The new code comprising limit state design yielded similar results to the original design.

As the floor in the finished building has been described as 'lively' it seems to suggest that the recommendations of both codes on deflections are unconservative when compared to human perception of movement.

Columns

Both the design method and the loading changed since the alternate method used under NZS 4203 for gravity columns was no longer permissible under DZ 4203. This resulted in larger gravity columns, which was also in part due to the increased live load reduction factor.

EBFs

The original design procedure was followed for the revised loads.

The resulting sizes reflected the change in applied loads (i.e., generally larger).

Foundations

In accordance with clause 3.4.1 of DZ 4203, the foundations have been considered as part of the structure for design. Both the original design and the design to DZ 4203 were capacity designed for the overstrength of the links. The limit of SM=2 from NZS4203 was not reached.

The increase in load to the foundations is 32 per cent which is a result of the increase in seismic loading due to the P-delta effects.

Non-Structural Components

For a building such as an EBF with a high ductility demand, the non-structural elements will require larger separations or a greater degree of movement by DZ 4203 than previously required by NZS 4203.

We note, that although a specific design was not executed for cladding/glazing, the new wind pressures were observed to be more than double those of NZS 4203.

COST IMPLICATIONS

For the building as a whole designed to DZ 4203, there was approximately a 7 per cent increase in the value of structural steel.

The original design to NZS 4203 included a 3.0 kPa floor loading even though a 2.5 kPa loading could have been used. If the value of 2.5 kPa had been used then the new code would have required that it be changed to 3.0 kPa and would have resulted in an extra increase of 5 per cent in the value of structural steel. However we feel that the value of 2.5 kPa was never appropriate and had designed the building accordingly. A breakdown of the major elements is as follows:

Beams

The serviceability recommendations of DZ 4203 require the beams to be precambered. The beams account for 70 per cent of the steel tonnage in the structure. We envisage precambering costs at less than 3 per cent of the cost of manufacture of the beams.

Columns

The tonnage of steel required increased by 25 per cent, due partly to loading considerations, and partly to the material strength reduction factor of 0.85.

EBF

The original EBF design was to the strength method and very little change occurred to the column loads as a result of DZ 4203. The braces had a slight increase in size (5 per cent average) due to the P-delta effects. The requirement of providing for a ductility demand of 6 resulted in a substantial (30 per cent) increase in beam active link sizes to accommodate the stresses induced by these large inelastic deformations. The increase in steel weight is 11 per cent per EBF.

Foundations

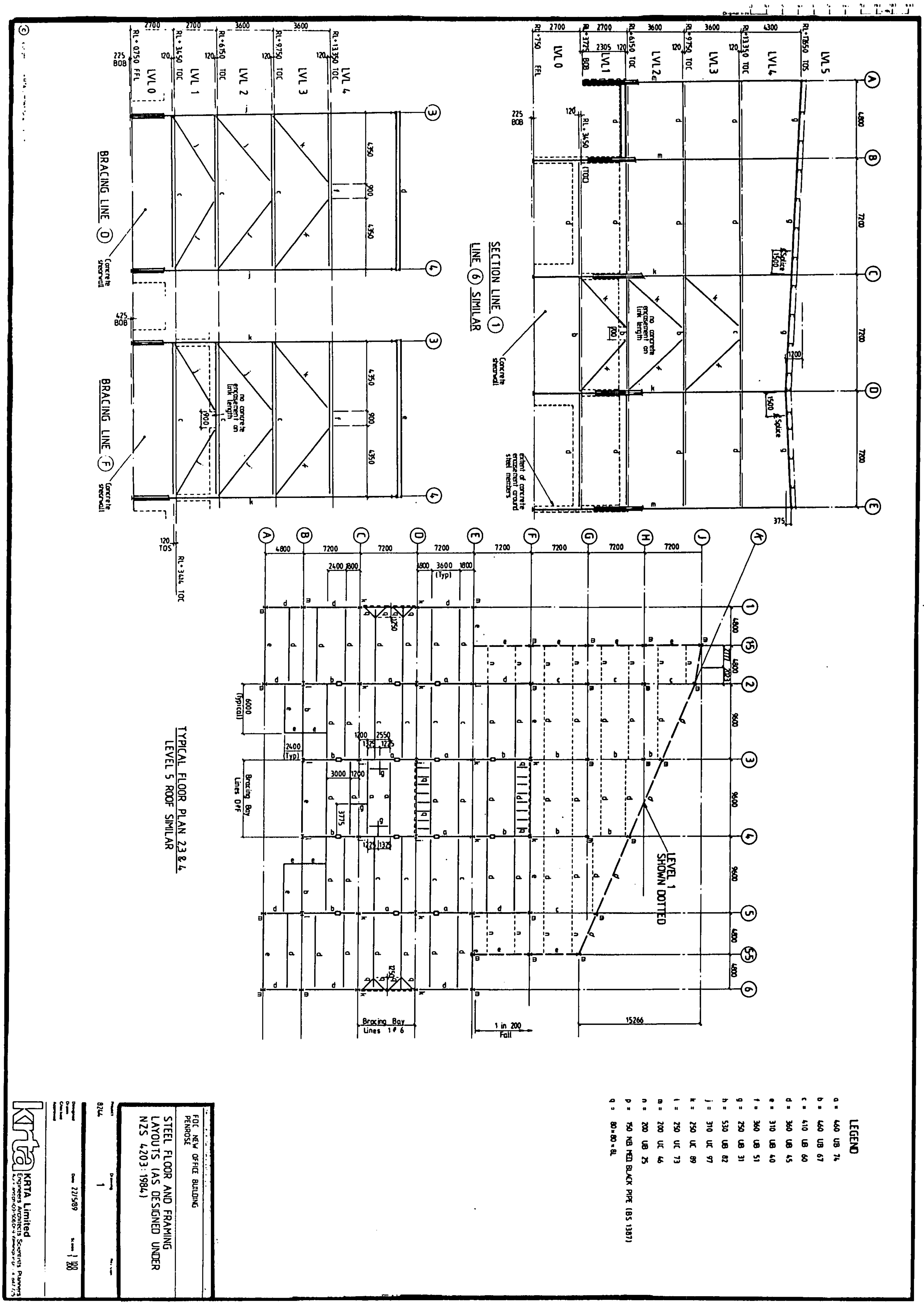
The 32 per cent increase in foundation loading due to seismic considerations results in some increase, although smaller, in foundation costs. This combined with the increase in gravity pad sizes results in an overall increase in foundation costs of 20 per cent.

Total Building Costs

Of a total building cost estimated at $5,500,000, the structural steel content for the building, designed under NZS 4203, was about $480,000. The cost of steel work as designed under DZ 4203 was $515,000, an increase of $35,000 (7.3 per cent). The increase in foundation sizes accounted for a further cost increase of $50,000 (20 per cent). Thus, for the structure as a whole, the cost increase was $85,000. Assuming the cost of the structure alone to be 35 per cent of the total building cost, this equates to approximately a 4.4 per cent increase in the structural costs and a 1.5 per cent increase in the total building costs.

Design Costs

Assuming an engagement on a percentage fee basis, there would be a proportional increase in design fees of 1.5 per cent. We are of the opinion that this fee increase would be insufficient to fully cover the extra costs involved in design and draughting. The increase in design time would be approximately 10 per cent primarily due to increase in analysis time.


SECTION LINE 1
LINE 6 SIMILAR
BRACING LINE D
BRACING LINE F
TYPICAL FLOOR PLAN 2,3 & 4
LEVEL 5 ROOF SIMILAR
LEVEL 1 SHOWN DOTTED
LEGEND
a = 460 UB 74
b = 460 UB 67
c = 410 UB 60
d = 360 UB 45
e = 310 UB 40
f = 360 UB 51
g = 250 UB 31
h = 530 UB 82
j = 310 UC 97
k = 250 UC 89
l = 250 UC 73
m = 200 UC 46
n = 200 UB 25
p = 750 NB MED BLACK PIPE (BS 1387)
q = 80 x 80 x 8L
FDC NEW OFFICE BUILDING
PENROSE
STEEL FLOOR AND FRAMING
LAYOUTS (AS DESIGNED UNDER
NZS 4203:1984)
8244
1
22/5/89
1:100
1:200
KRTA Limited
Engineers Architects Scientists Planners

BUILDING TYPE 6

Electric Power Transmission Steel

Lattice Tower, 20m high

Works Technical Services
Special Projects Office
Works & Development Services
Corporation (NZ) Ltd
PO Box 12003
Wellington

INTRODUCTION

A 20 m high steel lattice tower was designed using loading codes NZS 4203:1984 and 2/DZ 4203:1989. For the particular site considered, the implementation of the revised loading code 2/DZ 4203:1989 would result in a structure cost increase of 16.3 per cent over the existing loading code NZS 4203:1984.

NOMENCLATURE

S1 Topography factor

S2 Ground roughness, building size and height above ground factor

θ Wind direction being considered

$M_{s,\theta}$ A shielding multiplier in direction θ

$M_{c,\theta}$ A channelling multiplier in direction θ

$M_{o,\theta}$ An orographic multiplier in direction θ

$M_{t,\theta}$ A topographic multiplier for gust wind speeds in direction θ

Me An elevation multiplier

Mi A structure importance multiplier

VZ, The design gust wind speed in the direction θ at height Z, m/s

Vu The basic wind speed for ultimate strength limit state

Cf Force coefficient (NZS 4203:1984)

Cd Drag force coefficient for member in the wind stream (2/DZ 4203:1989)

STRUCTURE DESCRIPTION

An unnamed EPT (Electric Power Transmission design) steel lattice tower was chosen for the evaluation of the revised loading code 2/DZ 4203:1989.

The tower was built in 1973 to a height of 20 m. Its function is to support microwave antenna as part of a communications network.

The tower is located at an altitude of 945 m above sea level in mountainous terrain. An 8.5 m x 8.5 m x 1.2 m concrete gravity slab supports the tower which is bolted down with cast-in-place holding down bolts.

DESIGN WIND LOADING

A computer model was set up to carry out a three-dimensional analysis to evaluate forces in the structural members. This model was used to calculate forces in the orthogonal and diagonal directions.

Site Wind Speed

NZS 4203:1984

The calculation of a design wind speed for this site using NZS 4203 is difficult. Basic wind speeds are given for the region, but S1 values which allow for site topography were obtained from the Meteorological Service.

The following wind speeds were calculated over the height of the tower. A ground roughness of two and a building of class B were assumed. See Table 1 below.

TABLE 1: DESIGN WIND SPEEDS FROM NZS 4203:1984

Panel	Height (m)	Design Wind Speed (m/s)
1	1.7	48
2	4.4	52
3	6.9	55
4	9.0	62
5	11.0	63
6	13.5	67
7	16.6	68
8	19.0	71

2/DZ 4203:1989

The microwave tower was analysed using the DETAILED PROCEDURE: STATIC ANALYSIS of 2/DZ 4203:1989 for non-wind sensitive structures. Table 2 below sets out wind multipliers and resulting design wind speeds for each of these directions. Multipliers $M_{s,\theta}$, $M_{c,\theta}$ and $M_{o,\theta}$ were taken to be 1.0 as they do not apply to this site.

TABLE 2: BASIC DESIGN WIND SPEEDS FROM 2/DZ 4203:1989

θ	Vu (m/s)	Mt,θ	Me	Mi	VZ, (m/s)
30	43	1.32	1.14	1.1	71
75	40	1.27	1.14	1.1	64
120	38	1.54	1.14	1.1	73
165	41	1.54	1.14	1.1	79
210	45	1.20	1.14	1.1	68
255	49	1.27	1.14	1.1	78
300	48	1.38	1.14	1.1	83
345	46	1.32	1.14	1.1	76

Due to the symmetrical nature of the tower, the worst orthogonal and diagonal wind speeds were taken to give the worst tower forces.

The worst orthogonal and diagonal wind speeds over the height of the tower are tabulated in Table 3.

TABLE 3: DESIGN WIND SPEEDS FROM 2/DZ 4203:1989

Panel	Height (m)	Design Wind Speed (m/s) (orthogonal)	Design Wind Speed (m/s) (diagonal)
1	1.7	67	74
2	4.4	70	77
3	6.9	74	81
4	9.0	77	85
5	11.0	80	88
6	13.5	82	90
7	16.6	84	92
8	19.0	85	94

FORCES ON STRUCTURAL ELEMENTS

Ice Loading

As the tower is known to be heavily loaded with ice during the winter months, both the iced and non-iced wind loadings must be considered. NZS 4203:1984 contains no recommendations for tower ice load considerations so the Meteorological service was consulted. The Meteorological Service suggested that ice thickness of 800 mm on the windward member face and 120 mm thicknesses on the other member faces be considered. This would be in conjunction with the one-in-five year return period wind gust when calculating the wind forces. This differs substantially from ice loadings suggested by the new loadings code DZ 4203:1989, which recommends a maximum ice thickness of 200 mm on the windward member face and 30 mm on other member faces. The NZS 4203:1984 recommendation is in conjunction with the same five year return period wind gust. Such a discrepancy for iced-member wind loading makes comparison between the two codes impossible in the ice condition. It is considered that the new code 2/DZ 4203 does not truly reflect possible ice conditions in the area of New Zealand considered.

Wind Drag Coefficients

NZS 4203:1984

Drag coefficients are dependent on structural members which form the tower and ancillary members which pass up the inside of the tower. Results for the orthogonal direction are summarised in Table 4 below.

TABLE 4: FORCES FROM NZS 4203:1984

Panel	Non-Iced Structural Member Area (m2)	Cf	Non-Iced Ancillary Member Area (m2)	Cf	Force on Ancillary and Structural Members (kN)
1	2.6	3.0	0.4	1.8	11.9
2	2.4	2.9	2.6	1.8	19.7
3	2.1	2.8	2.4	1.8	19.4
4	1.4	3.0	1.6	1.8	18.4
5	1.6	3.0	2.3	1.8	21.4
6	2.0	3.1	2.2	1.8	30.2
7	1.3	3.4	1.9	1.8	21.8
8	1.5	7.8	0.8	1.8	18.7

2/DZ 4203:1989

The drag coefficients for the lattice tower members were derived using loading 2/DZ 4203:1989 in the same manner as results derived using NZS 4203:1984. The results using 2/DZ 4203:1989 are tabulated below. Drag coefficients are noted to be lower using 2/DZ 4203:1989 than for NZS 4203:1984 for the same ratio of structural steel to panel area.

TABLE 5: FORCES FROM 2/DZ 4203:1989

Panel	Non-Iced Structural Member Area (m2)	Cd	Non-Iced Ancillary Member Area (m2)	Cd	Force on Ancillary and Structural Members (kN)
1	2.6	2.6	0.4	1.4	19.6
2	2.4	2.6	2.6	1.4	29.1
3	2.1	2.5	2.4	1.4	28.3
4	1.4	2.6	1.6	1.4	21.0
5	1.6	2.6	2.3	1.4	26.3
6	2.0	2.7	2.2	1.4	34.3
7	1.3	2.9	1.9	1.4	27.1
8	1.5	2.5	0.8	1.4	21.2

Seismic Design

Seismic loading was not the governing load case for the tower design using NZS 4203:1984. It is also not critical for 2/DZ 4203:1989.

DESIGN OF TOWER MEMBERS

Computer Analysis

Forces on lattice members and antenna, derived from NZS 4203:1984 and 2/DZ 4203:1989 were input into two identical computer models of the tower.

Antenna forces were calculated using a computer program called 'SIFTER ANALYSIS'. The SIFTER program calculates wind loadings due to groups of antenna at various levels on a tower. Antenna wind loads then are summed in selected directions and critical loads determined. The SIFTER program was run using wind speeds derived from both 2/DZ 4203:1989 and NZS 4203:1984.

Member Design

Tower leg members and tower web members were designed using forces derived from loading codes 2/DZ 4203:1989 and NZS 4203:1984. An actual member size (from member tables) and a theoretical member size were calculated. The theoretical member size was used to calculate the percentage increase in structure cost.

Members were designed assuming forces calculated from the non-iced case. Reasons for this have been discussed previously (Refer **FORCES ON STRUCTURAL ELEMENTS**, Ice Loadings).

The ECCS (European Convention for Construction Steelwork) recommendations have been followed in designing the tower angle members.

For members of low slenderness such as tower legs, the ECCS method will give loads approximately 10 per cent higher than those given by AS 1250 with 1.33 increase for wind loading applied.

SERVICEABILITY OF THE TOWER

The serviceability of microwave towers is dependent on the number and type of antenna being supported. It is considered that the serviceability assessment of a microwave tower is a specialised area and is not covered by the code. 2/DZ 4203:1989 was therefore not evaluated for this consideration.

WIND SENSITIVE STRUCTURES

The wind sensitive section of 2/DZ 4203:1989 did not apply as the first mode period was less than three seconds.

COST IMPLICATIONS OF IMPLEMENTING 2/DZ 4203:1989

Increase in Structure Cost

It was calculated that the use of 2/DZ 4203:1989 would increase the cost of the structure and foundation by approximately 16.3 per cent. This cost increase is due mainly to the higher site wind speed derived from the new loadings code. If both codes recommended the same site wind speed, the reduction in load factors for wind from 1.3 to 1.0 using 2/DZ 4203:1989 would result in a decreased overall cost of the structure. As the lattice tower is a relatively light structure, the increased dead load factor from 1.0 to 1.2 for the 2/DZ 4203:1989 would have little effect on final member sizes. The structural members chosen for this costing exercise are considered typical of all members in the tower. Results are summarised in Table 6 below.

TABLE 6: COMPARISON OF WEIGHTS

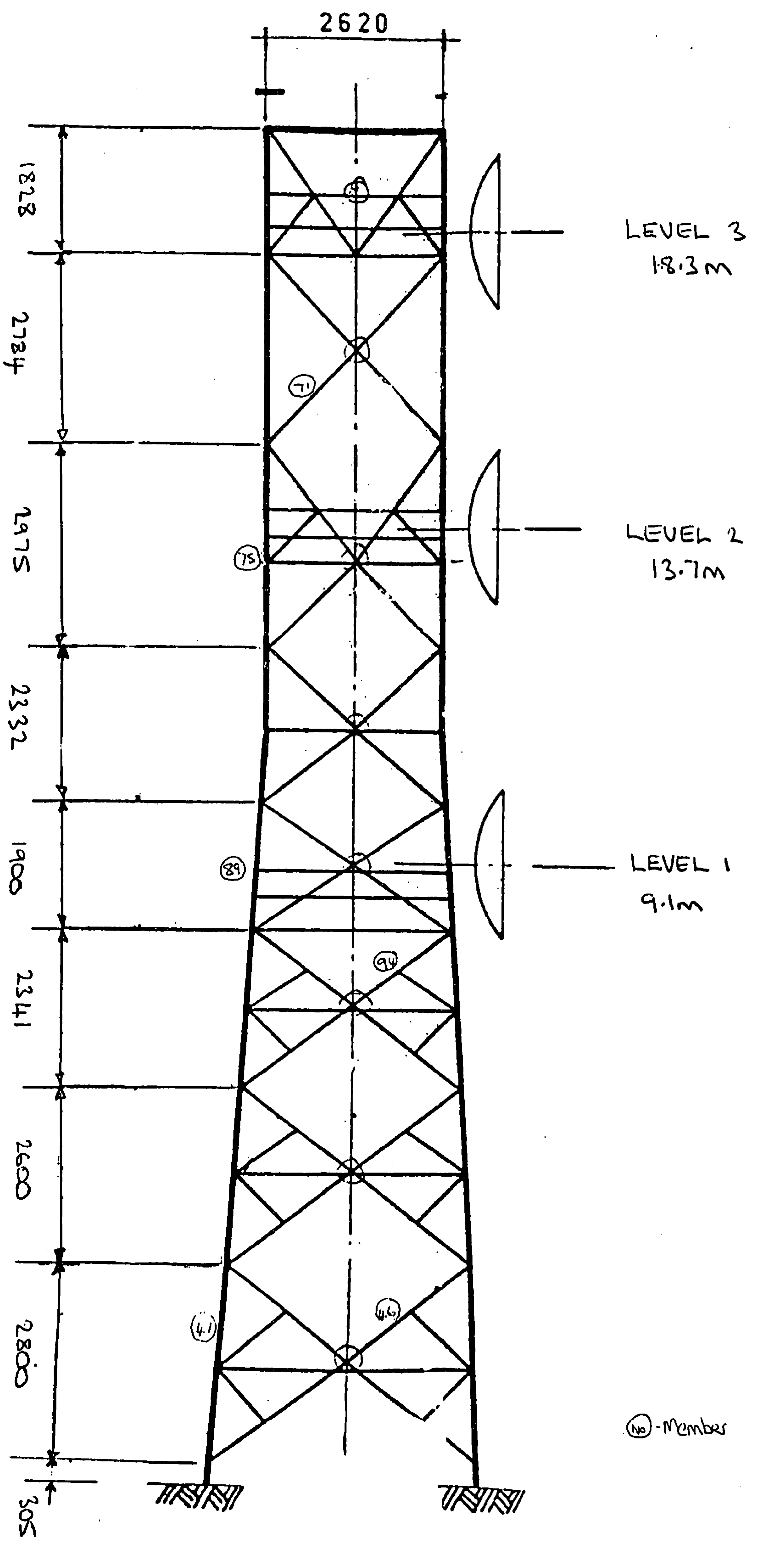
Member	Angle Member Area* NZS 4293:1984 (x 10-4 m2)	Angle Member Area* 2/DZ 4203:1989 (x 10-4 m2)	Contribution to Total Tower Percentage Increase in Steel Volume
47	55.3	61.6	3.4
89	31.2	35.5	1.3
75	16.4	18.7	1.3
46	9.1	19.5	
94	10.0	13.1	10.3+
71	6.4	7.8	

* Theoretical steel area.
\+ Contribution to total tower percentage increase in steel volume by web members (represented by web members 46, 94, 71)

Increase in Design Costs

If 2/DZ4203:1989 is implemented, the design cost for the tower would be approximately 5.5 per cent of the original tower cost. This percentage would be 5.0 per cent using NZS 4203:1984

Therefore an increase of 10 per cent in the tower design cost would be expected using 2/DZ4203:1989.



ELEVATION OF LATTICE TOWER

BUILDING TYPE 7

30 Bed Barracks Block Unit

Two-storey reinforced concrete building incorporating moment resisting H frames of limited ductility in its transverse direction and elastically responding shear walls with cantilever columns above, in its longitudinal direction

Works Consultancy Services
Wellington Technical & Consultancy Services Branch
P.O. Box 12-003
Wellington

INTRODUCTION

The building evaluated in this report is a 30 bed barracks unit designed as a standard bedroom block suitable for various locations. Approximately 20 of these units have been built to date. The original design was based on a flat, open site situated in seismic zone A (NZS 4203), with the buildings founded on dense gravels.

BUILDING DESCRIPTION

Each barracks unit is approximately 27 m x 9 m in plan, by two stories high (see Figure 1). Each structure consists of 10 precast concrete H-frames resisting gravity loads and lateral loads in the transverse direction. Longitudinal lateral loads are resisted by the cantilevering columns of the H-frames above first floor and by singly reinforced in-situ concrete shear walls below first floor. The H-frames were designed for "limited ductility" seismic loads in the transverse direction, while the walls and cantilevering columns were designed to remain elastic under design earthquake loading in the longitudinal direction.

Floor construction is 100 mm thick in-situ slab on grade at ground floor and 100 mm thick precast concrete slabs without topping, spanning between the H-frames at first floor level. In-situ concrete joints are provided between slab units to provide diaphragm continuity. Drag bars are also provided in these joints in the longitudinal direction to transfer inertia loads back to the in-situ walls.

The roof structure consists of lightweight timber trusses which were designed by the truss manufacturer. The design of these is not covered in this report.

The basic geometry and structural layout of the frames and walls were largely determined by architectural considerations and particularly by the bedroom layout. However, the design brief also required that the interior partitions be capable of being rapidly stripped out to provide large open floor spaces in emergency situations.

Economy in construction was achieved by minimising use of in-situ concrete in the superstructure and by using comparatively small precast units with a high degree of repetition. All 10 precast frames in each block are virtually identical. The total structural content of each block was approximately $120,000 (1985), which was about 12 per cent of the total cost.

COMPARISON OF DESIGNS TO NZS 4203 AND 2/DZ 4203

General

Only the primary concrete structural elements are considered in this design evaluation. The critical elements considered are the first floor slabs, the transverse H-frames and the elements resisting the longitudinal lateral loads. Each of these is considered separately in the following subsections.

However, several general aspects relating to this design evaluation are noted below first:

In redesigning the structure to the draft code, the original basic geometry and layout were adhered to, as these were principally architecturally determined.

Only changes resulting from differences in the respective code requirements are included in this evaluation. Redesign was carried out to the same basic strategies as the original design, e.g. components originally designed for limited ductile seismic loads were redesigned to the limited ductility provisions of the draft code.

Similarly, there was no attempt to take advantage of the redesign situation to improve the efficiency of the structure as this would have distorted the impact of the draft code.

The draft code explicitly identifies three limit states: serviceability, strength and severe seismic. All serviceability load combinations include a load component "I" which is required to be considered. This component includes all internal self-equilibrating loads induced by creep, shrinkage, differential settlements, and thermal effects, etc. It is not clear exactly what loads the code intends should be considered for this structure. For the purpose of this project, it has been assumed these effects are not significant for this structure and accordingly "I" was taken as zero in all load combinations. Part of the reason for this was that the structures are largely fabricated from precast components, and hence loads induced by shrinkage should be minimised. Also the structures were designed to be used in a location with excellent foundation materials (dense gravel). However, it is recognised that load effects of this type will always exist, however small, and that by ignoring them an "unconservative" bias is introduced. There may need to be more explicit guidelines to clarify what does or does not need to be considered in particular situations if widely differing interpretations are to be avoided. There is also likely to be wide variability in the magnitudes of loads predicted for some of these effects unless more definitive guidance is available.

For all components evaluated, seismic loads governed the lateral load design. This applied for both serviceability and ultimate strength criteria. The maximum wind loads were generally less than half the corresponding design seismic loads.

Overall, the differences introduced by the draft code were minimal for this structure. Indeed, the only differences significant enough to warrant modification to the original design, were as a result of a 10 per cent reduction in the seismic load in the longitudinal direction (severe seismic limit state).

Both the original design and redesign (DZ 4203) of the barracks blocks for seismic loading were performed by the equivalent static method.

First Floor Slab

All precast slab units are 100 mm thick and span 3 m centreline to centreline between the H-frames. The frame spacing was set by the bedroom layout and was not considered a variable in this redesign. Also, the 100 mm slab thickness was considered the minimum practical in this situation. The slabs were centrally reinforced.

Serviceability Limit State:

Load combination (1) of the draft code applies, i.e.

D & Ls & I (I = 0 for this case)

The code requires a short term live load of Ls = 0.7 x 2 kPa x R which is very similar to the original design live load (NZS 4203) of 1.5 kPa x R. (Long term live load Ls = 0.4 x 2 kPa x R.)

Four serviceability criteria were considered:

- yield strength (My = 0.75 Mi)
- cracking
- deflection (visual)
- vibrations (deflection under 1 kN load)

The maximum calculated moment under short term loading was approximately 0.95 My or 0.78 Mcrack. Because of the margin between the cracking moment and the maximum short term moment, it was considered realistic to use uncracked section properties for the deflection calculations (except that a crack at each support joint was allowed for by assuming the slabs were simply supported). As a result, the deflections calculated for the latter two serviceability criteria above were well within the limits recommended in the draft code.

Strength Limit State:

Load combinations (1) and (2) apply, i.e.

(1) 1.4D

(2) 1.2D & 1.6L (c.f. 1.4D + 1.7L in NZS 4203)
Combination (2) was critical. Despite the reduction in load factors as compared to NZS 4203, the resulting strength limit state design load was very similar to that used in the original design, as the basic live load required to be considered has increased from 1.5 kPa to 2 kPa.

It was not clear what criteria should be used in assessing these loads. That is, limit moment redistribution to 30 per cent or allow full mechanism to develop, but with a check on plastic hinge rotations (no longer hand design?).

For the reinforcement provided in the original design, the maximum redistribution required was only 15 per cent which, purely on the strength limit state criteria, is probably slightly more conservative than necessary.

Overall, the critical criteria for design of the slabs to DZ 4203 was the serviceability limit state yield criterion, i.e. maximum moment 0.95 My(min). In view of the conclusion that the slabs would remain uncracked under maximum serviceability load, the requirement for the yield criterion to be met could be debated. However, in this case the dependable moment for the slab as originally designed is only slightly above the cracking moment, and hence any reduction in the reinforcing content would be undesirable.

Transverse Frames

Details of the frames are shown in Figure 1. The columns are 800 x 250 mm in section and were cast with a projecting 650 mm beam stub. The central parts of the beams were cast separately and were spliced with in-situ lap joints to the columns. The end 400 mm of beam adjacent to the columns were designated as plastic hinge zones and were debonded from the floor slabs. The effective section of the plastic hinge zones was 600 x 250, with the remainder of the beam length having a total depth of 710 mm.

Because of the variation in section along the beams, the frame deflections were computed using PFRAME (1). Modelled effective cracked section properties were as follows:

- columns Ic = 0.5 Igross (because of low axial load)

- beam plastic hinge zones Ib = 0.5 Igross
- remainder of beam Ib = 0.8 Igross, since the maximum moments for earthquake load combinations were well below the section capacity

For seismic loading, the frames were designed to the limited ductility provisions of the codes. The calculated first mode period of vibration for the frames was 0.27 seconds and hence the "design period" for calculation of earthquake loads under the draft code was taken as 0.4 seconds (Clause 3.5.1.2).

Serviceability Limit State:

Three load combinations apply for the draft code:

(1) D & Ls & I

(2) D & Ls & Es & I

(3) D & Ls & Ws & I

As previously, I = 0 was assumed.

Serviceability criteria considered for the redesign were as follows:

- deflection under D + Ls
- racking deflection under Es (Ws not critical)
- maximum section moments under D + Ls (ST) and D + Ls (LT) + Es (Ws not critical)
- wind induced vibrations (Clause C4.3.1)

As for the slab design, there was almost no net change in the design short term serviceability live load on the frames as compared to the original design. The critical gravity load for deflection calculations was assumed to be Kcp (D + Ls (LT)) + Ls (ST), although the intent of the draft code is not clear in this respect (Kcp = creep factor from concrete code, LT = long term, ST = short term). The calculated maximum deflection was on 3.2 mm (span/2900) which was well within the recommended limit of span/500 in Table C1.5.1 for non-structural damage (c.f. 0.004 l = l/250 in NZS 4203).

The maximum racking deflection under Ls (end frames) was 2.3 mm (0.00085 x storey height). This is almost three times the limit specified in the draft code (Clauses 3.2.3 and 3.3.3.6. Note that there are sufficient plaster board lined partition walls to "alter the seismic response".) However, there appears to be little justification for such a small deflection limit in this situation when the corresponding limit for the serviceability wind load is 0.0015 times the storey height (five times the seismic deflection limit). A limit of 0.001 times the storey height would be more realistic for the seismic case where plaster board lined partition walls are not separated from the structure. Accordingly the frame was not stiffened to comply with the draft code limit. The maximum moment induced in the beams by the serviceability loads was less than 60 per cent of the nominal yield moment (My = 0.75 Mi) of the beams. Part of the reason for this margin is that the beams were overstrong in the original design. However, it is also noted that the same design serviceability earthquake load is used for both full and limited ductile structures. There is no reason from a serviceability point of view why this should not be the case, but this means that at least for strength criteria, the serviceability load combinations should always be less critical for limited ductility structures than for full ductility structures.

Because the building was designed for multiple use, wind loads were calculated for the highest directional design wind speed and most adverse terrain category (category 2), which incidently significantly reduced the design effort required. As confirmed by BRANZ, the orographic multiplier (Mo,Θ) was taken as 1.0 for the serviceability wind speed rather than 1.35 as incorrectly indicated in the draft code for NE winds at the design location (the orographic multiplier should apply only to the strength limit state wind speeds). The resulting wind loads on the frames were only about one-half the corresponding serviceability earthquake loads and hence had no effect on the design.

On the basis of the criterion in Clause C4.3.1 of the draft code, this structure, as expected, is well outside the range in which disturbing wind vibration may occur. (Note however that the draft code contains an error - the word "not" should be deleted from the last line of paragraph 4 in Clause C4.3.1.)

Strength Limit State:

Four load cases apply:

(1) 1.4D

(2) 1.2D & 1.6L

(3) 1.2D & L & W

(4) 0.9D & W

As for the slabs, load case (2) was critical for gravity loading. Again because of the opposing effects of increased basic live load (2 kPa c.f. 1.5 kPa for NZS 4203) and reduced load factors, the design load was within 2.5 per cent of that required by NZS 4203.

As for the serviceability limit state, allowance was made for the most severe wind load conditions in the general locality. In this case, use of $M_{o,\theta} = 1.35$ was required, yielding a critical design gust speed of 54.6 m/s. The resulting design wind loads on the frames were still less than half the corresponding seismic loads for the severe seismic limit state. However, had the frames been designed for full ductility the wind loads would have been within 20 per cent of the design seismic loads, even though it is a two-storey reinforced concrete building.

Severe Seismic Limit State:

The seismic loads calculated in accordance with load combination (1) D & L (LT) & E were very similar to the design seismic loads for the original design and hence the draft code would not affect the amount of reinforcing required.

Because the frames are very stiff, P-delta effects are insignificant.

Longitudinal Lateral Loads

The longitudinal lateral load resisting elements consist of the precast columns loaded about their weak axis, the first floor slab loaded by in-plane diaphragm and normal forces, the in-situ walls at each end of the buildings, and the wall footings. Again, wind loading was not critical and hence only the serviceability load combination D & Ls & Es & I (= 0) and the severe seismic limit state were considered for loads in the longitudinal direction.

Because adequate structural capacity was already available, or could readily be provided, all elements were designed to remain elastic under the design seismic loads in the longitudinal direction. Any torsional deformations due to seismic load in the longitudinal direction are resisted by the transverse H-frames.

Serviceability Limit State:

Because of the use of "elastic" design loads for the severe seismic limit state, the serviceability strength (yield) criteria for seismic loading cannot be critical, i.e. to satisfy the severe seismic limit state requirements, the structural components will need to be five times the strength required by the code serviceability requirements.

The most flexible elements in the longitudinal direction are the columns cantilevering above Level 1. These deflect approximately 2 mm (storey height/1200) under the serviceability seismic load. As discussed previously, this was considered an acceptable deflection for this type of situation.

Severe Seismic Limit State:

Using the nominally elastic spectra and a design response period of 0.4 seconds results in a design base shear coefficient of Co (T1,μe) = 0.55, c.f. C = 0.6 for the original design. Because of the different vertical distribution of seismic loads used in the draft code, this results in design seismic loads at roof level which are approximately 6 per cent less than those required by NZS 4203, while those at and below Level 1 are reduced by 10 per cent.

Because of minimum reinforcing requirements, discrete bars sizes, and other practical considerations, no reduction would be made in the column reinforcement or the drag bars in the first floor slab. An approximate 10 per cent reduction is possible for the principal horizontal and vertical steel in the walls and for the principal flexural reinforcement in the wall footings. However, in total this amounts to a difference of only 18 m of DH12 bar, 34 m of DH16 bar, and 28 m of DH20 bar (total 80 m of rebar).

COST IMPLICATIONS

As discussed in the preceding section, the changes introduced in the draft code have little effect on the critical design parameters for these low rise precast concrete buildings. The differences between the design requirements in 2/DZ 4203 and NZS 4203:1984 would result in a difference of about only 80 m of grade 380 bar (total for DH12, DH16 and DH20 sizes).

At current (May 1989) contract prices, this would amount to a cost difference of only about $230 per 30 bed barracks block. This is only 0.2 per cent of the contract sum for the structural content (approximately $120,000, 1985 costs), or 0.02 per cent of the total building cost.

Cost movements since 1985 would be expected to increase the total contract price by 50 per cent. Prices for concrete reinforcement have held steady over that period, or possibly even reduced slightly.

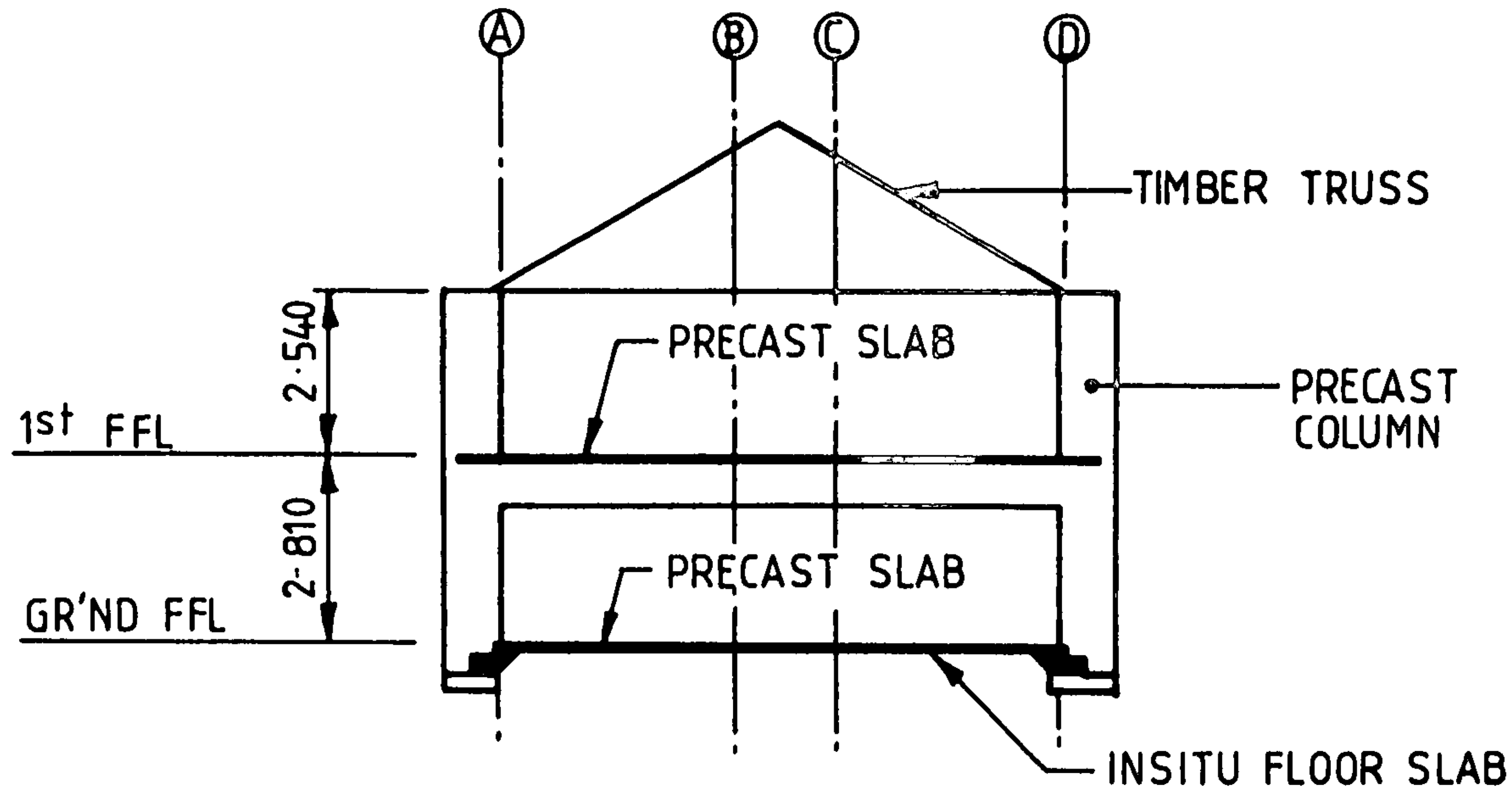
Probably the main reason for the small difference in this case is the fact that the structure is less than 15 m high and has a response period in the 0 to 0.4 second range. The design base shear coefficients for such structures have been limited to the value for 0.4 second period, apparently deliberately to limit the design base shear to levels similar to those applying in NZS 4203:1984.

For this structure, where it is permissible to use the equivalent static method for seismic design, the principal impact on the design process is in the serviceability limit state design. Some of these requirements appear somewhat vague, and it is possible that if carried through fully these requirements could add significantly to design costs. Also, the set-out of parts of the code make it necessary to refer back and forth between different sections of the code.

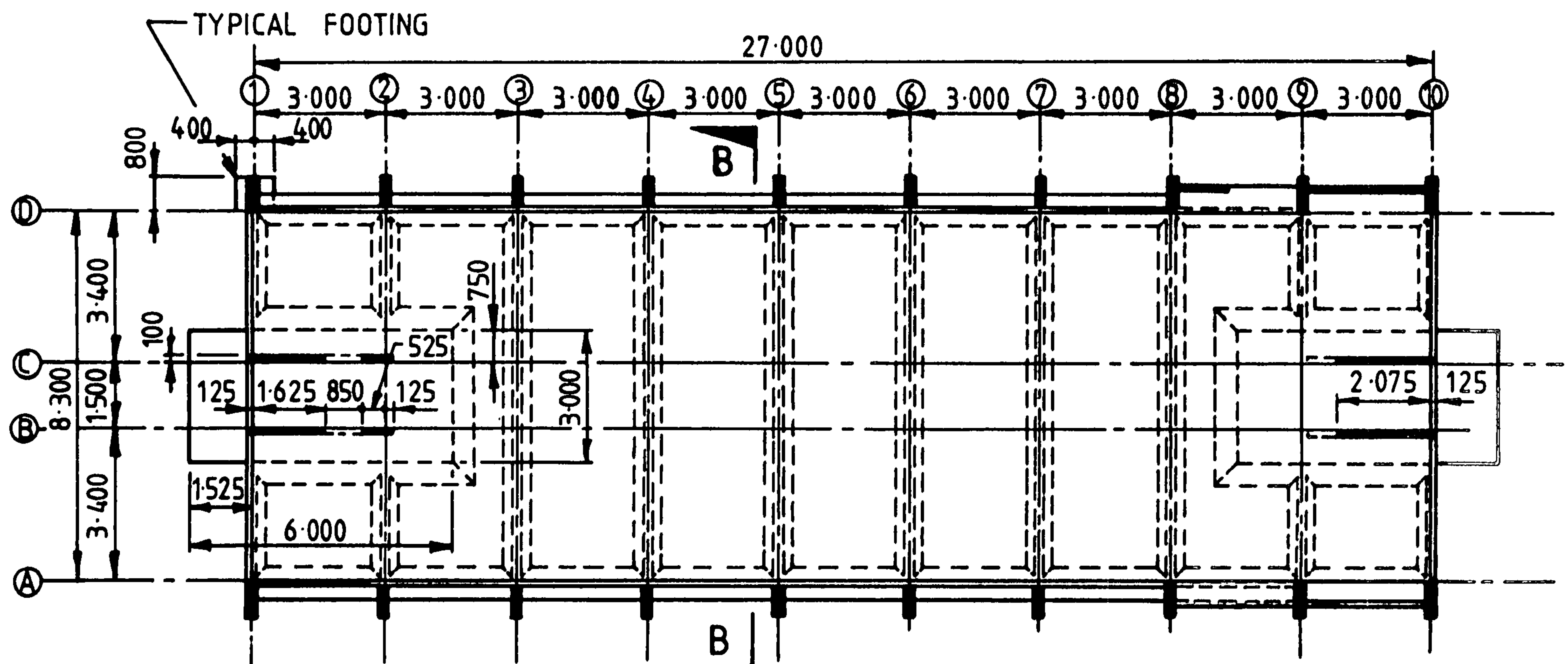
Because of the lack of familiarity with the draft code and the nature of the evaluation exercise which required frequent comparisons with the original design, it is very difficult to access the impact of the draft on design time. However it is expected that under normal conditions the time required to carry out the strength and severe seismic limit state designs for <u>this structure</u> would probably be similar for both codes, expecially if the set-out of the draft was improved. The serviceability requirements of the draft appear more time-consuming but the actual design time will depend on the extent of serviceability checks that are interpreted as being necessary.

REFERENCE

2-D frame analysis computer program, by Softek Services, Canada.



SECTION B - B



GROUND FLOOR PLAN

FIG. 1 BUILDING STRUCTURE

BUILDING TYPE 8

Wakefield Centre, Wellington

Five-storey reinforced concrete retail structure with West-East orientated ductile moment resisting frames and North-South orientated shear walls of limited ductility

Smith Leuchars Ltd
PO BOX 27-349
Wellington

INTRODUCTION

The Wakefield Centre is situated on the edge of the Te Aro flat, Wellington, between Wakefield Street and Courtenay Place. The building comprises 5 levels of retail, one basement and four suspended levels, occupying virtually the whole site and is reasonably rectangular in plan with overall dimensions of approximately 80m X 140m. Each floor has an area of approximately 9300 sq. m with interstorey heights of 5.5m between Levels 1 and 2 and 5.0m between the floors above.

BUILDING DESCRIPTION

Lateral load resistance is provided by ductile moment-resisting reinforced concrete frames, regularly spaced over the length of the building, in the east-west direction and four reinforced concrete shear walls of limited ductility, located along the boundaries, in the north-south direction.

The columns to the seismic frames are founded on a basement raft comprising concrete "spine" beams running across the site on the column/grid lines, linked by concrete slab.

The four boundary shear walls are founded on diaphragm wall "Barrettes" located at the ends of each wall. The Barrette piles are founded on the underlying Te Aro alluviums at a depth of approximately 20m. In terms of the criteria specified in NZS 4203 : 1984 and DZ 4203 : 1989, the site is classified as "flexible".

Secondary reinforced concrete frames along the Courtenay and Wakefield Street frontages and between the shear walls along the West and East boundaries are designed to carry gravity loading only. These structures have not been included in this study.

The floor system to the suspended levels consists of Dycore and Double Tees with cast-in-situ concrete toppings of sufficient thickness to meet structural integrity and fire rating requirements.

A steel roof structure covers the southern half of the building. Again this has not been included in the study.

COMPARISON OF DESIGN PARAMETERS

Summary of External Loads and Load Effects

(Refer Attached Table 1)

Gravity Loads

The basic live load for retail areas as used in the design is 25 per cent greater in the draft code than that in NZS 4203 : 1984. However the change in load factors in the draft code has resulted in a significant reduction in the gravity loading to be considered with the "severe seismic" earthquake from that required by the current code.

There is negligible difference in the ultimate gravity loading to be considered in the strength limit state between the draft and the current code, again as a consequence of the new load factors offsetting the increased live loads.

Seismic Loads

In this study a modal response spectrum analysis was undertaken for each principle direction utilising the same model used in the original design. Any reduction in design actions in the structure resulting from the use of such an analysis method, in lieu of the equivalent static method, could then be taken advantage of, subject to the structure being "regular".

In this analysis it was found that under seismic loading in the North-South direction, the second mode contributed significantly to the structures response to the ground motion. Consequently, as the vertical regularity requirements of clause 3.5.2.3 of the draft were not fully met, scaling of the structural actions in the shear walls derived from the model analysis, in accordance with clause 3.5.2.4, was undertaken. The structure was found however to satisfy the regularity requirements in the West-East direction, precluding any need for such scaling of the structural actions in the ductile moment-resisting frames.

The live load to be considered as part of the seismic weight has been reduced in the draft, resulting in a reduction in the seismic weight of the building to 97 per cent of that under the existing code. However, despite this, the severe seismic limit state forces resulting from the use of the draft are considerably greater than the corresponding forces derived from NZS 4203 : 1984, with increases in the seismic base shear of 23 per cent and 72 per cent in the West-East and North-South directions respectively.

The serviceability limit state base shears are correspondingly 83.5 per cent and 23.7 per cent of those of the severe seismic limit state.

The severe seismic limit state load combination $D + \psi L + \lambda E$, including P-delta effects assessed in accordance with clause C3.3.4 of the draft, was found to produce structural actions of a very similar magnitude as the load combination $D + \psi L + E$ for lateral loading of the ductile frames in the West-East direction.

Because the shear walls are very stiff, P-delta effects are insignificant for lateral loading in the North-South direction.

Wind Loads

From a conservative evaluation of the wind forces on the building, wind loading was not found to be critical for either the strength or serviceability limit state and hence was not considered further.

Structural Components

Shear Walls and Barrette Piles

With the large increase in seismic forces resulting from the use of the draft, as noted previously, and the consequent increase in base moment, it was found necessary to increase the thickness of the shear walls at each level (from 350 to 400mm at Level 1 - 3, 250 to 300mm at Levels 3 - 5) to keep shear stresses within the limits of NZS 3101, and to also increase the flexural reinforcement content. The increase in flexural reinforcement was not of the same order as the increase in overturning moment due to the presence of part of the live load in axial loading on the walls.

With the wall base moment resisted by a couple action between the diaphragm wall Barrettes, an increase in longitudinal reinforcement in the Barrette piles also resulted. The Barrettes themselves were of sufficient size and capacity to resist the increased loadings without rupture of the surrounding country.

Given the use of shear walls of limited ductility, (mu = 2), it was found that the serviceability limit state earthquake did not govern the wall design.

Ductile Moment Resisting Frames

The attached Table 2 allows comparison of the maximum ultimate negative beam end moments, prior to any redistribution, for a typical 10m span beam.

It should be noted that gravity loads contribute significantly to the overall loading on the beams under load combinations including earthquake inertia forces.

From Table 2 it can be seen that, given no moment redistribution can be carried out for the serviceability limit state, this limit state governs the flexural design of the beam end moments at each level once redistribution is undertaken for the strength limit state. The capacity provided at level 2 and 4 is respectively 38 per cent and 27 per cent greater than that provided in the original design to NZS 4203 : 1984. This is balanced somewhat by a reduction in the bottom midspan flexural reinforcement required under the draft due to the reduced uniform loading on the span under the severe seismic limit state and the minimal level of redistribution now necessary under the strength limit state. The resulting increase in flexural reinforcement as redetailed in the beams is in the order of 11 per cent overall for the building. The required transverse shear reinforcement was found to be unaltered, again due to the reduced distributed load on the span under the severe seismic limit state.

However, the requirement of NZS 3101 that $p' \geqslant 0.5p$ in the plastic hinges of the beams results in a significant increase in the column design moments derived from a capacity design procedure.

The resulting column flexural reinforcement requirements are approximately 50 per cent greater than that of the original design to NZS 4203 : 1984. The level of transverse reinforcement provided to the columns in the original design to NZS 4203 : 1984 was found to be adequate for the actions resulting from the draft code loadings.

As the design of the basement raft to the underside of the frames was governed by the gravity loading on the columns, this remained as initially designed as there was very little change in the level of column loading in the strength limit state compared with that derived using NZS 4203 : 1984. As noted previously, the increase in floor live loads tends to be offset by the reduction in load factors.

Serviceability and Secondary Components

Lateral deflections for both the serviceability and severe seismic limit states, evaluated in accordance with the suggested procedure in the draft, were found to be considerably less than the limits specified for loading in each principal directions, for both the overall deflection of the building and interstorey drifts.

The maximum interstorey drift of the seismic frames was 69.9mm, 200 per cent of that calculated under NZS 4203 : 1984. The draft requires that any element or part that could potentially endanger life, if not adequately separated, be separated from the structure by this amount, as well as any part that could alter the seismic response of the building. This must obviously apply to external cladding or glazing and it is questionable whether separations of this size can be accommodated by such glazing elements. Under NZS 4203 : 1984, separations of only half this magnitude are required.

It is apparent that whilst the deflection and interstorey drift limits of the draft suggest reasonably slender structures may be used, the maximum interstorey drifts which can be accommodated by secondary elements of external cladding etc, are going to dictate the final sizing of the members of the structure.

General Design Procedures

In this study, load effects due to earthquake loading were initially evaluated by a modal response spectrum analysis for each direction of loadings.
From the results of these analyses, design of the shear walls and the beams to the moment resisting frames could be simply performed. However, in accordance with the recommendations of C3.8.2, to undertake a capacity design of the columns to the seismic frames it was necessary to derive the first mode shears from the modal analysis and then reanalyse the structure under equivalent static forces to obtain a distribution of column and beam actions which were in equilibrium.

With the mandatory requirement that P-delta effects are to be checked, a further reanalysis was required with equivalent static forces (pseudo P-delta loads) to obtain these load effects on the structure.

It was necessary to combine these manually with the load effects from the modal analysis for the severe seismic loading to obtain the resulting seismic actions under the $D + \psi L + \lambda E$ load combination.

A significant increase in design work has resulted using DZ 4203 : 1989 and its recommendations, with the evaluation of load effects from the severe seismic load combination $D + \psi L + \lambda E$, including P-delta, being a three step process, and that for the load combination $D + \psi L + E$ a two step process at the very minimum.

This compares with the single step process required under NZS 4203 : 1984. The increased work at the analysis stage alone is in excess of 200 per cent of that required under the current code.

COST IMPLICATIONS

From a comparison of the estimated costs of the redesigned structure with those of the original design based on NZS 4203 : 1984, we estimate an increase in overall costs as follows:

Item	
Shear Walls (including Barrette piles)	$128,000
Seismic Frames	$152,000
	$280,000

This represents approximately 1.6 to 1.8 per cent increase in the building's structure cost, or approximately 0.5 to 0.6 per cent of the overall building cost.

The increase in design effort using DZ4203 is estimated to be in the order of $5,000 for this structure representing an increase of 0.9 per cent in the design fees. Assuming an engagement on a percentage fee basis the resulting proportional increase in design fee of 1.6 to 1.8 per cent would cover the cost of this additional work.

Table 1 - Summary of External Loads and Load Effects

Gravity Loading Data

ITEM		NZS 4203 :1984		DZ 4203 : 1989	
- Basic Live Load	Lb	- Retail	- 4 kPa		- 5 kPa
		- Parking	- 2.5 kPa		- 3 kPa
- Severe Seismic L.S. Seismic Weight (typical Floor)			- 10.2 kPa		- 9.9 kPa
- Severe Seismic L.S. Gravity Loading		$D + 1.3\ L$	= 121.5 kN/m	$D + \psi L$	= 87.1 kN/m
- Strength L.S. Loading		$1.4D + 1.7\ L_R$	= 162 kN/m	$1.2D + 1.6L$	= 160 kN/m
- Serviceability L.S. Loading		-		$D + L_S$	= 104 kN/m (short term)

Seismic Loading Data

ITEM	NZS 4203 : 1984	DZ 4203 : 1989	RATIO
- 1st Mode Period To			
North - South	0.2	0.2	
West - East	1.28	1.26	
- Base Coefficient (K) Cd			
North - South	0.216	0.384	177%
West - East	0.0475	0.061	127%
- Base Shear Vb			
North - South	82.5 MN	141.8 MN	172%
West - East	18.2 MN	22.4 MN	123%
Note Scale Factor for Serviceability EQ = 0.835	West - East		
= 0.237	North - South		
- Building Deflections			
a) Severe Seismic			
* At Top of Building			
- West - East : Delta Code X $\frac{K}{SM}$	117		-
Delta 0		203	-
Delta P		22	-
Total	117	225	-
Total/Allowable	-	225/410 = 55%	-

ITEM		NZS 4203 : 1984	DZ 4203 : 1989
- North - South	Delta Code X K/SM	5.4	
	Delta 0	-	11
	Total/Allowable	-	11/410 = 3%
* Interstorey (maximum)			
- West - East	Deflection	35	69.9
	Actual/Allowable	35/50 = 70%	69.9/150 = 47%
b) Serviceability EQ			
* Interstorey			
- West - East	Deflection	-	8.7
	Actual/Allowable		8.7/17.5 = 50%

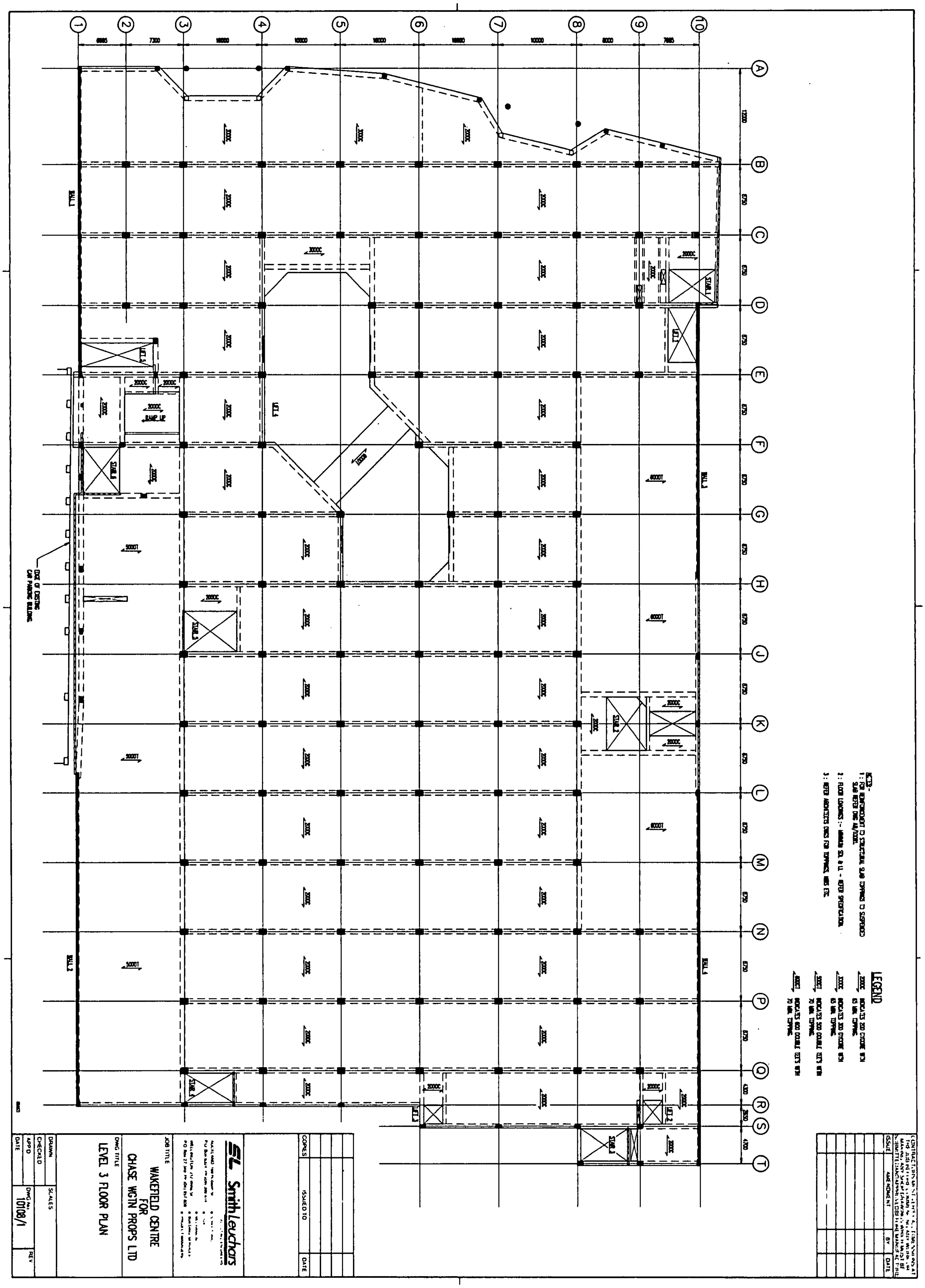
Wind Loading Data

ITEM	NZS 4203 : 1984	DZ 4203 : 1989
Base Shear : Vu		
North - South	3508 kN	4662 kN
West - East	6140 kN	4622 kN
Base Shear : Vs		
North - South	2698 kN	3531 kN
West - East	4722 kN	2811 kN

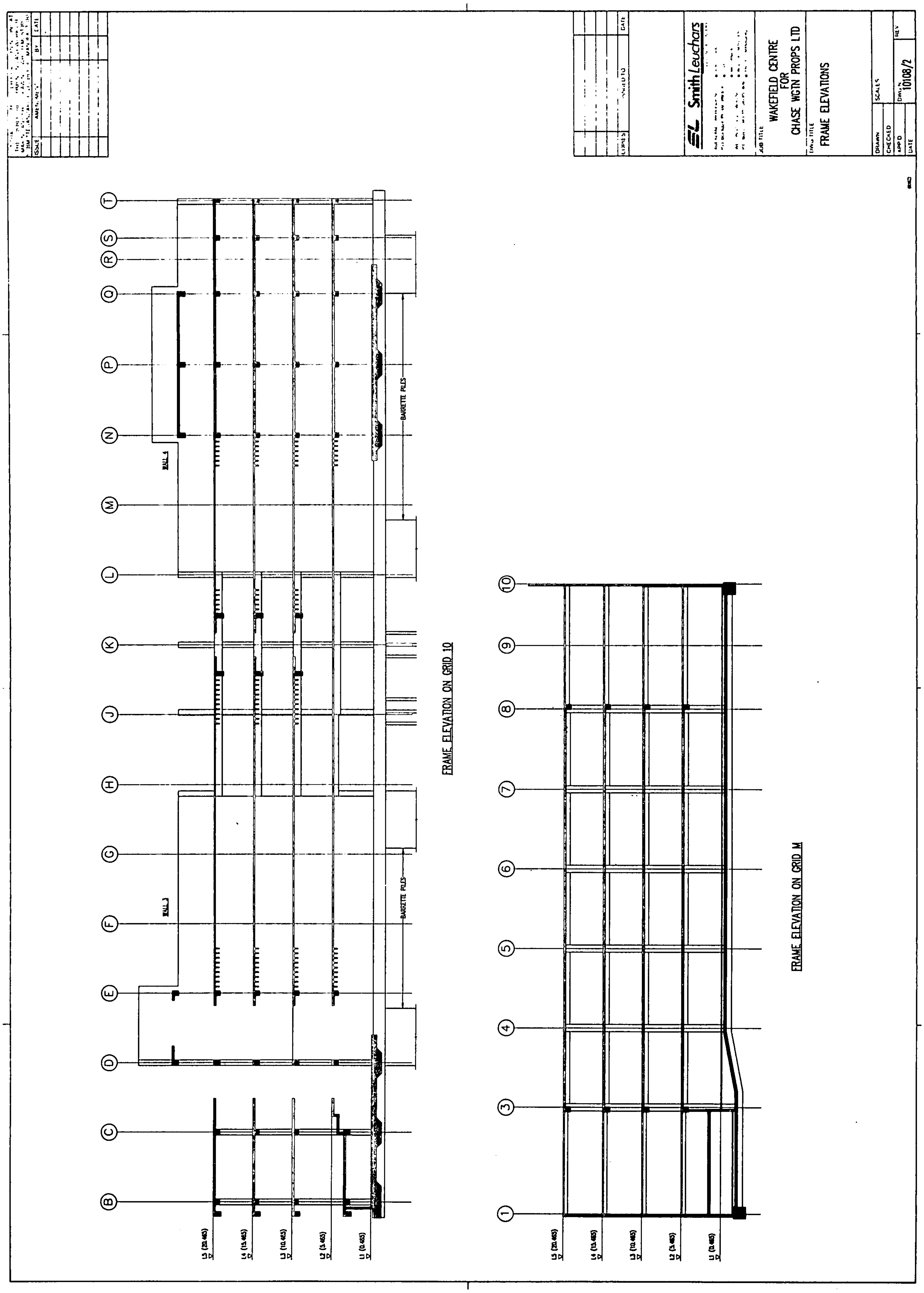
Table 2 - Ductile Moment Resisting Frames - Comparison of Load Effects

STOREY LEVEL	BEAM MOMENTS Mu (kNm) SEVERE SEISMIC L.S.		STRENGTH L.S.	SERVICEABILITY L.S.
	$D + \psi L + E$	$D + \psi L + \lambda E$ (PΔ included)	$1.2\ D + 1.6L$	D + Ls + Es *
5	753	716	1110	870
4	861	817	1110	978
3	932	906	1110	1048
2	971	974	1110	1087

* Ms limited to 0.75 Mi (as recommended in "Supplementary Notes for Reinforced Concrete") hence equivalent Mu derived from Ms x $\frac{0.9}{0.75}$ = 1.2 Ms.



Smith Leuchars
JOB TITLE
WAKEFIELD CENTRE
FOR
CHASE WGTN PROPS LTD
DWG TITLE
LEVEL 3 FLOOR PLAN
10108/1
LEGEND


Smith Leuchars
WAKEFIELD CENTRE
FOR
CHASE WGTN PROPS LTD
FRAME ELEVATIONS
10108/2
FRAME ELEVATION ON GRID 10
FRAME ELEVATION ON GRID M
BARRETTE PILES
WALL 4
WALL 3

BUILDING TYPE 9

Wilson Neill House, Dunedin

Ten-storey office building incorporating ductile reinforced concrete moment resisting frames

Hadley and Robinson Ltd
PO BOX 6068
Dunedin

INTRODUCTION

Wilson Neill House is located on the southeast corner of Moray Place and George Street, Dunedin. It is one of but a few buildings in the locality of any great height (30 m from ground). Only one other building of equivalent height is nearby - Cargill House, about 100 m to the South.

The site is at a fairly level part of Dunedin, one block from the Octagon. However, there is a rise of about 20m at 1:10 grade commencing about 50m from the building and running down to the reclaimed flat area of the Queens Gardens and the railway yards and industrial area beyond.

Proximity to the harbour is rather surprising - 1000m to the south and east, and 500 m to the southeast.

BUILDING DESCRIPTION

Wilson Neill House is a glass walled building of 11 Levels:

Level 1:	carpark
Level 2:	retail (ground floor)
Level 3:	carpark
Levels 4-10:	offices
Level 11:	concrete roof with access for pedestrians

The basement is excavated into rock. This is generally andesite, with some basalt boulder inclusions in weathered upper levels. There is also a lens of extremely dense sand, of the consistency of sandstone but not cemented, crossing the site. Foundations are simple pads or strip footings.

The principal structure is a two-way reinforced concrete frame system. Floors are the Stahlton type, with precast prestressed ribs at 900 centres, permanent timber infills, and a 75 thick concrete topping. Beams are composite, also. Standard U shell beams of precast prestressed concrete are 400 x 400 overall, and have an insitu concrete core poured with the floor topping and column joint.

Beams are 600 deep overall. Columns are 500 x 500 insitu reinforced concrete.

The main grid is 10.500 x 7.400 m. Floors span the 7.400 direction onto the beams which span 10.500 between column centres. Additional spans of 6.000 occur over part of the building width. Further spans of 4.600 occur at these same plan locations, but only between Level 2 and Level 4 where they terminate at a lightweight roof. The two-bay frame arrangement is repeated at one end, near carpark ramps, between Level 2 and Level 4 where they terminate in a lightweight roof.

All walls are of flexible timber construction, or are detailed so that they do not participate in resistance to lateral loading, as for glass curtain walling or precast concrete on the boundaries.

DETAILS OF FRAMES

Frames are detailed as ductile moment-resisting two-way frames. Each beam is detailed with four potential plastic hinge regions, two at each end. At the ends, the negative moment hinge is located 1.000 clear of the column face, and the positive hinge at the column face. Each is so detailed that hinging can occur in the intended direction only - reversal of hinging is thus precluded, but cracks will close on reversal of moment sense.

This arrangement was adopted because the frames tend to be dominated by gravity load, due to the rather long beam spans.

The beam strength at the hinges is that which arises when minimum flexural reinforcement is used. The location is therefore best where service load moments are just met. The arrangement also prevents the occurrence of large overstrength in the frames in a global sense, and the relocated negative hinge eases anchorage of a large number of bars into columns. It is also believed that the relocation of the negative hinge will permit reduction on demands on joint regions, even given the location of the positive hinge at the column face (but with much smaller moments).

There is no debonding of the shell beams near ends. Increasing positive moment away from the column faces, together with the transfer of the hinge moment at the column, is therefore taken by the prestressed construction.

SELECTED APPRAISAL OF ELEMENTS

The two-bay frames were adjudged as being representative of the building as a whole, and this study is restricted to them. A rough appraisal of salient quantities has been made of typical elements elsewhere, but details are not recorded herein, for reasons of brevity.

DESIGN LOADS

Representative loads on the selected two-bay frame for each of DZ4203 and NZS4203 are listed in the following table.
Seismic loads are based on a fundamental period of 1.85 seconds.

Table: Design Loads

Load Description	Code	Value	Unit
Floor dead loads, including ceilings, services, partitions etc (unfactored)	BOTH	2.9	kPa
Floor live loads (unfactored)	NZS	2.5	kPa
	DZ	3.0	kPa
Wind load base shear (factored) *	NZS	300.0	kN
	DZ	264.4	kN
Earthquake base shear (SM=0.64) *	NZS	307.1	kN
(mu=6.00)	DZ	136.2	kN

* Base shears are shears at ground floor level. Shears are transmitted through the ground floor slab (transfer diaphragm) to basement walls.

GENERAL APPRAISAL

As outlined above, beam strengths are dictated by gravity load considerations and the requirements of minimum reinforcement.

For NZS, wind and earthquake base shears are of similar magnitude, but earthquake overturning moment, which is a better measure of strength demands, is greater than for wind. For DZ, base shear and overturning moment for wind are both of greater magnitude than for earthquake.

In both cases, however, the requirements of capacity design lead to earthquake being the dominant effect for all but the flexural design of beam hinges.

Capacity design is effected by a direct method in this office. A variety of load patterns, including the code pattern, are applied to the structure. All patterns have the load intensity scaled so that the overturning moment is equal to that produced by the code pattern of load. For each case the load is progressively increased until the deflection at the top of the structure has reached the value implied by the selected ductility factor (4/SM, or mu). The computations allow for overstrength in all hinges.

It is of interest to note that the effective SM product revealed by this procedure applied to NZS earthquake, is 1.1, and the effective mu for DZ is 2.3. The inelastic demands are thus greater for the NZS case than for DZ. This is reflected in the beam plastic hinge rotations, which are as follows.

NZS4203	Negative hinge rotation	= 0.029 radians
	Positive	= 0.021 radians
DZ4203	Negative hinge rotation	= 0.012 radians
	Positive	= 0.006 radians

Plastic hinge rotations for all other load cases not involving earthquake are modest. It can also be shown that satisfaction of serviceability requirements does not control.

The effect of P-delta is interesting. P-delta reduces the strength available for earthquake resistance by 21 per cent when NZS loads are considered, but by only 8 per cent when considering DZ loads. This is a direct consequence of the greater deflections arising under the higher loads specified for NZS earthquake.

Deflections under the various loads are best illustrated in diagrams. The attached diagrams, drawn to scale, show the comparisons between the earthquake generated deflections when the maximum deflection at the top of the structure has been reached under the code specified loading pattern and flexural overstrength has developed. This is load case 15, as noted, and is one of the capacity design cases which are explored. In the diagrams, filled circles represent potential plastic hinges which have formed, and open circles those which have not formed. For comparison, case 11 for DZ4203, which is for the loading D & 0.4L & W, is shown with deflections enhanced by a further ten-fold. In this case, of course, hinges shown have developed dependable flexural strength only. In all cases, column base hinges have not formed - it requires other patterns of earthquake loading to produce this.

BEAMS AND COLUMNS

Bending moment and shear force diagrams for a typical floor are shown in further diagrams. Level 4 (second level above ground, beam elements 39 to 42) has been chosen for illustration. In these diagrams, all significant load combinations are shown. The diagram is for strength demand, found by normalising the actions by dividing the strength reduction factor (which is dependent on load cases and, for columns, on the level of axial load). All cases not involving capacity earthquake are included within the hatched area. Capacity design cases lie partly outside this area. Other lines (e.g., the dashed lines) represent ideal strength for pre-selected combinations of reinforcement. It is seen that DZ has introduced significant economies into the beams.

For flexural design, a measure of savings effected by the DZ amendments is the area of the bending moment diagrams. This, tempered by practical considerations of bar placement and curtailment, demonstrates a reduction in main reinforcement of 21 per cent from the NZS requirements. The most significant cause of this is the reduction in live load for combination with strength limit state loads, and the fact that, for earthquake, the live load is held constant.

For shear steel, direct comparison is somewhat obscured by doubts about the contribution of the concrete to shear resistance for DZ cases, where the plastic hinge rotations are less than for fully ductile design but greater than the suggested limits for limited ductility. It is estimated, however, that shear reinforcement would reduce by about 35 per cent.

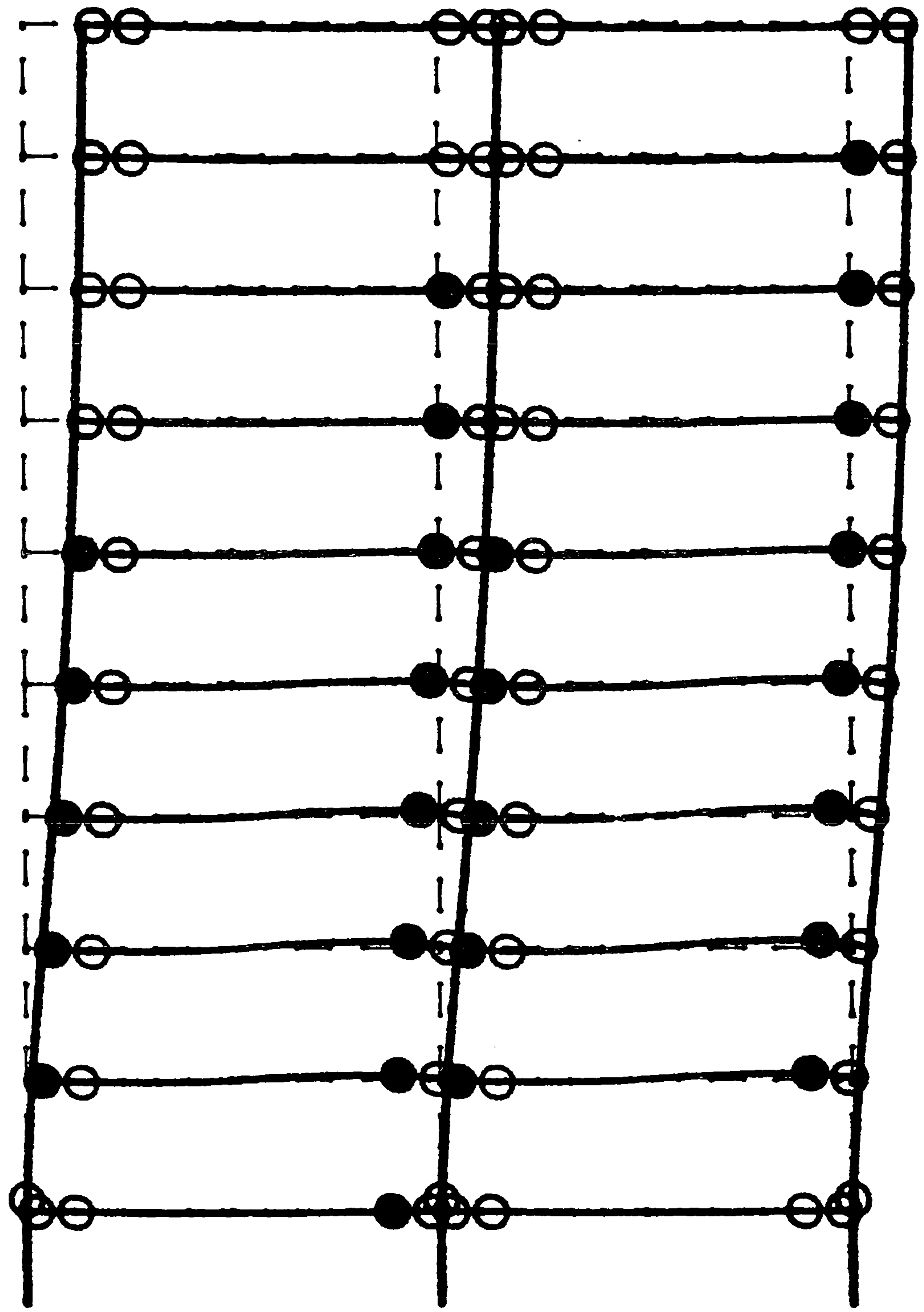
Similar economies are evident for the columns, but direct comparison cannot be made from bending moment and shear force diagrams because of the effects of axial load and the requirements for confinement and joint reinforcement. The further diagrams for one full-height exterior column show the bending moment diagrams for each of NZS and DZ. Using an actual bar count, derived from interaction diagrams, and an assessment of confinement requirements, it is estimated that total savings are 49 per cent for longitudinal bars and transverse reinforcement.

These savings represent a saving in the structural content of some 7.5 per cent, or 2.5 per cent in the total cost (before fit-out) of the building.

DESIGN COSTS

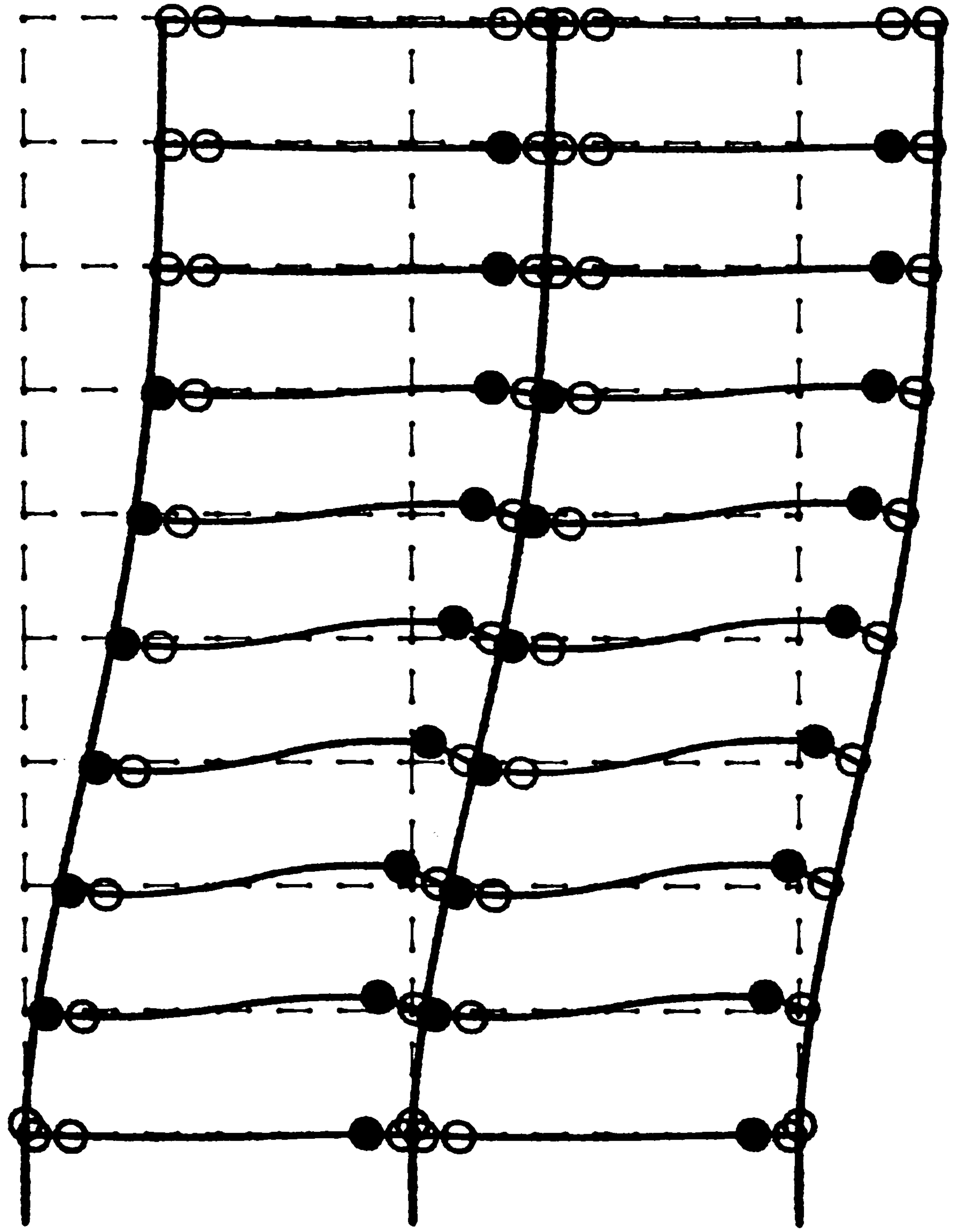
Actual time spent in design is affected by the number of controls that need to be checked. The considerably greater freedom afforded by the draft code eases the need to check against rather arbitrary controls, but does require, if this freedom is to be used, a more sophisticated analytic procedure. In this office, these procedures are employed anyway, so that there was minimal change in design time, if set-up times for slight changes in solution controls are discounted.

DZ4203/2



DEFL CASE # 15 [A] : Exaggeration = 10

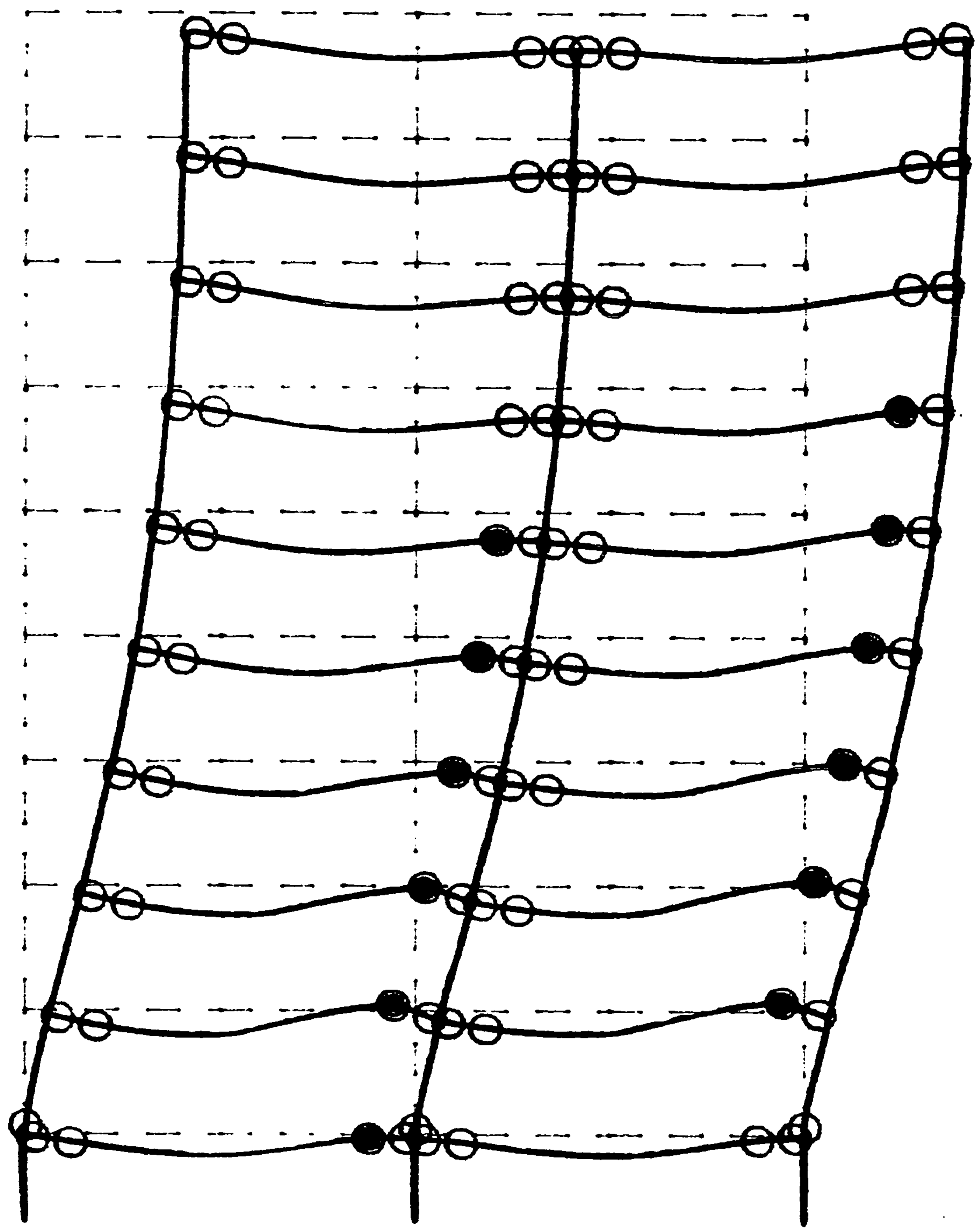
NZS4203



DEFL CASE # 15 [A] : Exaggeration = 10

83c

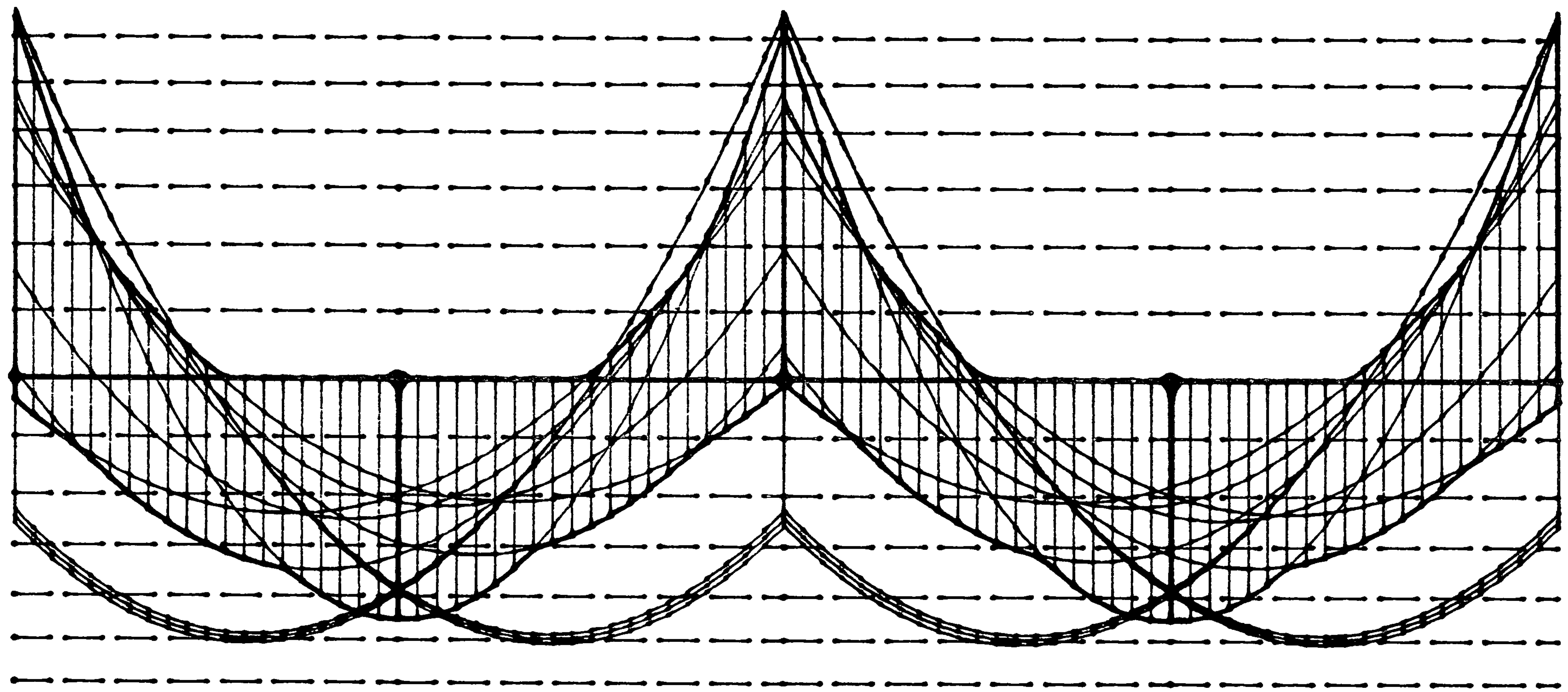
DZ4203/2



DEFL CASE # 11 [A] : Exaggeration = 100

ELEMS 39-42 :Case #ALL [FACTORED] 1:100

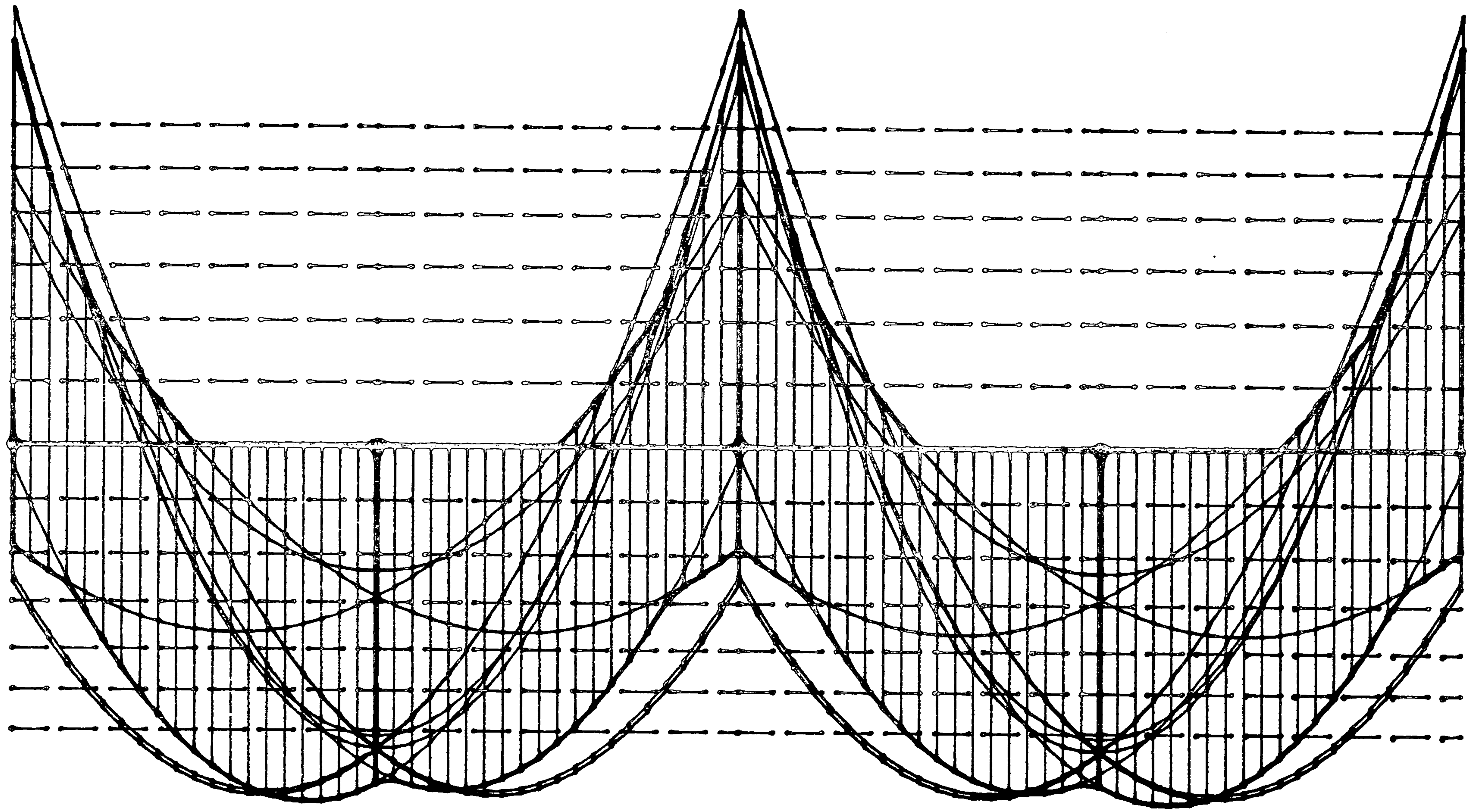
DZ4203/2



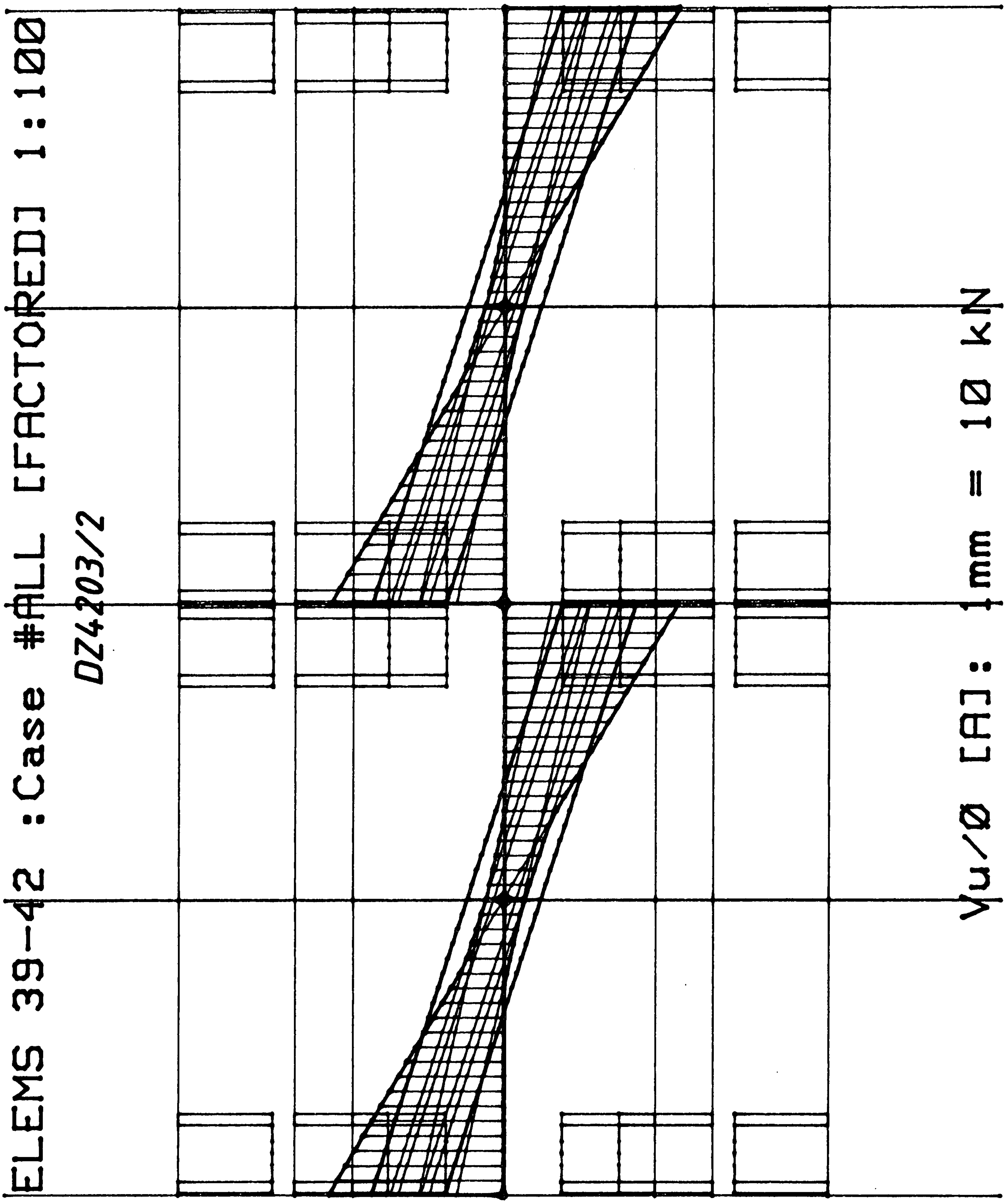
Mu/Ø [A]: 1mm = 10 kNm

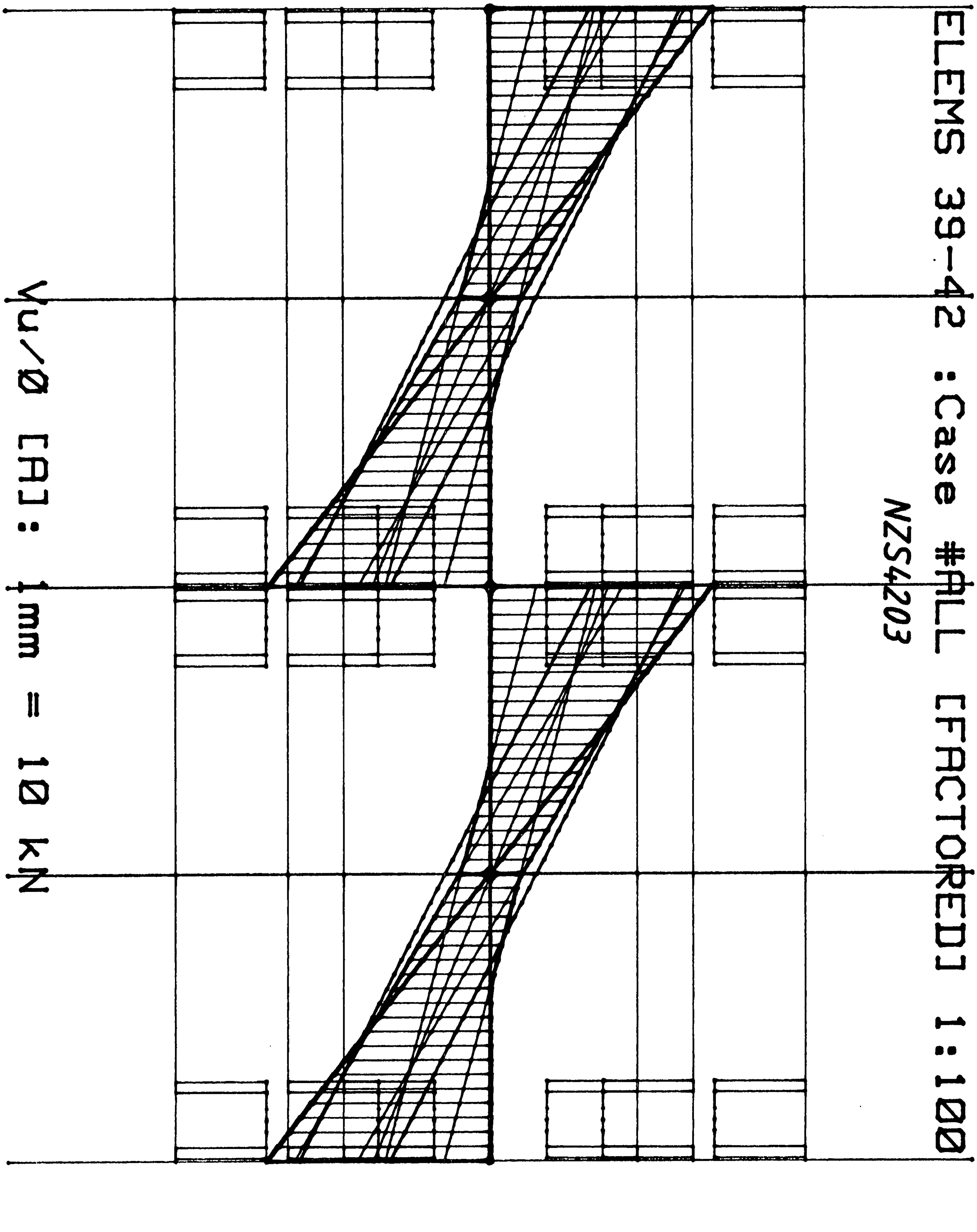
ELEMS 39-42 :Case #ALL [FACTORED] 1:100

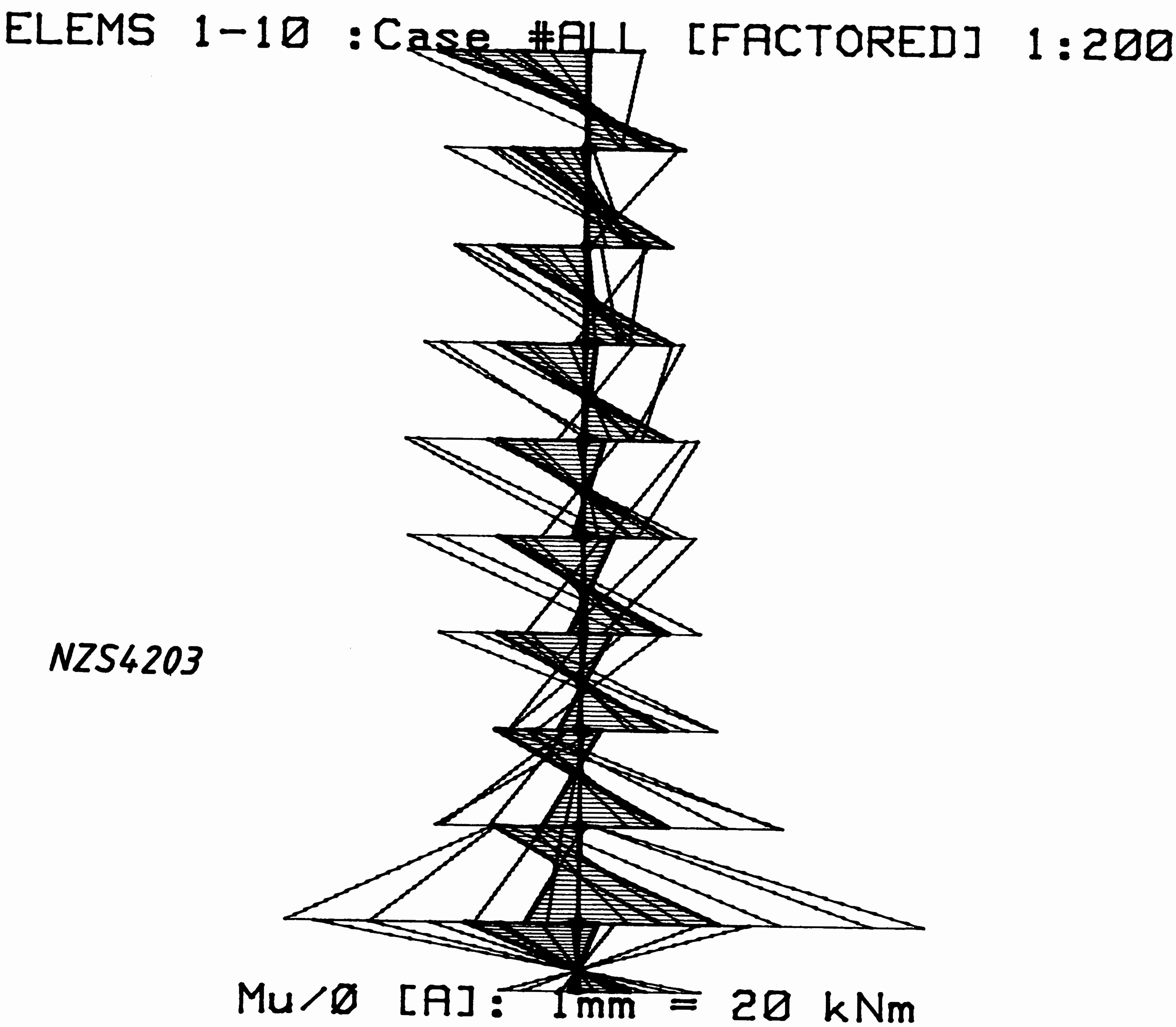
NZS4203



Mu/Ø [A]: 1mm = 10 kNm


ELEMS 39-42 :Case #ALL [FACTORED] 1:100
DZ4203/2
Vu/Ø [A]: 1mm = 10 kN

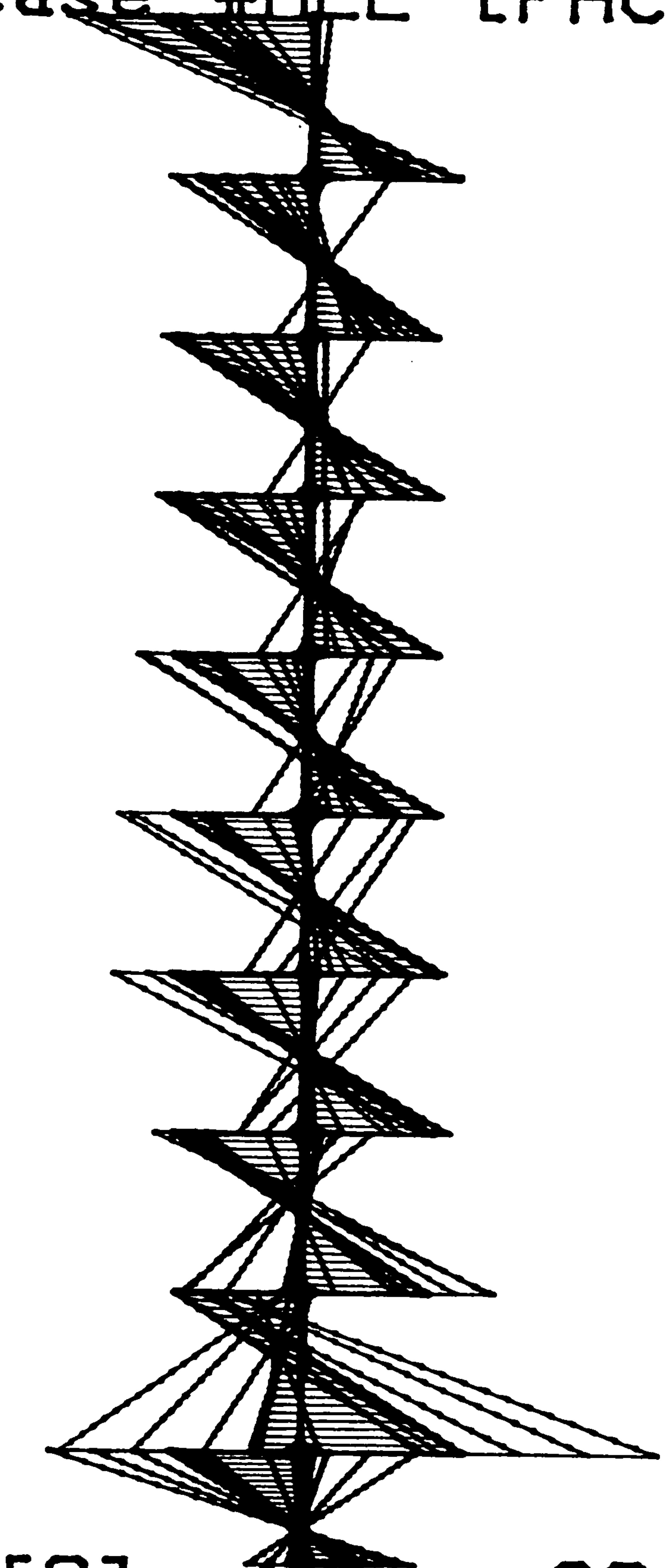

ELEMS 39-42 :Case #ALL [FACTORED] 1:100
NZS4203
Vu/Ø [A]: 1mm = 10 kN



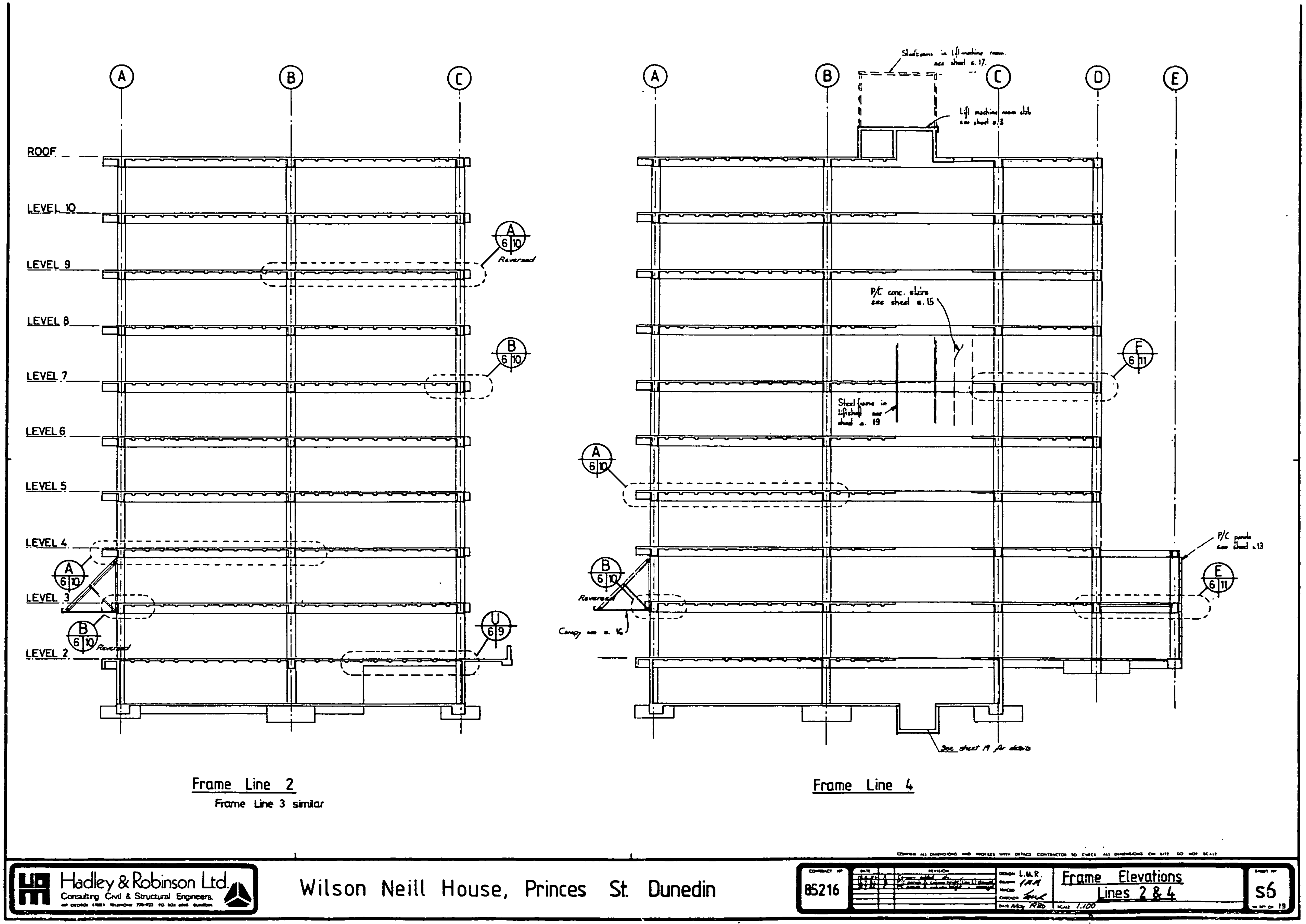
83h

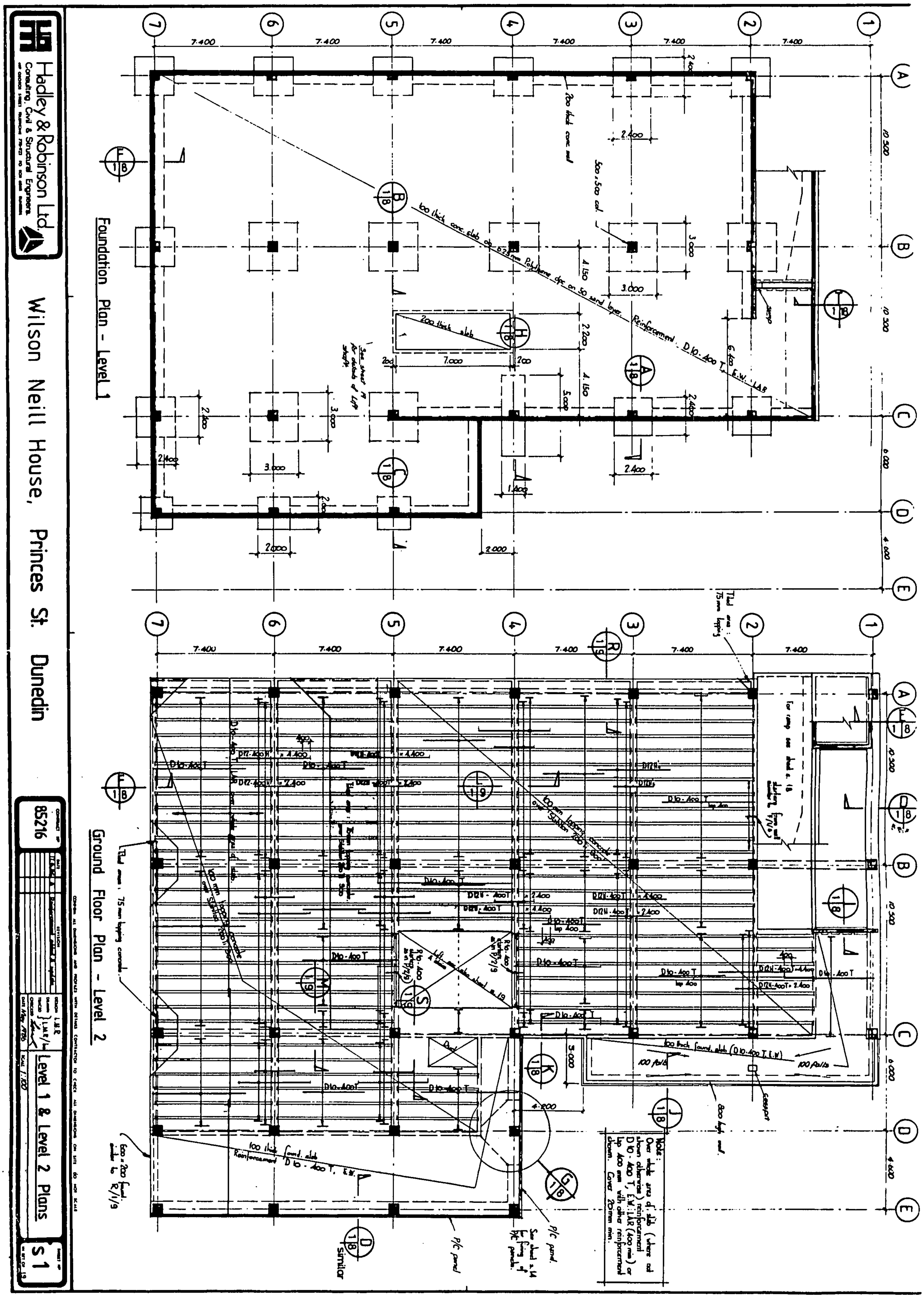
ELEMS 1-10 :Case #ALL [FACTORED] 1:200

DZ4203/2



Mu/Ø [A]: 1mm = 20 kNm


ROOF
LEVEL 10
LEVEL 9
LEVEL 8
LEVEL 7
LEVEL 6
LEVEL 5
LEVEL 4
LEVEL 3
LEVEL 2
A
B
C
D
E
Reversed
Frame Line 2
Frame Line 3 similar
Frame Line 4
Steelbeams in Lift machine room. see sheet s.17.
Lift machine room slab see sheet s.13
P/C conc. stairs see sheet s.15
Steel frame in Liftshaft see sheet s. 19
P/C panels see sheet s.13
Canopy see s. 16
See sheet 19 for details
Hadley & Robinson Ltd.
Consulting Civil & Structural Engineers.
Wilson Neill House, Princes St. Dunedin
85216
L.M.R.
May 1980
1:100
Frame Elevations
Lines 2 & 4
s6
19


Hadley & Robinson Ltd.
Consulting, Civil & Structural Engineers
Foundation Plan - Level 1
Ground Floor Plan - Level 2
Wilson Neill House, Princes St. Dunedin
85216
Level 1 & Level 2 Plans
S1

BUILDING TYPE 10

Unisys House, Wellington

Thirteen-storey octagonal shaped building incorporating ductile reinforced concrete moment resisting frames

Morrision Cooper Ltd
PO BOX 10-283
WELLINGTON

INTRODUCTION

This report covers the analysis of the seismic and gravity frames of Unisys House in accordance with the draft code DZ4203:1989, and the design of the reinforced concrete elements of the frames to NZS 3101.

Unisys House was designed originally in 1985 to the requirements and intent of NZS 4203:1984 and also NZS 3101:1982.

The design office procedures, the resulting structural elements, structural deflections and their effect on non-structural elements are also investigated and reported. The cost implications involved between designs to NZS 4203:1984 and DZ4203:1989 for the basic structural elements and for the design time are also assessed.

BUILDING DESCRIPTION

Unisys House is situated on The Terrace, Wellington. It was designed for Aurora Group, and built by Fletcher Development and Construction. The building is 13 storeys high, and octagonal in plan with overall dimensions of 37m x 27m. Each floor has an area of approximately 860 m2.

The building is founded on enlarged base cast-in-situ bored piles, one under each column. The piles are founded in greywacke at a depth of approximately 6m. The greywacke is overlain by sandy silts and silty gravels. In terms of the criteria specified in Clause 3.4.3 of NZS 4203 the site is classed as "flexible".

Lateral load resistance is provided by a peripheral reinforced concrete moment resisting frame, which is a closed ring. This frame comprises precast cruciform units consisting of a two-storey high column with two levels of beam stubs. The columns are spliced with grout-filled NMB splices. The beam reinforcement is spliced with a 180 degree hook lap in a cast-in-situ midspan joint.

The two internal reinforced concrete frames are designed to resist gravity loads only, but are detailed for ductility under the seismic deflections. These frames comprise precast beams and columns with the beam-column joint cast in-situ. The base of the columns are tied to the column below with two shear dowels.

The ground floor slab is cast on the ground and is designed as a diaphragm to transfer the earthquake generated base shear to the peripheral seismic frame. The suspended floors and the roof consist of 200mm Dycore with a 65mm cast-in-situ topping. A steel canopy is attached to the building at third floor level on all sides. This structure was not considered for this review.

COMPARISON OF DESIGN PARAMETERS

Seismic Frame - Subject to Earthquake

In the original design of the structure a modal response spectrum analysis was used in accordance with NZS 4203. A set of storey forces was derived from the differences in the combined storey shears and scaled to produce 90 per cent of the code base shear. This set of forces was then used in a static analysis to determine the member design actions. The capacity design procedures recommended in the Commentary of NZS 3101 were used for the design of the structural elements.

In this review a modal response spectrum analysis in accordance with Clause 3.8 of DZ 4203 was carried out. Contrary to the recommendations of DZ 4203 (Clause C3.8.2), the storey shears from the modal analysis were used to produce a set of forces for a static analysis. This was done because it was considered that the approach suggested in the commentary of the draft to use the first mode analysis or the equivalent static analysis method were inappropriate for this structure. The basis for the decision was that the participating mass of the first mode is only 75 per cent and thus use of the first mode only does not comply with clause 3.8.1.3 which requires that a sufficient number of modes be included in the analysis so that a minimum of 90 per cent of the mass is participating in the direction of the analysis. The equivalent static method was not used because the structure does not meet the vertical regularity criteria of the draft.

The low tail of the response spectrum in the draft means that the first mode is significantly less dominant than that of NZS 4203.

For this study it was found that four modes in each direction were necessary to attain 90 per cent participating mass as required by Clause 3.8.1.3.

Only three modes in each direction were considered in the original design.

Seismic forces resulting from the use of the draft were considerably less than the NZS 4203 forces. The code base shears for the draft were 2934 kN and 2272 kN in the two principal directions. The original design base shear (90 per cent of the code base shear) was 3902 kN in each direction. This represents reductions in seismic base shear of 25 per cent and 42 per cent respectively.

Of the two severe seismic limit state load combinations specified in Clause 1.6.4, P-delta effects were found to be critical in the lower nine floors in the short direction (see Tables 1 and 2).

There was a reduction both in size and reinforcement quantities in the beams, and a reduction in reinforcement quantities in the columns. The beams were reduced in depth from 700mm to 650mm, but the column size was unchanged. Some of the reduction in column reinforcement was due to the fact that part of the live load is now always considered to be present, increasing the column axial forces. Under NZS 4203 the load combinations for column design were D+L+E and 0.9D+E, whereas in terms of the draft the only combination that need be considered is D+0.4L+E (clause 1.6.4.4).

TABLE 1. SEISMIC STOREY SHEARS - SHORT DIRECTION

STOREY	NZS 4203	DZ 4203 SEVERE SEISMIC LIMIT STATE	
		COMB. (1)	COMB. (2) (P-delta)
13	528	530*	455
12	985	888*	799
11	1372	1089*	1030
10	1720	1259*	1241
9	2033	1400	1435*
8	2316	1522	1620*
7	2572	1646	1811*
6	2799	1751	1993*
5	2998	1855	2179*
4	3168	1959	2371*
3	3315	2061	2574*
2	3415	2185	2791*
1	3460	2272	2979*

TABLE 2. SEISMIC STOREY SHEARS - LONG DIRECTION

STOREY	NZS 4203	DZ 4203 SEVERE SEISMIC LIMIT STATE	
		COMB. (1)	COMB. (2) (P-delta)
13	523	599*	493
12	986	1068*	903
11	1377	1364*	1187
10	1722	1598*	1428
9	2028	1794*	1644
8	2302	1963*	1843
7	2549	2128*	2043
6	2771	2274*	2233
5	2970	2414	2422*
4	3145	2552	2615*
3	3300	2691	2817*
2	3408	2836	3023*
1	3458	2933	3192*

TABLE 3. WIND STOREY SHEARS

STOREY	LONG DIRECTION		SHORT DIRECTION	
	NZS 4203	DZ 4203	NZS 4203	DZ 4203
13	164	69	250	111
12	328	203	500	328
11	492	331	750	542
10	655	453	1000	751
9	819	569	1250	953
8	983	682	1500	1152
7	1147	789	1750	1351
6	1311	890	2000	1546
5	1475	991	2250	1748
4	1638	1092	2500	1950
3	1802	1193	2750	2152
2	1966	1294	3000	2340
1	2130	1395	3250	2510

Seismic Frame - Subject to Wind

The strength and serviceability limit state wind loading were determined in accordance with Section 4.3 of the draft using the Detailed Procedure Static Analysis. Terrain Category 4 with a roughness length of 2.0m was assumed as the terrain description of the area in which Unisys House is sited.

The effect of shielding was investigated for winds from the east and south-east, but the building spacing parameter of 16 resulted in a shielding multiplier of 1.0. The effect of the escarpment caused by the Terrace was also investigated. It was found that the topographic multiplier varied form 1.18 at ground level to 1.0 at level 9 (27.52 m) with no effect above this level. Design gust wind speeds were determined at each level for each of the eight directions. The maximum gust wind speed was 48.6 m/s from the south-east direction at level 14 and the minimum 34.0 m/s at level 2 and ground. Directional gust wind speeds for the strength limit state are shown in Figure 4.

Wind loads for the strength limit state were then determined for the north-east and the south-east directions which gave the greatest loads at each level. These are set out in Table 3.

The ETABS program was used to analyse the frames for winds in the north and east directions. In all cases it was found that actions resulting from seismic loadings governed.

Gravity Frames

The reduction in member size and reinforcement in the gravity frames was not as significant as in the seismic frame. Member sizes remained the same and reinforcement was reduced by only 5 per cent. The increase in floor load from 2.5 kPa to 3.0 kPa offsets the reduction in the load factors. As in the original design, 10 per cent redistribution of negative moment was carried out.

Serviceability and Non-structural Components

The serviceability limit state was found to govern the beam reinforcement at all levels except the top three, where the minimum reinforcement requirements of NZS 3101 governed.

The lateral deflections calculated in accordance with the serviceability limit state of the draft method (a) (clause 3.2.1(a)) were found to be less than the limits specified in clause 3.2.3 (0.0035 of the storey height or 12mm).

Maximum interstorey deflections were 6.1mm and 8.97mm under NZS 4203, and 5.9mm and 7.4mm under the draft code (for the set of forces determined from the scaled storey shears). For NZS 4203 the separation distance required for elements that could alter the structural behaviour to a significant degree is 42mm and 62mm in the two principal directions. For the draft the maximum separations required are 50mm and 64mm between ground and first floors. These were calculated from the suggested deflection envelope in fig. C3.3.1. This small increase should not cause any problems with current practice for separation of such elements. However the separation distances required for elements such as glazing, which could be considered to potentially endanger life if not separated, are 21mm and 31mm under NZS 4203, but are increased to 50mm and 64mm under the draft code. Allowance for deformation of this magnitude could be difficult to accommodate in conventional window sections.

COST IMPLICATIONS

Construction Cost

The costs of the primary structural components assessed in this review and the costs of the same components in the existing building, based on labour and material rates as at May 1989 are:

COMPONENT	NZS 4203	DZ 4203
seismic frame	\$ 705,637	\$ 651,056
gravity frames	\$ 232,183	\$ 221,391
	\$ 937,820	\$ 872,447
	difference =	\$ 65,373

These represent a 8 per cent reduction in the cost of the seismic frame, and a 5 per cent reduction in the cost of the gravity frames. Unisys House was completed in 1987 at a cost of \$12.7M(excluding the canopy). The estimated cost at May 1989 rates is \$14.2M. The above cost reduction on the structure represents a 0.5 per cent reduction in the total cost of the building, and a 7 per cent reduction in the cost of the structure.

Design Cost

The fee for the structural design of Unisys House, in 1989 terms, was approximately \$240,000. It is estimated that an additional 250 man hours, or \$20,000, would be required to design to DZ 4203:1989. This represents an increase of 8 per cent in the design fees or 0.1 per cent of the total building cost and 2 per cent of the structure cost.

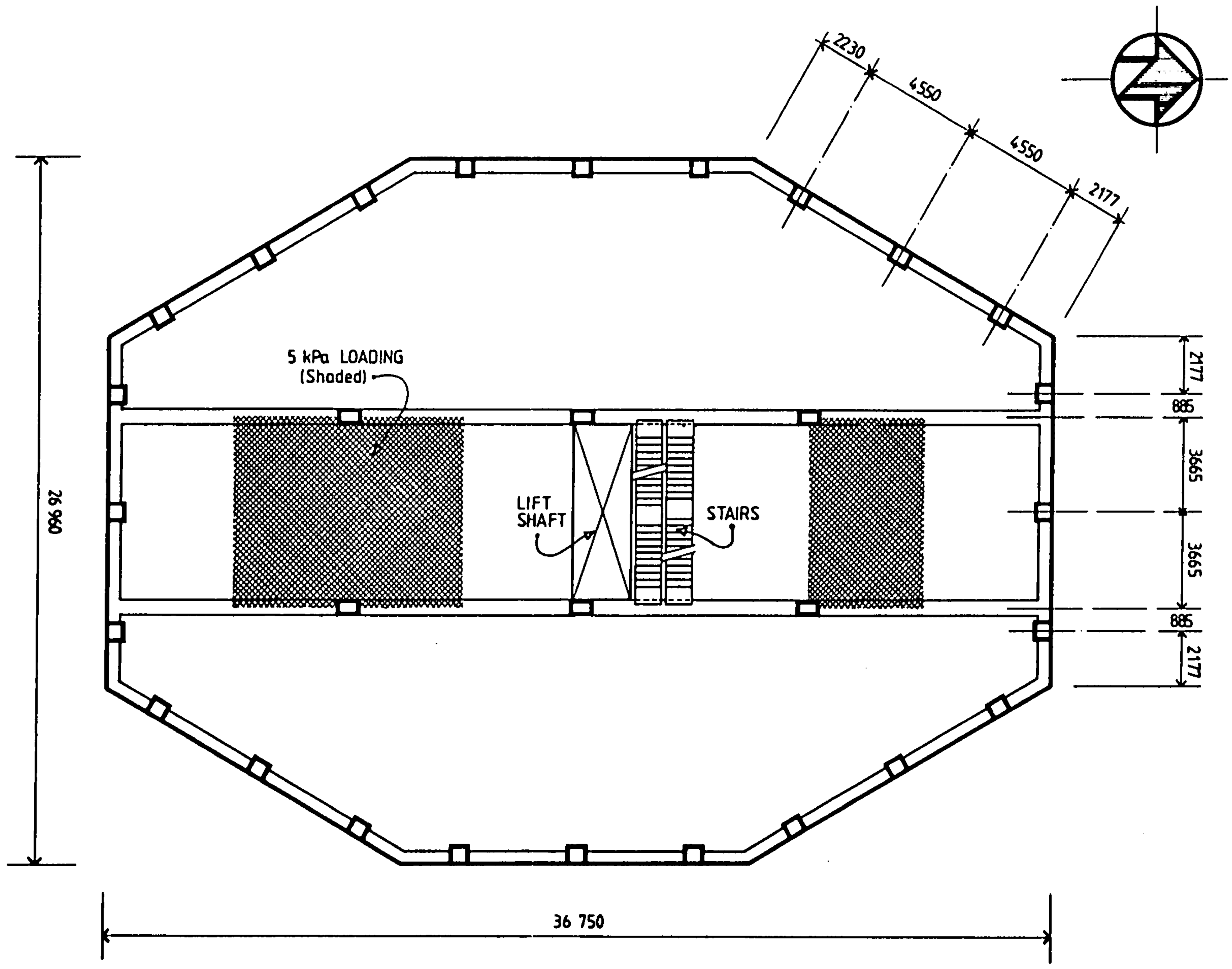
CONCLUSIONS

The use of DZ 4203:1989 resulted in a reduction in the base shears of 25 per cent and 42 per cent in the principal directions when compared with NZS 4203. However, a corresponding reduction in the cost of the seismic frame was not possible as P-delta effects were found to govern the design at the lower floors.

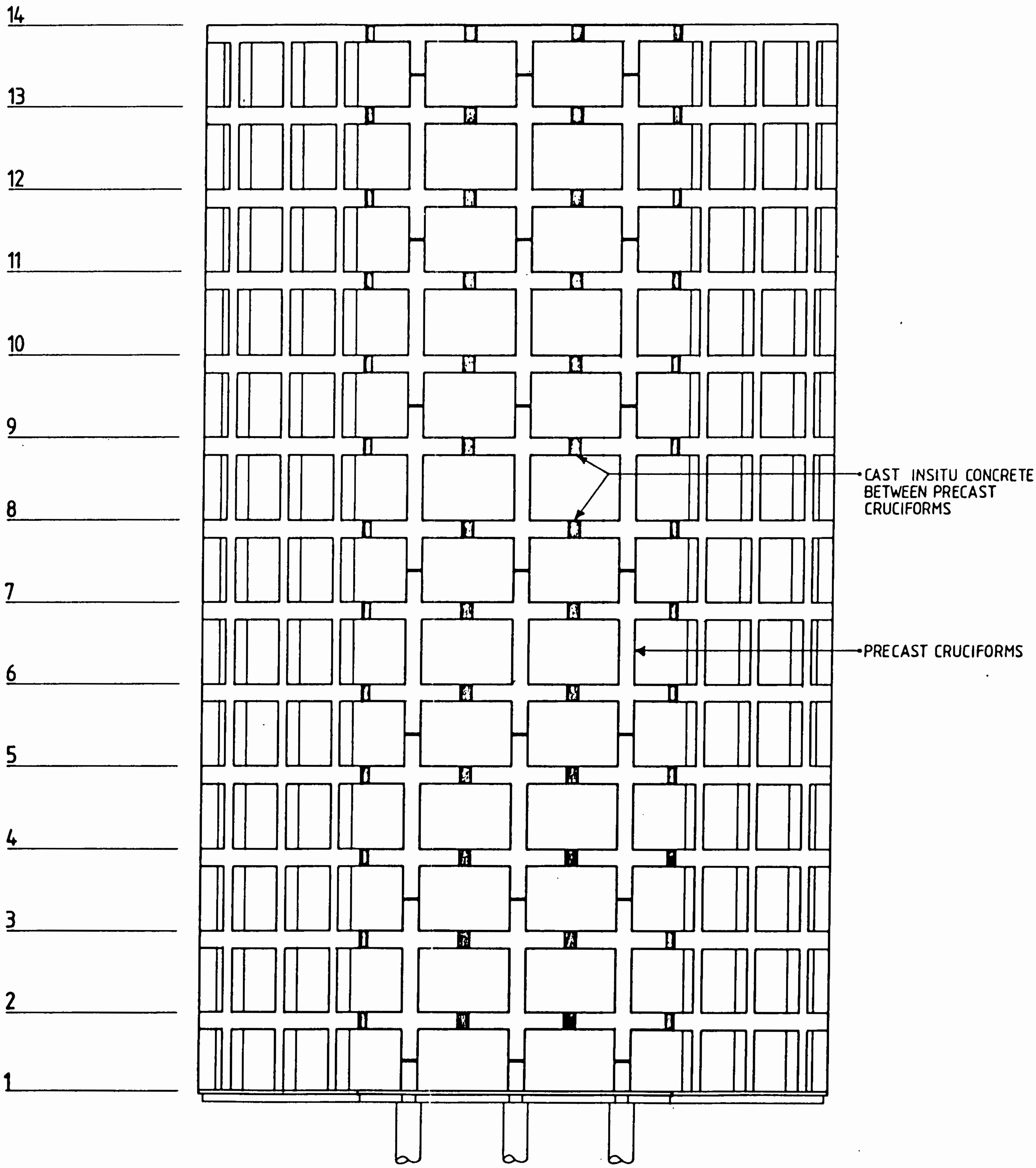
The use of the draft resulted in only a small decrease in the cost of the gravity frames, as the reduced load factors were offset by the increase in the basic live load to 3 kPa.

Structural deflections under the severe seismic limit state forces were reduced. However, the magnitude of the separation distances for secondary elements has increased significantly. The increased separation distance was minor for rigid elements, but the 106 per cent increase in separation distance for elements such as glazing could prove difficult to accommodate in conventional window sections.

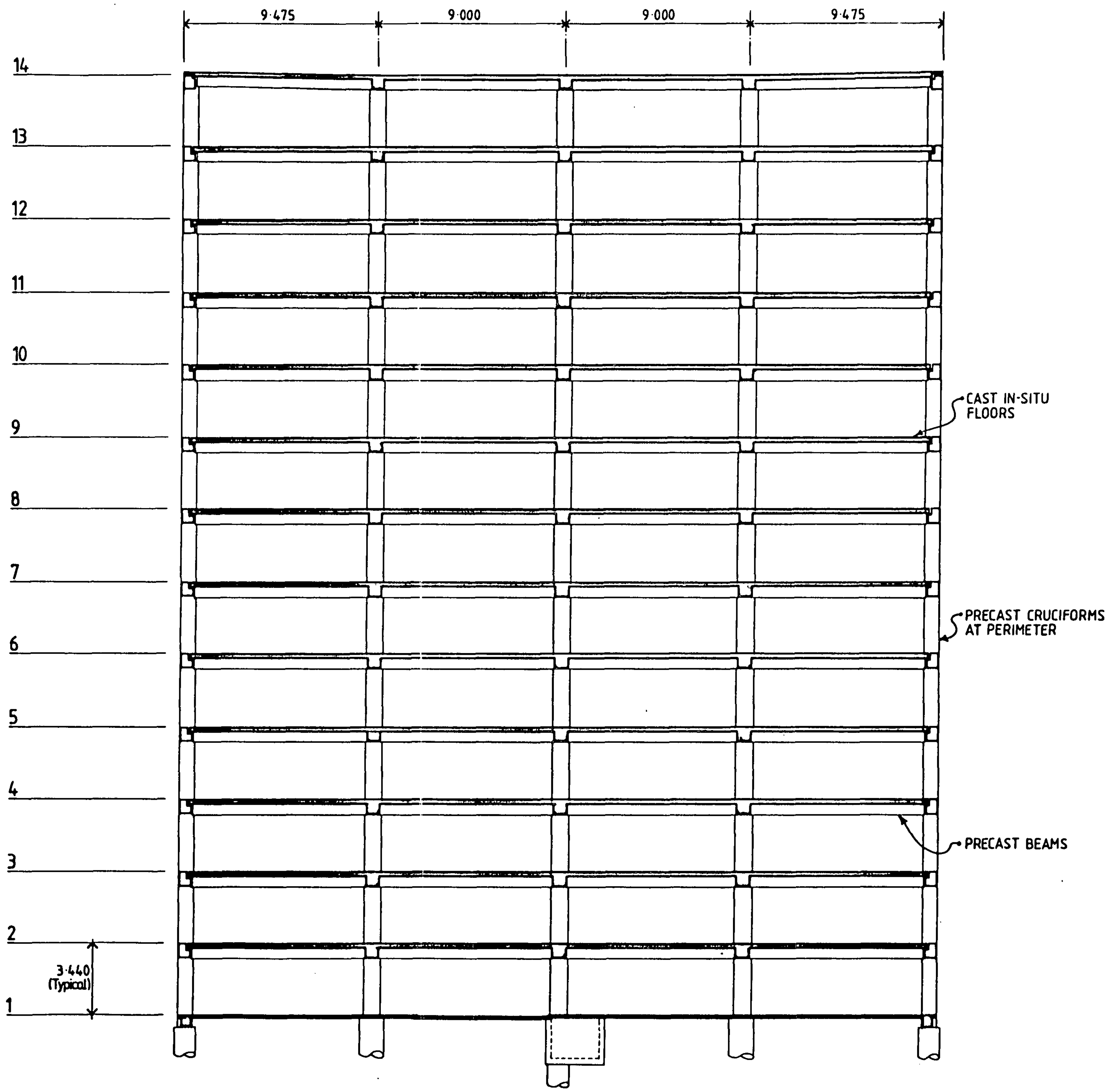
We estimate that the use of DZ 4203:1989 produced a small decrease in overall building cost of the order of 0.5 per cent ($65,373), but an increase in design cost of approximately $20,000.



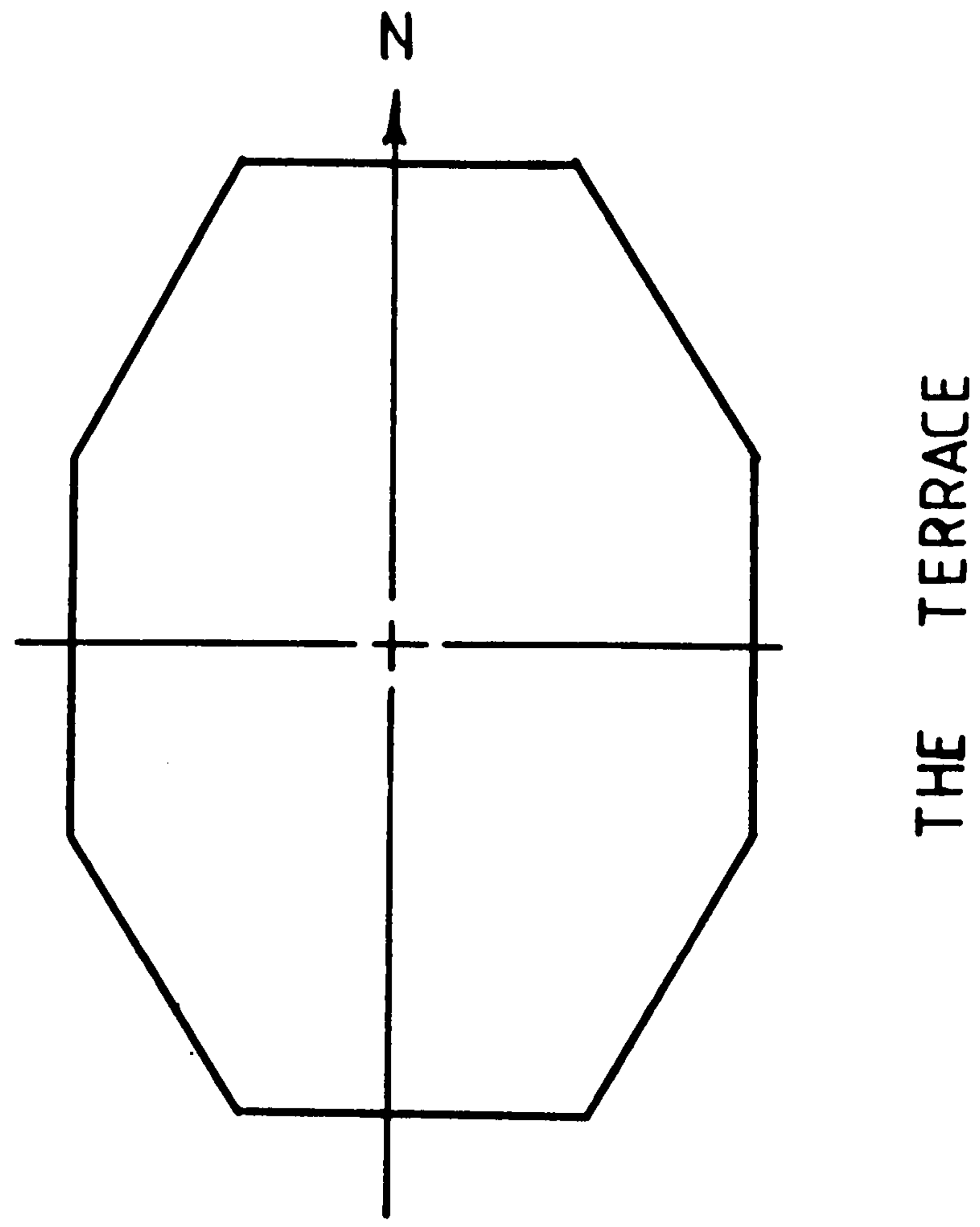
TYPICAL FLOOR PLAN



PERIPHERAL SEISMIC FRAME ELEVATION



GRAVITY FRAME ELEVATION



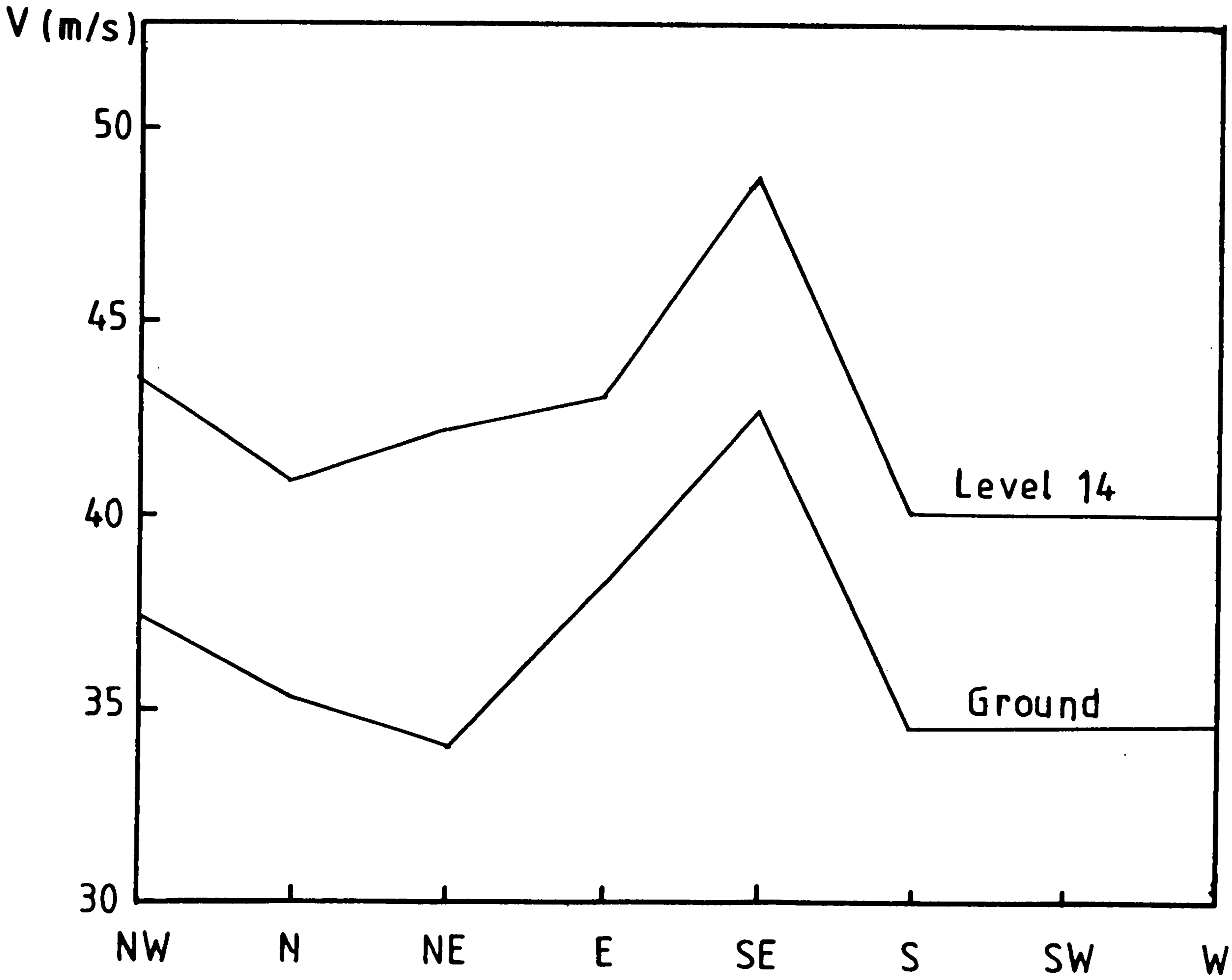


FIG 4 : Design Wind Velocity

BUILDING TYPE 11

Twenty-eight-storey Building, Auckland

Office building incorporating ductile reinforced concrete moment resisting frames above level 4 and reinforced concrete shear wall core below level 4.

Smith Leuchars (Auckland) Ltd
PO Box 6324
Wellesely Street
AUCKLAND

INTRODUCTION

In this report key elements of a typical 28 storey building have been redesigned to DZ4203, and the results compared with those of the design based on NZS4203.

The report covers the basic design parameters used, a comparison of the effects on typical members, and an assessment of the resultant cost implications.

BUILDING DESCRIPTION

Location

123 Albert St, (Downtown) Auckland.

Dimensions

Storeys - 27 suspended levels
Interstorey Hgt - typical = 3.42m
- Lvl 1-2 = 5.94m
Floor Plan - Typical tower = 30.8 x 32.3m
- Podium (Lvl 1-3) = 30.8 x 44.m
Height to Roof - 94.86m

Structure

Lvl 28 to 4 - Ductile Reinforced Concrete Frame
Lvl 4 to 1 - RC Frame plus Core Shear Walls (designed elastically for frame overcapacity)
Seismic Ground is at Level 1

Typical Members :-
Flooring System - 200 Dycore
Beams - perimeter - 600w x 950 dp
- internal - 600w x 600-800dp
Columns - perimeter - 1100 x 800
- internal - 1000 x 1000

Foundations

Piles at each column location, founded in well cemented Waitemata series sandstone

Seismic Mass

(Of unmodified building to NZS4203:1984)

Typical tower storey = 924t
Podium level = 1400t
Total @ G.L. = 26950t

Design Loads

Superimposed Dead Load - 0.5 kPa
Live Load - 4.0 kPa

COMPARISON OF DESIGN PARAMETERS

External Load Effects

1. Seismic Data

Item		NZS4203:1984	DZ4203:1989	Ratio
Original Building				
First mode period		2.8 sec	2.8 sec	
Base Coefficient	Cd	0.036	0.012	33.3 %
Typ. floor seismic mass		924 t	874 t	94.5 %
Base shear	Vb	9520 kN	3000 kN	31.5 %
Base Moment	Mb	611 MNm	200 MNm	32.8 %
0.75EQ + P-Delta @ base		N/A	9625 kN	101 %
" " " " " - Moment		N/A	355 MNm	58.0 %
Modified Building (Perim. Beams 950x500, Perim. Cols 1100x600)				
First Mode Period				3.0 sec
Base Coefficient	Cd	0.0112		31.1 %
Typ. floor seismic mass			834 t	90.3 %
Base shear	Vb		2695 kN	28.3 %
Base Moment	Mb		191 MNm	31.2 %

Scale Factor for Serviceability Eq = 0.78

NOTE 1: For the original design, beam stiffnesses were based on 0.5 Ig. In accordance with the "Supplementary Notes for Reinf Conc" for the project which suggests that 0.5 Ig is too low, this was modified to Icr = 0.65*Ig.

NOTE 2: The base shear value given above as `0.75EQ + P-delta @ base' of 9625 kN is given as a comparison value only. The P-delta effect is not due to any external loads, but is simply a modelling procedure to simulate the internal actions of the structure due to inelastic deformations.

Building Deflections

@ Top of Building	-	Delta nought	= 205 mm
(Severe EQ)		Delta p	= 95 mm
		Total/Allowable	= 300/1900
			= 16 %
Interstorey	-	Deflection	= 28 mm
(Severe EQ)		Actual/Allowable	= 28/103
			= 27 %
Interstorey	-	Deflection	= 2.0 mm
(Serviceability)		Actual/Allowable	= 2.0/12.0
			= 17 %

2. Wind Load Data

The two critical wind directions were north, and west, the data for which is presented below. Wind speeds and pressures are for 100 m height, which is top of the structure.

Item	DZ4203 NORTH	WEST	NZS4203 NORTH
Terrain Category			
- @ location of building	4	4	
- upstream of structure	1	3	
Basic Wind Speed Vu	44 m/s	46 m/s	33 m/s
Multipliers - terrain Mz	1.19	1.12	
- product of other, Mt, Mc etc	1.03	1.05	
Design Wind Speed (V[100])	53.9	54.1	
Design Wind Pressure (q[100])	1.75 kPa	1.76 kPa	0.79 kPa
Force on a typical top storey	243 kN	248 kN	125 kN
Base Shear	5685 kN	5460 kN	2765 kN
Ratio of Original EQ Vbase	59.7 %	57.4 %	29%
Base Moment	303 MNm	291 MNm	156 MNm
Ratio of Original EQ Mbase	49.6 %	47.6 %	25.5%
Scale Factor for Service Wind Load	0.625	0.580	

3. Comparison of Beam Actions

Due to the different shape of the various "external" loads, each becomes dominant at different regions of the structure. We found the following general grouping :

Top 1/4 of Structure - Severe EQ
Mid 1/2 " " - Wind
Btm 1/4 of Structure - 0.75EQ + P-Delta

This is illustrated in the following table, which gives the design group levels, and comparison of redesign with previous. Moments are averaged beam end moments for typical perimeter beam H4 - H5.

	Beam Moments (kNm)		Critical	
Storey Level	**NZS4203**	**DZ4203**	**L.Case**	**Ratio**
28 - 23	497	157	EQ	32 %
22 - 17	909	358	Wind	39 %
16 - 13	1223	520	Wind	43 %
12 - 8	1407	748	EQ+P-Delta	53 %
7 - 5	1352	820	EQ+P-Delta	61 %

By inspection, it can be seen that the redesign actions are from 30 to 70 per cent lower than those of the original design, for this structure. This means a significant reduction in reinforcing steel content for those members in which seismic or wind actions dominate.

Gravity Load Design

1. Strength / Severe Seismic Design

These are based on the external load effects above, which, in conjunction with the appropriately factored gravity loads, will determine the critical design case. However, even for a seismic dominated member, the serviceability criteria must be checked, and may still govern.

The following table shows the average end reinforcing steel (top and bottom combined) of perimeter beam H4 - H5, and compares new and old resteel contents.

Average Beam Steel

Storey Level	NZS4203	DZ4203	Ratio of Steel
28 - 23	6.8 D32	7 D28	79 %
22 - 17	8.2 D32	7 D28	66 %
16 - 13	12 D32	7 D28	45 %
12 - 8	13.5 "	9 D28	59 %
7 - 5	12 "	12 D28	77 %

In our structure, minimum steel governed from level 13 up.

2. Serviceability Load Design

This design load case which has (essentially) not been required previously, will now govern almost all internal beam design, and even some of what was previously considered to be "seismic dominated" members. Though the load levels are reduced, (from strength/severe loads) this is largely offset by the reduced allowable moment capacity, (0.75*Mi for the purposes of this project) the major effect is due to the fact that no moment redistribution is allowed. This means that whereas large negative moments at column faces were able to be reduced by redistributing to the positive midspan region, now, increased top steel may often be required at the column faces with a corresponding decrease in bottom steel. This limitation, whilst having little effect on the overall steel content of a beam, will tend to increase congestion in the beam-column joint region, and necessitate careful thought on the designer's part to ensure that this often congested area doesn't become even more so.

Structural Stiffness and Deflections

By inspection of the ratios of actual/allowable ("Building Deflections" in 4.1.1) it would appear that the structure could be softened significantly, with resultant savings in cost due to reduced member sizes. This has two drawbacks however, when considering the secondary effects. Firstly, reduced member size means increased P-delta actions, and this load case could easily become an overriding and impractical load criterion. The other major consideration is that of the secondary components. Under NZS4203, interstorey drifts were limited to approximately 25 mm for a typical floor (Auckland). This then was a typical value for which secondary items (glazing, external cladding etc) were detailed for interstorey movement. With the much more relaxed provisions in DZ4203, this becomes an important initial design criteria. The designer must now decide on a maximum interstorey drift which is appropriate to the proposed secondary elements, to prevent the failure of the connections and resultant collapse of these items in the event of severe seismic excitation. We used 25-30 mm as the limit for this structure, which we consider to be a realistic maximum for a typical multistorey office building.

A study should be made to determine deflection criteria at which P-delta effects become of major importance. Exceeding this criteria would require that checks and/or design of P-delta actions be mandatory, and optional otherwise.

Structural Design - General

There is a significant increase in the work required for beam design due to the increased number of load cases. Also, for those structures where vertical irregularity precludes equivalent static seismic analysis, it is required that modal (severe EQ) be combined with static (pseudo P-delta) actions. This would appear to be a manual operation, or perhaps semi-automatic at best, due to the loss of sign for the modal analysis.

With column design, account must be taken of the critical load case for the beam design. From this, the appropriate beam overstrength factor, column moments etc are used for the column design at each level and location. As the current column design procedure set out in NZS3101 Part 2 Section C3A is based on member actions which are in equilibrium, the analysis needs to be rerun with equivalent static actions (derived from first mode shears) to obtain column actions. These factors result in significant additional design work, compared to current column design methodologies.

CONCLUSIONS

The draft code DZ4203 is a major revision compared to the current code.

Wind loads are now significantly increased, seismic actions are higher for low period structures, but significantly reduced for buildings with long periods.

DZ4203 is oriented towards modal seismic analysis, rather than the equivalent static approach of the current code.

The refinement in loads means that the designer must pay close attention to details not previously considered, e.g., specific location multipliers for wind, and P-delta actions for seismic.

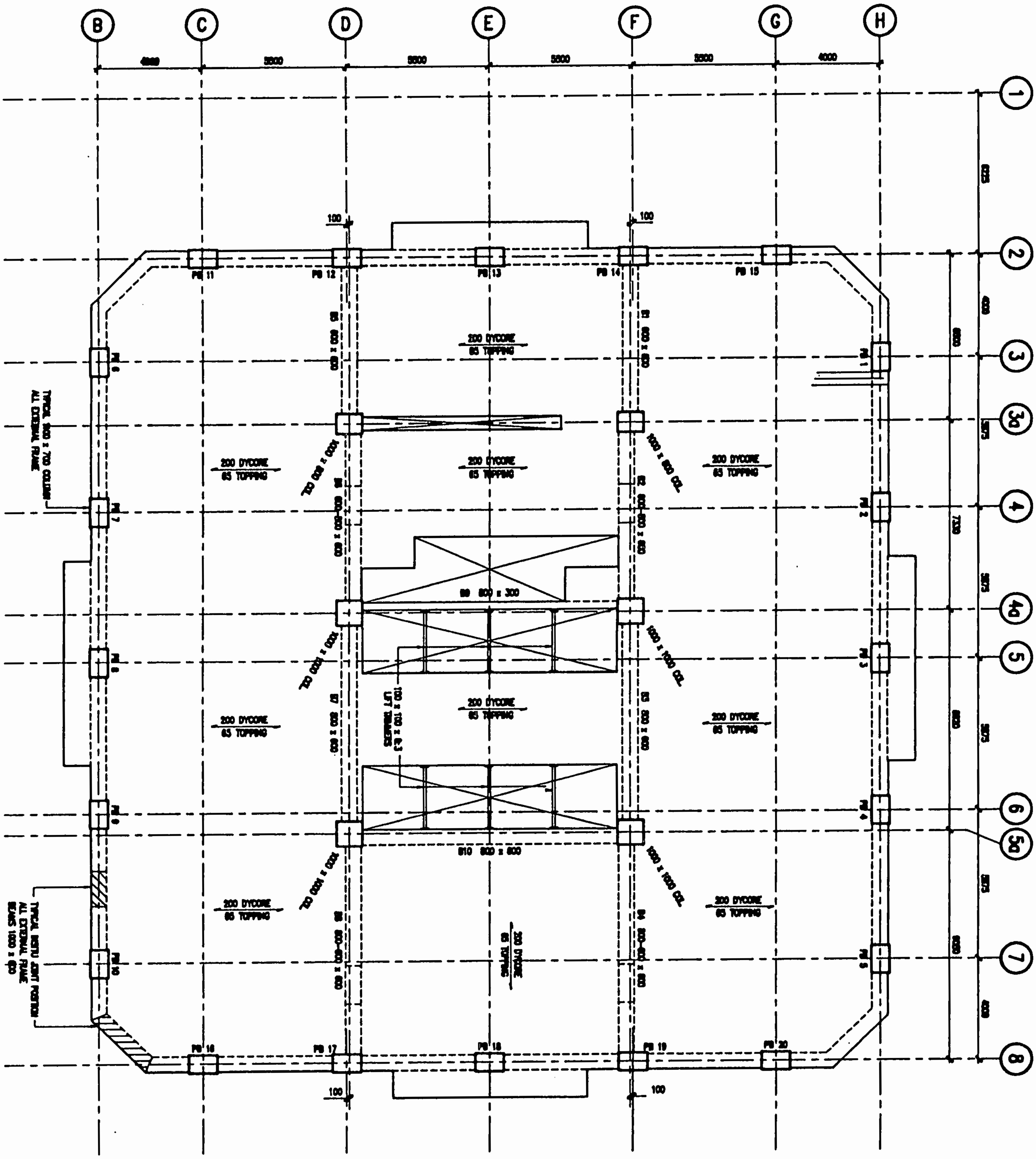
The increase in load combinations means significant additional work compared to that currently required, with no "quick and simple" alternative, except for wind loadings. Each load combination must be evaluated at each level for any member design, to obtain the critical load case. With column design, the critical load case from beam design must be carried over to enable use of the appropriate member actions and multipliers.

Deflection controls have been significantly relaxed in accordance with limiting conditions and stability criteria for the primary structure. This means that at the commencement of a project, the designer must establish allowable deflection limits appropriate to the secondary elements systems, their connections, and ability to withstand interstorey drifts without failure.

COST IMPLICATIONS

We have compared the costs of the redesigned structure with the previous design based on NZS4203. We estimate savings of approximately 5 to 6 per cent in the cost of the basic structure, which is likely to be of the order of 1.5 to 2 per cent in terms of the total cost of the building.

We note also the significant increase in design effort, which we estimate may be up to 80 per cent higher than with current codes and methodologies. This would equate to approximately 20 per cent increase in the design/documentation/supervision fee, or 0.4 per cent increase relative to the total building cost.



123 ALBERT STREET. TYPICAL FLOOR PLAN

APPENDIX A: Supplementary Notes to Material Codes

The following supplementary notes to material codes, which were provided to the participating consultants of this study, are attached:-

1. "Supplementary Notes for Reinforced Concrete"
   - L.M. Robinson

2. "List of Changes to NZS3603 for Conversion to Limit States Design Format" - G. B. Walford

3. "Recommended Tentative Strength Reduction Factors to Apply to DZ3404/A2 for use with DZ4203" - G. C. Clifton

   Note: An amended version of DZ3404/A2 was also provided to participating consultants of this study but has not been appended to the report.

BRANZ AUDIT OF DZ4203:1989 (Second Draft for Comment)

SUPPLEMENTARY NOTES FOR REINFORCED CONCRETE

1.0 GENERAL

These notes have been prepared for designers reviewing existing designs to the requirements of the proposed new Loadings Code, DZ4203, as part of the review programme to be conducted by the Building Research Association of New Zealand.

The notes are not intended to be exhaustive. Rather more they are intended to give guidance in those areas where design parameters are not readily discernible from established theory, but rely more on data obtained from testing of building elements detailed to the currently accepted standards specified in NZS3101, and to signal some warnings to designers about issues of which they need to be aware.

2.0 DESIGN REQUIREMENTS

2.1 General

The draft code now requires two principal areas to be examined in the design process. They are the serviceability limit state, and the strength limit state. For buildings subject to earthquake effects, the principles of the strength limit state are extended, with additional requirements, as the severe seismic limit state. The design criteria for each of these limit states are stated in the DZ4203. Broadly these are as follows.

2.2 Serviceability Limit State

The structure is to be designed to remain elastic. This does not mean linearly elastic, a condition inappropriate to engineering materials which all undergo creep or shrinkage related deformations to some degree. Deflections are not to exceed acceptable limits. Crack widths are not to be such as to promote corrosion or be unsightly.

2.3 Strength Limit State

DZ4203 now allows free choice of analytic procedures, for example not arbitrarily limitting the percentage of redistribution of flexural actions, provided that the designer can demonstrate that the strains imposed on the material do not exceed the useful material strain capacity. Designers not wishing to avail themselves of this freedom may follow the established rules by complying with the redistribution limits specified in NZS3101. Designers who wish to use general approaches will need to assess both the imposed strains and the strain limits.

2.4 Severe Seismic Limit State

Close attention needs to be paid to the relatively greater importance of P-delta effects, and the influence that these have on appropriate capacity design methods, an influence that can be gauged from the changes in the distribution and magnitude of shear forces in the displaced configuration, even given the distribution of the (externally applied) earthquake forces.

It is also likely that buildings may be dominated by wind forces rather than by earthquake forces. Nevertheless, capacity design for earthquake needs to be pursued to demonstrate acceptable performance in extreme earthquakes.

3.0 APPLICATION

3.1 General

Designers in reinforced concrete will be more familiar with requirements under the strength limit state. DZ3101 is in a strength limit state format, although there are serviceability considerations built, rather obscurely in places, into the design rules. Examples of direct serviceablity criteria are in 3101:4.4. Indirect examples are in the likes of 3101:3.3.4(b), intended to prevent the development of wide cracks where serviceability limit state moments exceed the dependable flexural strength -- the 0.7 factor is obtained as the inverse of the smallest partial load factor for the case 1.4D+1.7L (1/1.4 ~ 0.7). Such indirect requirements can be ignored, but their identification is left to the knowledge and experience of designers engaged in this review process.

3.2 Serviceability Limit State

A reasonable rule, fairly directly applicable, is to ensure that the tension reinforcement does not yield. (Note, however, that to limit cracks to acceptable widths the steel stress at the serviceability limit loads may need to be calculated directly -- see 3101:4.4.2). Then, provided that detailing standards are maintained, no additional conceptual problems arise.

Compliance with such a rule through "pseudo-elastic" strain compatibility can be rather time-consuming and is of little practical value for real structures, especially where creep and shrinkage are significant. Compliance might therefore be sought by ensuring that the applied limit moment does not exceed a reasonable fraction of the ideal flexural strength, where the ideal flexural strength takes into account all axial forces, including those related to shrinkage and the like, and the applied limit moment is similarly derived.

For beams and columns this is more easily achieved than for walls, especially for those walls with reinforcement distributed across the width. However, a reasonable compromise is to limit the applied moment to a defined fraction of the ideal flexural strength, for all element types (beams, columns, and walls). A fraction of 0.75 is suggested for this present review, for all elements. It is to be appreciated that compliance with a rule of this type may mean that the strength is dictated by the serviceability limit state, especially when earthquake effects dominate and the design is based on an assumed large displacement ductility.

For serviceability earthquake it should not be necessary to adjust the period to allow for the greater stiffness of the structure. The serviceability earthquake is a rather gross aproximation in any event, and the stiffnesses are not known with any great precision, especially the stiffnesses of soil systems which are strain and strain-rate dependent and usually greatly affect to the overall stiffness of the structure. However, if an assessment of period shift is required, stiffness might be based on the gross, uncracked, section properties. If the stiffness is based on the reduced section properties suggested in NZS3101 (which are thought to be too low) for the severe seismic limit state assessment, a period of about 70% of that used for the severe seismic limit state would be the likely period for the serviceability earthquake.

3.3 Strength Limit State

Imposed strains may be assessed by any established method. The most useful strain measure for flexurally yielding members is the plastic hinge rotation. For framed structures of normal proportions idealisation of hinge regions as hinges at points, and assumed elastic/perfectly plastic behaviour should be adequate to derive the likely order of plastic hinge rotations.

Useful hinge rotation capacity may be assumed as not less than the values given below for the severe seismic limit state. Monotonic hinging can achieve larger rotations, but values larger than those listed will seldom be of practical importance.

3.4 Severe Seismic Limit State

The following table lists the reliable plastic hinge rotations (plus or minus) that can be sustained during several load reversals. The values given for full ductility are derived from typical laboratory tests. Those for limited ductility have been assessed from limited testing and from the likely behaviour of members detailed to the standards of NZS3101:Chapter 14. The values for nominally elastically responding structures are fairly typical, but the actual rotation capacity can be determined from first principles and an assumed hinge length (say half the effective depth).

	Element Type		
Element Detailed For	Beam	Column	Wall
Full Ductility	0.035	0.025	0.025
Limited Ductility	0.010	0.010	0.010
Elastic Response	0.003	0.003	0.003

The values given in the table for ductile structures are believed to be conservative, but for most situations they should be adequate. Laboratory tests have achieved larger rotations, typically 0.03-0.04 for columns, but such large values will be of little use if adequate attention is also to be given to P-delta effects. The values given are plus or minus rotations, implying that rotations between these two limits can be sustained (ie 0.05 radians for a column hinge).

DZ4203 does not disallow structures with column sidesway mechanisms. However, if the redistribution methods of NZS3101 are to be used, the limitations on such mechanisms in the existing NZS4203 must be observed. Where a general analysis method is used, these limitations may be waived, but designers should appreciate that they will have extreme difficulty in proving the adequacy of multi-storey structures with respect to both plastic rotations and strength (especially under the influence of P-delta).

Designers wishing to use the capacity design approach of NZS3101:Appendix C3A will need to make some adjustments to the procedure to take adequate account of P-delta effects. Full descriptions of these adjustments and of the present limitations of the method are outside the scope of these notes.

L M Robinson
Dunedin
22 March 1989

LIST OF CHANGES TO NZS3603 FOR CONVERSION TO LIMIT STATES DESIGN FORMAT

by G.B. Walford   Date 17 April 1989

The following notes need to be read in conjunction with NZS3603:1981. They give the changes required to specific clauses.

Clause 1.1.2
This clause mentions NZS3615 and there will be consequential changes required for that standard. Also NZS3616(finger jointing), NZS3606(glulam) manufacture), NZS3618 (machine stress grading) and possibly others.

Clause 1.3.1 STRESS
Delete the definition of **BASIC WORKING STRESS** and replace with "**CHARACTERISTIC**". This is the near minimum strength defined as the lower 5 percentile value derived with 50% confidence from standard strength tests on representative timber samples.

Clause 1.3.1 PERMISSIBLE STRESS
Change the term "basic working stress" to "characteristic stress".

CARRY THIS CHANGE THROUGH THE WHOLE DOCUMENT

Clause 1.4 SYMBOLS
Change F' to F^*.

Clause 1.5.1
Change the words "alternative method" to "strength method".

Table 2
Remove factors allowing for load duration and safety by dividing the listed stresses by the following factors:-

- bending, tension and shear	0.45
- compression parallel	0.60
- compression perpendicular	0.70

The revised tables 2A and 2B are appended.

Clause 2.4.1
Replace entire clause with "The permissible stress shall be the product of the characteristic stress, F^*, the capacity reduction factor, ϕ, and those modification factors, K, given in clauses 2.6 to 2.11 inclusive that are appropriate to the service conditions".

Clause 2.4.3 (New clause)
The capacity reduction factor, ϕ, has the values given in Table 2C.

Table 3
Replace with new Table 3 appended.

Clause 2.12
Delete this clause. A new clause needs to be drafted to describe the measures necessary to achieve "elastic", "limited ductility", and "ductile" structures.

Clause 4.1.5 (New clause)
For strength limit states, the permissible load on a mechanically fastened joint shall be the product of the characteristic load, the capacity reduction factor, Ø, and those modification factors that are appropriate to the service conditions.

(A decision has yet to be made on how to handle permissible loads for fasteners under serviceability limit state loads. By analogy with the timber stresses, load/deformation curves could be used to calculate joint deformations, or the Canadian approach could be followed where permissible fastener loads for serviceability limit states are given.)

Clause 4.1.6 (New clause)
The capacity reduction factor, Ø, for mechanically fastened joints shall have the values given in Table 9A. (Should combine with 2c)

Table 11
Multiply values by 4.15

Table 13
Multiply values by 2.0

Table 14
Multiply values by 4.15

Table 15
Multiply values by 2.5

Table 17
Multiply values by 2.8

Table 19
Multiply values by 2.8

Table 20
Multiply values by 2.8

Table 22
Multiply values by 2.8

Table 23
Multiply values by 2.5

Table 28
Multiply values by 1.4

Table 34
Modify as for table 2, i.e. divide by the following factors:-

- bending, tension, shear	0.45
- compression parallel	0.60
- compression perpendicular	0.70

Table 40
Modify as above for Table 34 and Table 2.

Appendix A

- Clause A1.1 Replace

"(d) Basic working loads shall be calculated as the 5 percentile load based on a log-normal distribution."

with

"(d) Characteristic loads shall be calculated as the 5 percent lower probability limit based on a log-normal distribution."

The formulae for calculating the characteristic loads become:

Withdrawal loads:

for nails	LPL/30	N/mm
for screws	LPL/15	N/mm

Lateral loads:

category A fasteners	$LPL_2/2$	N
category B fasteners	LPL/2	N
category C fasteners	LPL_2/n	N
category D fasteners	LPL_2/n	N

Appendix E

Replace "where P is the basic lateral load for the fastener in green timber" with "where P is the product of the capacity reduction factor and the characteristic lateral load for the fastener in green timber".

Table 2A. Characteristic stresses for visually graded timber (MPa)

Species	Grade	F_b^*	F_c^*	F_t^*	F_s^*	F_p^*	E
(a) Moisture condition: Dry(16% m.c. average)							
Radiata pine	Engineering ≤150 x 50mm	20.9	14.5	10.0	2.9	4.3	10500
	Engineering >150 x 50mm	18.4	13.7	8.8	2.9	4.3	10000
	No. 1 Framing	13.3	11.8	6.4	2.9	4.3	8000
Douglas fir	Engineering ≤150 x 50mm	18.9	15.3	9.1	2.2	4.3	10400
	Engineering >150 x 50mm	16.9	14.3	8.1	2.2	4.3	9900
	Std Building	13.3	12.5	6.4	2.2	4.3	8000
Larch	Engineering	26.4	18.8	12.7	2.7	4.3	11000
	Std Building	17.1	15.3	8.3	2.7	4.3	9600
Rimu	Engineering	23.1	14.0	11.1	2.9	5.3	10900
	Building A	14.9	11.3	7.2	2.9	5.3	9500
Kahikatea	Engineering	16.9	13.7	8.1	2.2	2.9	7800
	Building A	10.9	11.9	5.2	2.2	2.9	6800
Silver beech	Engineering	27.6	17.5	13.2	2.7	3.4	10600
	Building A	17.8	14.0	8.5	2.7	3.4	9300
Red beech	Engineering	32.4	21.2	15.6	4.0	6.0	15300
	Building A	21.1	17.2	10.1	4.0	6.0	13400
Hard beech	Engineering	33.3	17.5	16.0	3.8	6.9	15500
	Building A	22.2	15.0	10.7	3.8	6.9	13600

(b) Moisture condition: Green(>25% m.c. average)

Radiata pine	Engineering ≤150 x 50mm	17.1	9.0	13.8	1.8	2.6	8800
	Engineering >150 x 50mm	15.1	8.5	12.0	1.8	2.6	8100
	No. 1 Framing	11.1	7.2	8.9	1.8	2.6	6500
Douglas fir	Engineering ≤150 x 50mm	17.1	10.3	13.8	1.8	2.3	8700
	Engineering >150 x 50mm	15.1	9.7	12.0	1.8	2.3	8000
	Std Building	11.1	8.2	8.9	1.8	2.3	6500
Larch	Engineering	17.1	12.2	13.8	2.0	2.7	8900
	Std Building	11.3	9.8	9.1	2.0	2.7	7700
Rimu	Engineering	17.1	10.0	13.8	2.0	3.3	9500
	Building A	11.3	8.2	9.1	2.0	3.3	8300
Kahikatea	Engineering	16.0	9.8	12.9	1.8	2.1	6900
	Building A	10.4	8.0	8.4	1.8	2.1	6000
Silver beech	Engineering	23.6	13.3	18.9	2.0	1.9	8600
	Building A	15.6	10.8	12.4	2.0	1.9	7500
Red beech	Engineering	28.6	12.7	22.9	2.9	3.7	13000
	Building A	18.9	10.3	15.1	2.9	3.7	11300
Hard beech	Engineering	32.2	16.6	25.8	3.3	5.1	14100
	Building A	21.3	13.7	17.1	3.3	5.1	12100

Table 2B. Characteristic stresses for mechanically graded timber (MPa)

Species	Grade	F_b^*	F_c^*	F_t^*	F_s^*	F_p^*	E
(a) Moisture condition: Dry(16% m.c. average)							
Radiata	F11 ⩽150x50mm	25.6	16.2	12.3	3.1	4.6	12000
pine	F11 >150x50mm	22.9	15.3	10.9	3.1	4.6	12000
	F6	13.3	11.8	6.4	2.9	4.3	8000
Douglas	F11 ⩽150x50mm	24.9	17.0	13.1	2.4	4.7	12000
fir	F11 >150x50mm	22.4	16.0	10.8	2.4	4.7	12000
	Std Building	13.3	12.5	6.4	2.2	4.3	8000
(b) Moisture condition: Green(>25% m.c. average)							
Radiata	F11 ⩽150x50mm	20.0	9.7	9.6	2.0	2.9	9200
pine	F11 >150x50mm	17.1	9.0	8.3	2.0	2.9	8700
	F6	11.1	7.2	5.3	1.9	2.6	6500
Douglas	F11 ⩽150x50mm	20.0	11.2	9.6	1.9	2.4	9300
fir	F11 >150x50mm	17.1	10.3	8.3	1.9	2.4	8200
	Std Building	11.1	8.2	5.3	1.8	2.3	6500

Table 2C. Capacity reduction factor, ϕ

	F_b	F_c	F_t	F_s	F_p	E
Visually graded	0.90	0.95	0.90	0.90	1.0	1.0
Mechanically graded	0.95	0.95	0.95	0.90	1.0	1.0
Glulam, plywood,	1.0	1.0	1.0	1.0	1.0	1.0
Laminated veneer lumber	1.0	1.0	1.0	1.0	1.0	1.0
Round timbers	0.90	0.95	0.90	0.95	1.0	1.0

Table 3. Duration of load factor K_1 for strength

Duration of load	Examples	K_1
Permanent	Dead and live loads that are essentially permanent such as stores (including water tanks and the like), library stacks, fixed plant, soil pressures.	0.60
Medium	Snow loads, live loads, crowd loadings, erection and maintenance loadings, concrete formwork, vehicle, pedestrian and cattle loadings.	0.80
Brief	Wind, earthquake, impact, pile driving,	1.00

Table 9A. Capacity reduction factors for mechanically fastened joints

	ϕ
Nails, screws - laterally loaded	0.90
Nails - in withdrawal	0.85
Screws - in withdrawal	0.95
Bolts, split rings, framing anchors	0.90
Nail plates,	0.95

RECOMMENDED TENTATIVE STRENGTH REDUCTION FACTORS TO APPLY TO DZ 3404/A2 FOR USE WITH DZ 4203

Written by: G C Clifton, HERA Structural Engineer
Date: 11 April 1989

1 SCOPE OF RECOMMENDATIONS

These recommendations are aimed solely at providing tentative strength reduction factors for SANZ to enable trial designs utilising DZ 4203 and DZ 3404/A2 to proceed.

DZ 3404/A2 is currently in the final process of being prepared for publications at SANZ, with publication expected in late April or early May this year. It has been written to be used in conjunction with the current loadings code, NZS 4203 1984, but in such a manner as to involve the minimum of alteration necessary to comply with DZ 4203.

The current strength reduction factors for Strength Design of steel members are 1.0 for all bare steel member design and as per NZS 3101 for composite (steel/concrete) member design, except for determining the strength of shear connectors where $\varnothing_{sc} = 0.75$ is used. These are used in conjunction with the load factors in NZS 4203 is such a manner as to ensure that the dependable member (and structure) strength equals or exceeds the factored applied loads. However the assignment of both the load and strength reduction factors has been arbitrary, based on qualified assessment confirmed by satisfactory behaviour.

The load factors proposed for DZ 4203 differ from those in NZS 4203 both in value, and in the method by which they have been derived. Their derivation is (loosely) based on the Limit State Design philosophy, which rationalises the design process by calibrating both the load and strength reduction factors to meet certain levels of reliability.

The Limit State Design procedure is briefly described in the next section and is followed, in section 3, by a brief outline of the process by which the tentative strength reduction factors given in section 4 have been derived.

Finally, some other matters regarding material properties that require revision for the undertaking of the proposed trial designs are presented in section 5, followed by the references in section 6.

2 BRIEF DETAILS OF THE GENERAL PROCEDURE USED TO DERIVE LOAD AND STRENGTH REDUCTION (MATERIAL) FACTORS FOR THE LSD PROCESS

The procedure consists of using a probabilistic safety analysis to facilitate the selection of load factors that produce desired levels of uniformity in safety which are consistant with existing general practice. It is a 4 step procedure, as outlined below:

Step 1 Estimate the level of reliability implied by the use of the current design standards for various common types of members and elements.

This involves the use of the reliability index, β as a safety measure for comparison of member and structure reliability.

Step 2 Observe the β - levels over ranges of material, limit states, nominal load ratios (eg live-to-dead, wind-to-dead), load combinations and geographical locations.

From a study of Limit State codes worldwide, it has been found that a level of β = 3.0 is consistant with average current practice for load combinations involving dead plus live loads, while β = 2.5 and β = 1.75 respectively are appropriate for combinations containing wind and earthquake loads, respectively. It is argued, eg in the NBS Special Publication 577 (1), that a higher value of β = between 4.0 and 4.5 - is more appropriate for connection components.

Step 3 Based on the observed (target) β levels, determine load factors consistant with the implied safety level and the selected safety checking format.

Step 4 Using the load factors and target β values select material (strength reduction) factors so that the target β values, are adhered to as closely as practicable for the range of variable to fixed (eg wind or live to dead) load ratios typically encountered in practice.

This process is called "code calibration" and is aimed at producing more consistant levels of reliability in the design of the members and the structure as a whole. To undertake it in full is expensive and time consuming, however in the writers' opinion it will be required at some time in the future when the new limit state steel code (DZ 3404) is calibrated to be used in conjunction with DZ 4203, which will (presumably) be the ratified loadings code by that time.

However, for the purposes of undertaking the planned trail designs to DZ 4203, the above 4 step process will be short-circuted to provide ,rapidly, tentative strength reduction factors to be used for that purpose and for initial calibration of DZ 3404/A2 to DZ 4203. Details of how these strength reduction factors have been derived are presented in section 3, with the factors themselves presented in section 4.

3 DETAILS OF THE PROCEDURE USED TO DETERMINE THE TENTATIVE STRENGTH REDUCTION FACTORS RECOMMENDED HEREIN

3.1 For the Strength Limit State

As mentioned above, a much abbreviated version of the full calibration procedure has been used for this purpose. This consists of taking the available overseas material and applying the details most appropriate to the two codes. The details relating to each of the 4 steps outlined above, and assumptions/approximations made, are presented below.

Step 1 Estimate the level of reliability in the current design procedures. The relevant current procedure is the Strength Method of Design in accordance with DZ 3404/A2 Section 14. This is to some extent a local hybrid of plastic and factored working stress design procedures, however it is reasonable to assume that the level of reliability, as it currently stands, would not be too different to that inherent in either the Australian or American working stress design procedures.

Step 2 Determine the target β values. These are taken as stated in step 2 above, as the values given therein are established world-wide values adopted for Limit State Design.

They are also similar to, or the same as, β values recommended in studies undertaken recently at the University of Canterbury by Elms et al.

<u>Step 3</u> Based on the observed (target) β values, determine appropriate load factors for the various load combinations.

Load factors are given in DZ 4203 and it is assumed that they have been derived logically in accordance with the general Limit State Design procedure outlined in section 2 above. Unfortunately, there have been no recorded details of the derivation of these factors, by the origional DZ 4203 committee, for the review committee (42/12) to study, so it cannot be stated for certain how the factors were derived.

They appear to have been largely derived in accordance with the NBS Special Publication 577 (1), which proposed the following load combinations for the strength limit state design:

$$1.4D \quad (1)$$
$$1.2D + 1.6L + 0.6\,(Lr \text{ or } S \text{ or } R) \quad (2)$$
$$1.2D + 1.6\,(Lr \text{ or } S \text{ or } R) + (0.5\,L \text{ or } 0.8W) \quad (3)$$
$$1.2D + 1.3\,W + 0.5L + 0.5\,(Lr \text{ or } S \text{ or } R) \quad (4)$$
$$1.2D + 1.5E + (0.5L \text{ or } 0.2\,S) \quad (5)$$
$$0.9D - (1.3W \text{ or } 1.5E) \quad (6)$$

where:
D = Dead Load (Ultimate)
L = Live Load (Ultimate)
Lr = Roof Live Load
S = Snow Load
R = Rain/Ice Load (excluding ponding effects)
W = Wind Load
E = Earthquake Load

The load factors proposed in DZ 4203 do differ significantly from these in terms of Snow, Wind and Earthquake loadings. The load combinations proposed in DZ 4203 for the strength limit state design are given below:

$$1.4\,D \quad (7)$$
$$1.2\,D + 1.6\,L \quad (8)$$
$$1.2\,D + 0.4L_{(1)} + W \quad (9)$$
$$1.2\,D + 0.4\,L_{(1)} + 1.2S \quad (10)$$
$$0.9\,D - W \quad (11)$$
$$1.0D + 0.4\,L_{(1,2)} + 1.0E \quad (12)$$

where: symbols as defined for equations 1 to 6.

note: (1) The value of 0.4 is the typical strength combination factor for the strength (ultimate) limit state.

(2) In this instance the live load is neglected when considering tension loading on members and overturning effects to give a load case involving earthquake equivalent to the load case involving wind represented by equation 11.

Comparing the two sets of equations, it can be seen that dead and live loads in combination are treated very similarly. However even with these combinations the roof is not treated as a separate entity in DZ 4203, although it is by NBS 577 (1), with regard to load combinations. This is not likely to be of concern in New Zealand except where snow loadings are significant, ie the closest equivalent to equation (3) of NBS 577 is equation 10 in DZ 4203. The method by which the snow load is determined and return period used in each country will be significant factors in determining the extent of any difference between the two sets of load combinations regarding reliability values.

The treatment of live load throughout all load combinations is very similar, ie a factor of 0.5 for equations 1 to 6 and a typical factor of 0.4 for equations 7 to 12. According to examples in the literature on code calibration, eg from (1, 2) the difference in live load factor would have a very small impact on reliability values.

The factors applied to wind loading are, however, very different for the two documents, with factors of 0.8 and 1.3 used in NBS 577 (1) and 1.0 used in DZ 4203. Private correspondance with Mr A King of BRANZ has indicated that the reduction of factor for equations 9 and 11, compared with equations 4 and 6, is to do with the fact that the wind loadings as derived for New Zealand conditions are modelled on the philosophy determined for Australian conditions, which is more severe than that used to derive American wind loadings. Therefore, in effect, a factor exceeding 1 is built into the design values for W, as will be used in DZ 4203, compared with those used in the relevant American codes, eg (3). (However this should mean a load factor of around 0.65, being (1/1.3) x 0.8, be applied to the wind load in equation 9 to maintain consistancy with the factor in equation (3)).

The factors applied to earthquake loading in each document are also quite different. This difference is increased when one tries to apply the argument above for the wind load to the earthquake case. It is the writers understanding that the base response spectrum, from which lower responses associated with appropriate levels of structural ductility are derived, has been obtained in the USA for the highest seismic zone corresponding to a return period of 450 years. The return period used in New Zealand spectrum determination is only 150 years, although this effectively increases to 450 years for longer periods (over about 2 seconds). There are many other factors involved in spectrum derivation, however the above would suggest that the level of New Zealand earthquake design loading, in terms of Limit State Design, has a lower inherent exceedance factor built into it than for the USA. This is contrary to the situation regarding wind loading, yet the NBS 577 equations involving earthquake loading include a factor of 1.5 (equations 5 and 6) while the DZ 4203 equation involves a factor of only 1.0 (equation 12, with or without live load). The writer is concerned that the level of reliability inherent in the use of DZ 4203 equation 12 could be significantly lower than intended as a result.

Step 4 Select the appropriate strength reduction factors. This is the most involved step and the one where the major short-circuit of the full calibration process is being undertaken.

The calibration process involves assessment of the statistical variation in all section and material properties used in the particular member design process being considered, as well as considering the variation that these cause to the design strength obtained through the use of the appropriate design process (eg the relevant design equations).

For the purpose of making these tentative recommendations on load factors,it has been assumed that the statistical variation in the relevant properties and variables used to derive the strength reduction factors in the American LRFD Specification (3), as summarised by Iwankiv (5) are applicable to DZ 3404/A2. This assumption enables the strength reduction factors derived for (3) and presented in (3, 5) to be adopted directly. These are summarised in section 4.

The validity of this assumption must be considered in two parts. Firstly, regarding statistical variation of material and section properties, it appears that the variation as derived for the LRFD calibration (eg from (1, 3, 5)) in these properties is reasonably similar to that derived for the Limit State Calibration (eg as referenced in (4)) undertaken for the draft AS 1250. The first set of data applies to American sections and material, the second to Australian sections and material. It would be reasonable to assume a similar variation in material of Japanese or NZ Steel origin and therefore the first part of the assumption is valid.

The validity of the second part of the assumption cannot be ascertained without at least a limited calibration procedure being undertaken, as the design processes in the Strength Method of Design to DZ 4203 are quite different to those in the American LRFD Specification (3). However a lack of time in which to derive the tentative strength reduction factors means that this assumption must be made in order to derive workable data for designers. There is likely to be significantly greater variation in actual β values obtained for given ratios of fixed to variable loads from the DZ 3404/A2 design requirements because most are derived simply from factored working stress design provisions and calibration analyses undertaken (1, 4) have highlighted considerable variation in the American and AS 1250 WSD provisions.

In view of the status of DZ 3404/A2 (ie only being an interim revision prior to the production of a formal limit state steel code) it would seem logical to undertake only a limited calibration of DZ 3404/A2 to make it compatible with DZ 4203.

3.2 For the Serviceability Limit State

The serviceability limit state involves primarily checking on deflection and vibration, although for earthquake loading there is requirement to check for strength, as well as stiffness.

The appropriate level of material reduction factor for the serviceability limit state strength check (and for vibration checking where required) has not been thought through to the same extent as for the strength limit state.

However a value of 1.0 would appear to be appropriate for bare steel members and connection design and for composite member design the use of the strength reduction factors as currently recommended for use in strength design to NZS 4203 is tentatively recommended.

4 TENTATIVE RECOMMENDED STRENGTH REDUCTION FACTORS TO APPLY TO DZ 3404/A2

4.1 For the Strength Limit State

The following values of $\varnothing_o$ are recommended:

Value	Application
1.0	- Bearing on pin connected members - Friction mode HSFG shear values - Web yielding (web crippling) under concentrated loads
0.9	- Tension yielding on nett area - Steel beams in bending and shear - Full penetration butt welds - Local flange bending - Doubler plate design

Value of $\varnothing_0$	Application
0.85	- Columns - Composite beams in bending in accordance with Part 2 Clause 13.4 (both positive and negative moment)
0.80	- Shear failure in elements of a connection
0.75	- Tension fracture (function of F_u) - Combined tension/shear-block failure - Edge distance, spacing, bearing strength at holes - Pin-connected members in tension or shear - Web buckling - Bolts in tension - Fillet weld stresses on effective area - Partial penetration butt weld stresses on effective area - Plug or slot welds - Bearing on milled surfaces
0.65	- Bearing on all bolts except grade 4.6
0.60	- Bearing on grade 4.6 bolts - Bearing on concrete

These values are derived from the AISC LRFD calibration work, as summarised by Iwankiv (5), with some changes in terminology made to wording more familiar to New Zealand design engineers.

A different and simpler set of strength reduction factors have been developed for the draft AS 1250 Limit State Code, these may be adopted for use with DZ 3404/A2 on the basis of a more detailed study.

4.2 For the Serviceability Limit State

Value of $\varnothing_0$	Application
1.0	For all bare steel member applications listed above
0.9	For composite beams in positive moment bending

These values have been determined from "educated gut-feeling".

5 REVISED MATERIAL FACTORS FOR USE WITH DZ 3404/A2

5.1 Material Properties

Current design practice involves the use of nominal or specified minimum material properties such as yield strength or ultimate tensile strength.

Limit state design can involve the use of either the same material properties, or the characteristic material properties, where these are available.

The approach recommended herein, at this time, is to use the characteristic yield strengths for G 250 and G 350 steel, the data for which is readily available (6) for sections ex BHP and, in the writers opinion, can be utilised for other steels of the same grades prequalified in accordance with Clause 2.2 of DZ 3404/A2.

The tensile strengths used should be the characteristic or nominal strengths as appropriate for steel.

For bolts and welds, use the nominal values.

For concrete use the characteristic strength if available.

The appropriate characteristic values for F_y and F_u to use for G250 and G350 steel are as given below, rounded to the nearest 5 MPa (6):

(1) For grade 250 steel, $F_{cy} = 270$ MPa, $F_{cu} = 450$ MPa

(2) For grade 350 steel, $F_{cy} = 360$ MPa, $F_{cu} = 495$ MPa

5.2 Overstrength Factors

The overstrength factor used in DZ 3404/A2 consists of two parts, namely a material variation component and a strain hardening component. The material variation component for both G 250 and G 350 is taken as 1.35, based on a study of all the available literature on the subject during the writing of some of the background documentation from which DZ 3404/A2 was derived.

However, for G250 and G350 sections of BHP origin the recent work by Erasmus (6) can be used to obtain much more accurate answers on the material variation component. The overstrength concept is based on a 95% confidence limit, involving determination of the range which excludes the highest and lowest 2.5% of results for each grade.

Undertaking this exercise for the data presented by Erasmus gives a material variation component of 1.19 for G250 steel and 1.24 for G350 steel. These values can be used, for primary seismic-resisting members of BHP origin only, to replace the code value of $\varnothing_{om}$ in determination of the overstrength factor, $\varnothing_{om}$ for each grade of steel in accordance with Table 12.3 of DZ 3404/A2 Part 2.

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ISSN 0113 - 3675