Under	**THE COMMISSIONS OF INQUIRY ACT 1908**
In the matter of the	**CANTERBURY EARTHQUAKES ROYAL COMMISSION OF INQUIRY INTO THE COLLAPSE OF THE CTV BUILDING**

THIRD STATEMENT OF EVIDENCE OF DOUGLAS ALEXANDER LATHAM IN RELATION TO DESIGN REVIEW

Dated 1st August 2012

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THIRD STATEMENT OF EVIDENCE OF DOUGLAS ALEXANDER LATHAM IN RELATION TO DESIGN REVIEW

1. My full name is Douglas Alexander Latham. I reside in Christchurch. I am a Structural Engineer.

2. I refer to my second statement of evidence dated 25 July 2012 for full details of my qualifications and experience. I again confirm that I have read the Code of Conduct for expert witnesses and that my evidence complies with the Code's requirements.

3. As signalled in my second statement of evidence I have carried out a design review of the secondary frames in the CTV Building, relevant to Code compliance.

4. I **attach** my report on Secondary Frame Design Review Report dated 31 July 2012.

Summary of attached Secondary Frame Design Review Report

5. In summary, my Secondary Frame Design Review Report concludes:

   (a) Consideration of Gravity Frames as Secondary Frames: It is reasonable to expect the gravity elements of the CTV building, such as the beams and columns, to be considered as secondary elements and detailed accordingly to the requirements of NZS 3101:1982 outlined in Clause 3.5.14. This was the basis of the lateral analysis carried out in my Seismic Analysis Report dated 25 July 2012.

   (b) Requirement of Ductile Detailing: Based on the equivalent static drifts determined in my Seismic Analysis Report, the additional seismic requirements of NZS 3101:1982 were not required to be satisfied, as the imposed deformations on the secondary frame elements did not result in plastic behaviour.

   (c) Column Design: The design of the columns appears to be consistent with the provisions of the required codes and standards for load combinations involving earthquake loadings.

   (d) Beam Design: The design of the beams appears to be consistent with the provisions of the required codes and standards for load combinations involving earthquake loadings.

(e) Beam Column Joint Design: The design of the beam column joints does not appear to be consistent with the provisions of the required codes and standards for load combinations involving earthquake loadings.

6. I elaborate and explain each of these conclusions in my Secondary Frame Design Review Report.

Dated this 1st day of August 2012

D A Latham

Alan Reay Consultants

SECONDARY FRAME DESIGN REVIEW REPORT:

BUILDING:

CTV Building

249 Madras Street

Christchurch

By:

Alan Reay Consultants Limited

Date: 31 July 2012

Revision: v1

Innovation
by design

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SECONDARY FRAME DESIGN REVIEW REPORT

Report Prepared By:

Doug Latham, B.E. (Hons), GIPENZ

Structural Engineer

ALAN REAY CONSULTANTS LIMITED

31 July 2012

Disclaimer: This report has been prepared solely for presentation to the Canterbury Earthquakes Royal Commission of Inquiry into the collapse of the CTV Building, and data and opinions contained in it may not be used in other contexts or for other purpose without our prior review or agreement.

Contents

1. Introduction

1.1 Scope

The scope of this report covers a review of the design of the secondary frames of the CTV building under seismic loading for the purposes of considering whether the design was consistent with the design standards and codes applicable at the time of design in 1986. A review of the frames under loading combinations other than those applicable to seismic loading, such as maximum factored gravity, wind, snow, fire etc. have not been considered in this report. No review of elements other than the secondary frames has been undertaken as part of this report.

The review has been undertaken with a 1986 context in mind. The review does not intend to replicate the original design procedures, however the basis of decisions made during the original design and the information available at the time of design have been considered and followed through where appropriate. All calculations are able to be undertaken by hand analysis.

This report should be read in conjunction with the "Seismic Analysis Report" (SAR) dated 25 July 2012 prepared by Alan Reay Consultants Limited (ARCL).

1.2 Review Procedure

The lateral seismic design forces and deflections applicable for the design of the CTV building were determined from an ETABS analysis and were presented in the "Seismic Analysis Report" (SAR) dated 25 July 2012 prepared by ARCL. It was determined in the SAR that the equivalent static method could be used for the CTV building.

These equivalent static analysis deflections, in conjunction with the relevant gravity loadings, will be used as the basis for determining the demands on the secondary frames, which will subsequently be used to determine if the design of the secondary frames was consistent with the design standards and codes applicable at the time of design in 1986.

1.3 References

This report relies on and makes reference to the following documents:

a) NZS 4203:1984 Code of practice for General Structural Design and Design Loadings for Buildings

b) NZS 3101:1982 Code of practice for the Design of Concrete Structures

c) Structural Drawings – Office Building – 249 Madras St, by ARCE dated August 1986

d) "Seismic Analysis Report" (SAR), dated 25 July 2012, prepared by ARCL

e) Structural Calculations – Office Building – 249 Madras St, by ARCE

f) DBH CTV Building Collapse Report prepared by Dr. Clark Hyland and Mr. Ashley Smith

g) New Zealand Reinforced Concrete Design Handbook – Ultimate Strength Design in accordance with NZS 3101:1982 Code of Practice for Design of Concrete Structures

h) NZS 1170.5:2004 Structural Design Actions Part 5: Earthquake Actions – New Zealand

i) NZS 3101:2006 Concrete Structures Standard, The Design of Concrete Structures

2. Basis of Secondary Frame Classification

2.1 Design Loadings Standard

The general section in the design loadings standard NZS 4203:1984 provides a definition for primary and secondary elements.

A primary element *"means elements forming part of the basic load resisting structure, such as beams, columns, diaphragms, or shear walls necessary for the building's survival when subjected to the specified loadings."*

A secondary element *"means elements such as partition walls, panels, or veneers not necessary for the survival of the building as a whole but subject to stresses due to loadings applied directly to them or to stresses induced by the deformations of the primary elements."*

It is noted that these definitions are in the general section, and are not specific towards earthquake or any other form of loading.

The general interpretation from these clauses, in absence of a more refined definition, is that any structural element forming part of a load resisting system would be considered a primary element.

2.2 Concrete Standard

The concrete standard NZS 3101:1982 does not appear to provide a formal definition for primary elements.

Clause 3.5.14 of NZS 3101:1982 provides provisions for secondary structural elements, with particular reference to seismic loading. It states, *"Secondary elements are those which do not form part of the primary seismic force resisting system, or are assumed not to form such a part and are therefore not necessary for the survival of the building as a whole under seismically induced lateral loading, but which are subjected to loads due to accelerations transmitted to them, or due to deformations of the structure as a whole."*

The commentary to this section provides further detail, and Clause C3.5.14 of NZS 3101:1982 states: *"The definition of a secondary element is more particular than that in NZS 4203, and includes such primary gravity-load resisting elements as frames which are in parallel with stiff shear walls and do not therefore participate greatly in resistance to lateral loads."*

The general interpretation from these clauses is that the primary gravity load resisting elements, such as the beams and columns, would be considered secondary structural elements, as they were not considered part of the primary seismic force resisting system, and as such were not modelled in the ETABS analysis both during the original design of the CTV building and in the analysis presented in the SAR.

2.3 *Consistency between the Loadings Standard and Concrete Standard*

The definitions in the loadings and concrete standards for secondary elements do not appear to be consistent. However, the concrete standard specifically and explicitly provides a more particular definition for a secondary element than NZS 4203, which suggests that if designing a concrete structure, it is reasonable to assume that the definition and subsequent detailing procedure provided in the concrete standard would take precedence over that in the loadings standard.

2.4 *Modern Code Definition of Secondary Element*

While not specifically relevant to the assessment of the design of the CTV building, it can be noted that the current loadings standard NZS 1170.5:2004 provides a definition for secondary members. It describes secondary members as *"Members that are not considered to be part of the earthquake resisting system and whose strength and stiffness against seismic actions is neglected. They are not required to comply with all the requirements of NZS 1170.5, but are designed and detailed to maintain support of gravity loads when subjected to the displacements caused by the seismic design condition."*

The current concrete standard NZS 3101:2006 provides a near identical definition and procedure for detailing to that of its predecessor NZS 3101:1982. In this respect, the process of classifying and detailing gravity load resisting elements has not significantly changed.

3. Gravity Loadings

3.1 General

The gravity actions on the secondary frame elements must be considered in order to assess the design of these elements under seismic loading for two main reasons:

- The secondary frames must be capable of carrying the required gravity loads under the design level seismic induced displacements of the primary lateral load resisting structure.
- The demands and capacities of the secondary frame elements are dependent on the level of gravity load applied to the elements.

3.2 Summary of Gravity Load Combinations

The structure was required to be designed for the load combinations listed in Clause 1.3.2.3 of NZS 4203:1984.

Two load combinations involve earthquake loadings and include:

- $U = 1.0D + 1.3L_R + E$ – dead, reduced live and earthquake actions
- $U = 0.9D + E$ – dead and earthquake actions

The dead (D) and reduced live (L_R) loads on the secondary frames can be determined on a tributary area basis for the area of floor or roof supported by a beam or column.

The earthquake (E) loads on the secondary frames are induced by the building deflections under the required design loadings specified by NZS 4203:1984. Refer to Section 4 of this report for the determination of these demands.

3.3 Column Axial Loads

The gravity axial load demand on the columns has been determined from a tributary area approach consistent with the masses used in the SAR and in accordance with NZS 4203:1984. The axial loads for the relevant combinations are presented in Tables 1 and 2 below:

Table 1: Gravity Axial Loads on Typical Perimeter Column (kN)

Level	D	L	L_R	0.9D	$1.0D+1.3L_R$
Level 6-roof	17.6	8.7	8.7	16	29
Level 5-6	280.3	72.9	65.0	252	365
Level 4-5	543.0	137.1	98.5	489	671
Level 3-4	805.7	201.3	129.2	725	974
Level 2-3	1068.3	265.5	158.2	961	1274
Level 1-2	1331.0	329.8	186.2	1198	1573

Table 2: Gravity Axial Loads on Typical Internal Column (kN)

Level	D	L	L_R	0.9D	1.0D+ 1.3L_R
Level 6-roof	21.2	13.1	13.1	19	38
Level 5-6	290.4	144.4	103.1	261	424
Level 4-5	559.6	275.6	163.4	504	772
Level 3-4	828.7	406.9	219.3	746	1114
Level 2-3	1097.9	538.1	272.8	988	1453
Level 1-2	1367.1	669.4	324.8	1230	1789

Refer to Appendix A for a detailed calculation of the gravity loadings.

3.4 Beam Gravity Demands

The gravity loadings on the beams have been determined from a tributary area approach consistent with the masses used in the SAR and in accordance with NZS 4203:1984. The flexural demands at critical sections along the beam have been determined based on these loadings. The flexural demands for the relevant combinations are presented in Tables 3 and 4 below:

Table 3: Perimeter Beam Demands Due to Gravity Loading

Span = 7.5m		0.9D	1.0D+1.3L_R
Demand UDL	w	25.6 kN/m + 2x18.7 kN point	38.3 kN/m + 2x20.8 kN point
Beam Moment @ Centreline	M*b	-125 kNm	-185 kNm
Beam Moment @ Column Face	M*b	-103 kNm	-153 kNm
Beam Moment @ Mid-span	M*b	60 kNm	90 kNm

Table 4: Internal Beam Demands Due to Gravity Loading

Span = 7.0m		0.9D	1.0D+1.3L_R
Demand UDL	w	33.6 kN/m	54.7 kN/m
Beam Moment @ Centreline	M*b	-137 kNm	-223 kNm
Beam Moment @ Column Face	M*b	-114 kNm	-186 kNm
Beam Moment @ Mid-span	M*b	69 kNm	112 kNm

Refer to Appendix A for a detailed calculation of the gravity loadings.

4. Earthquake Loadings

4.1 General

The secondary frames were not considered to form part of the primary seismic force resisting system. They were classified as a Group 2 secondary element under the definitions provided in NZS 3101:1982 and as such were subjected to loadings induced by the deformation of the primary lateral load resisting elements.

4.2 Drift Induced Actions on Secondary Frames

The drift induced actions on the secondary frames can be easily determined using moment-area theorem, which simplifies down to the following expression when considering a single level:

$$\Delta = \frac{V_{col}.h^3}{12EI_{col}} + \left(\frac{h}{L}\right)^2 . \frac{V_{col}.L^3}{12EI_{beam}}$$

Where:

Δ = inter-storey drift
V_{col} = induced column shear
h = inter-storey height
L = beam span
E = elastic modulus of concrete
I_{col} = second moment of area of column
I_{beam} = second moment of area of beam

Once the column shear has been determined, the column moments and beam moments can be determined from an equilibrium approach.

4.3 Effective Stiffness of Members

Consideration is required for the effective stiffness of the section to allow for the degree of cracking which occurs in the concrete member. The degree of cracking is dependent on the level of load imposed on the member. Members that are subject to high flexural demands can be expected to have a higher degree of cracking and hence have a lower effective stiffness than those with low flexural demands. Similarly, members that have low axial compression loads can be expected to have a higher degree of cracking and hence have a lower effective stiffness than those with high axial compression loads.

Equation 4-4 of NZS 3101:1982 provides a standard formula for determining the effective stiffness:

$$I_e = \left(\frac{M_{cr}}{M_a}\right)^3 I_g + \left[1 - \left(\frac{M_{cr}}{M_a}\right)^3\right] I_{cr}$$

Figures 1 to 3 show the relationship between the effective stiffness of the elements to the flexural demands and neutral axis depth for the CTV columns and beams. The figures have been generated using the above equation, and have been used to determine the effective section for the analysis of the secondary frames.

The demand along the length of a member will vary, and as such the effective stiffness will also vary. Clause 4.4.1.3 of NZS 3101:1982 stated that *"The effective moment of inertia may be taken as the average of the values obtained from eq. 4-4 for the critical positive and negative moment sections."* This approach has been adopted.

Details of the determination of the effective stiffness for the beam and column elements have been presented in Appendix B.

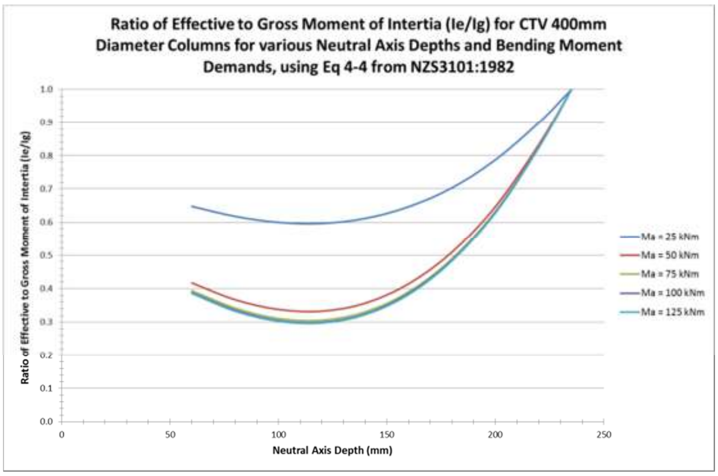


Figure 1: Effective Stiffness for Columns

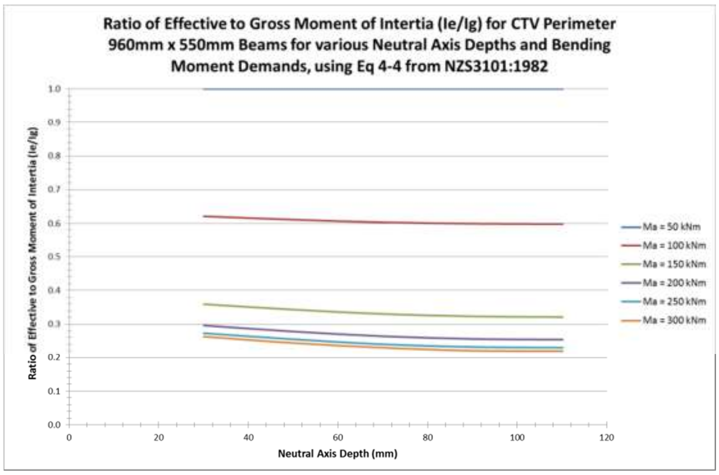


Figure 2: Effective Stiffness for Perimeter Beams

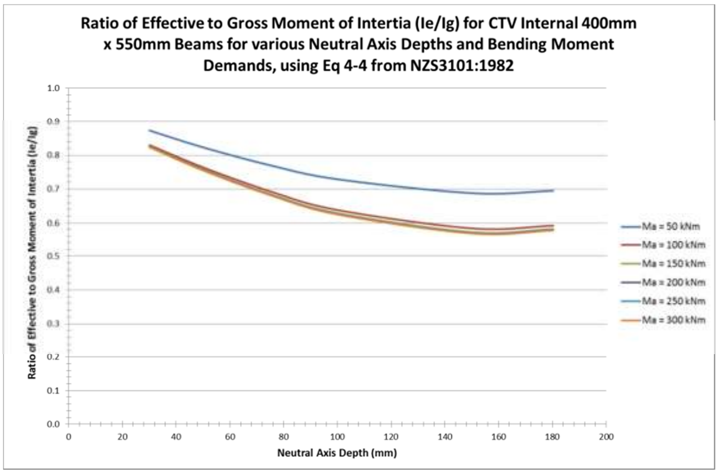


Figure 3: Effective Stiffness for Internal Beams

4.4 Beam-Column Joint Demands

The design requirements of the beam-column joints are dependent on the demands imposed on the joint from the adjoining beam and column elements. Figure 4 below, taken from NZS 3101, shows the internal forces of a typical beam-column joint.

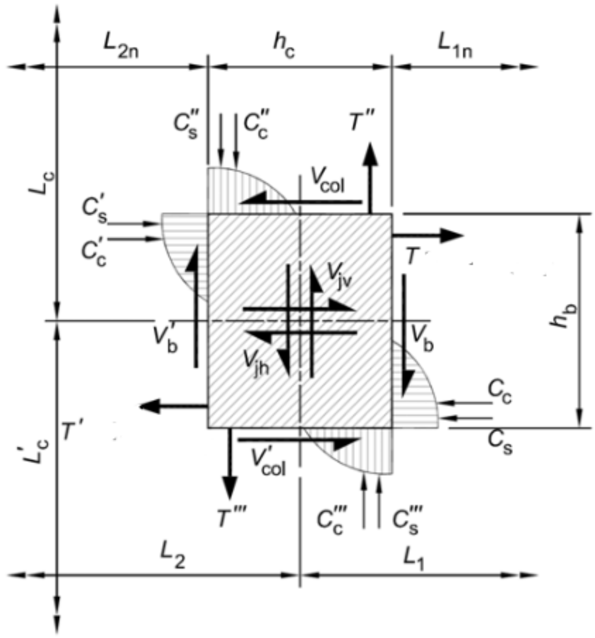


Figure 4: Beam-Column Joint Internal Forces (from NZS3101)

The design horizontal joint shear can be taken as:

Vjh = C's + C'c + T – Vcol

4.5 Perimeter Secondary Frame Design Actions

The design actions on the perimeter secondary frames have been determined based on the maximum equivalent static analysis drifts determined in the SAR. A summary of the effective stiffness assumptions along with the member and joint demands are presented in Tables 5 to 9 below:

Table 5: Perimeter Frame Actions, Levels 5-6

Design Drift = 0.46%		**0.9D+E**	**1.0D+1.3LR+E**
Axial Load (kN)	N*	252	365
Column Effective Stiffness	Ie/Ig	0.38	0.43
Beam Effective Stiffness	Ie/Ig	0.60	0.46
Column Shear (kN)	V*	52	55
Column Moment (kNm)	M*c	70	75
Beam EQ Moment (kNm)	M*b	80	85
Beam Gravity Moment (kNm)	M*b	103	153
Total Beam Moment (kNm)	M*b	183	238
Horizontal Joint Shear (kN)	V*jh	313	335

Table 6: Perimeter Frame Actions, Levels 4-5

Design Drift = 0.45%		0.9D+E	1.0D+1.3LR+E
Axial Load (kN)	N*	489	671
Column Effective Stiffness	Ie/Ig	0.50	0.62
Beam Effective Stiffness	Ie/Ig	0.58	0.45
Column Shear (kN)	V*	64	72
Column Moment (kNm)	M*c	86	97
Beam EQ Moment (kNm)	M*b	98	111
Beam Gravity Moment (kNm)	M*b	103	153
Total Beam Moment (kNm)	M*b	201	264
Horizontal Joint Shear (kN)	V*jh	386	437

Table 7: Perimeter Frame Actions, Levels 3-4

Design Drift = 0.40%		0.9D+E	1.0D+1.3LR+E
Axial Load (kN)	N*	725	974
Column Effective Stiffness	Ie/Ig	0.71	0.88
Beam Effective Stiffness	Ie/Ig	0.58	0.45
Column Shear (kN)	V*	75	83
Column Moment (kNm)	M*c	101	112
Beam EQ Moment (kNm)	M*b	115	128
Beam Gravity Moment (kNm)	M*b	103	153
Total Beam Moment (kNm)	M*b	218	281
Horizontal Joint Shear (kN)	V*jh	454	502

Table 8: Perimeter Frame Actions, Levels 2-3

Design Drift = 0.33%		0.9D+E	1.0D+1.3LR+E
Axial Load (kN)	N*	961	1274
Column Effective Stiffness	Ie/Ig	0.98	1.00
Beam Effective Stiffness	Ie/Ig	0.57	0.45
Column Shear (kN)	V*	85	80
Column Moment (kNm)	M*c	114	107
Beam EQ Moment (kNm)	M*b	130	122
Beam Gravity Moment (kNm)	M*b	103	153
Total Beam Moment (kNm)	M*b	233	275
Horizontal Joint Shear (kN)	V*jh	511	481

Table 9: Perimeter Frame Actions, Levels 1-2

Design Drift = 0.19%		0.9D+E	1.0D+1.3LR+E
Axial Load (kN)	N*	1198	1573
Column Effective Stiffness	Ie/Ig	1.00	1.00
Beam Effective Stiffness	Ie/Ig	0.60	0.46
Column Shear (kN)	V*	42	39
Column Moment (kNm)	M*c	67	61
Beam EQ Moment (kNm)	M*b	74	68
Beam Gravity Moment (kNm)	M*b	103	153
Total Beam Moment (kNm)	M*b	177	221
Horizontal Joint Shear (kN)	V*jh	297	274

Further details of the determination of the effective stiffness for the beam and column elements have been presented in Appendix B.

4.5 Internal Secondary Frame Design Actions

The design actions on the internal secondary frames have been determined based on the maximum equivalent static analysis drifts determined in the SAR. A summary of the effective stiffness assumptions along with the member and joint demands are presented in Tables 10 to 14 below:

Table 10: Internal Frame Actions, Levels 5-6

Design Drift = 0.36%		**0.9D+E**	**1.0D+1.3LR+E**
Axial Load (kN)	N*	261	424
Column Effective Stiffness	Ie/Ig	0.44	0.62
Beam Effective Stiffness	Ie/Ig	0.61	0.58
Column Shear (kN)	V*	40	50
Column Moment (kNm)	M*c	53	67
Beam EQ Moment (kNm)	M*b	60	76
Beam Gravity Moment (kNm)	M*b	114	186
Total Beam Moment (kNm)	M*b	174	262
Horizontal Joint Shear (kN)	V*jh	234	292

Table 11: Internal Frame Actions, Levels 4-5

Design Drift = 0.35%		**0.9D+E**	**1.0D+1.3LR+E**
Axial Load (kN)	N*	504	772
Column Effective Stiffness	Ie/Ig	0.71	1.00
Beam Effective Stiffness	Ie/Ig	0.61	0.58
Column Shear (kN)	V*	53	64
Column Moment (kNm)	M*c	72	86
Beam EQ Moment (kNm)	M*b	82	98
Beam Gravity Moment (kNm)	M*b	114	186
Total Beam Moment (kNm)	M*b	196	284
Horizontal Joint Shear (kN)	V*jh	316	379

Table 12: Internal Frame Actions, Levels 3-4

Design Drift = 0.31%		**0.9D+E**	**1.0D+1.3LR+E**
Axial Load (kN)	N*	746	1114
Column Effective Stiffness	Ie/Ig	1.00	1.00
Beam Effective Stiffness	Ie/Ig	0.61	0.58
Column Shear (kN)	V*	58	57
Column Moment (kNm)	M*c	78	76
Beam EQ Moment (kNm)	M*b	89	87
Beam Gravity Moment (kNm)	M*b	114	186
Total Beam Moment (kNm)	M*b	203	273
Horizontal Joint Shear (kN)	V*jh	343	336

Table 13: Internal Frame Actions, Levels 2-3

Design Drift = 0.25%		**0.9D+E**	**1.0D+1.3LR+E**
Axial Load (kN)	N*	988	1453
Column Effective Stiffness	Ie/Ig	1.00	1.00
Beam Effective Stiffness	Ie/Ig	0.61	0.58
Column Shear (kN)	V*	49	48
Column Moment (kNm)	M*c	66	65
Beam EQ Moment (kNm)	M*b	75	73
Beam Gravity Moment (kNm)	M*b	114	186
Total Beam Moment (kNm)	M*b	189	259
Horizontal Joint Shear (kN)	V*jh	291	284

Table 14: Internal Frame Actions, Levels 1-2

Design Drift = 0.19%		**0.9D+E**	**1.0D+1.3LR+E**
Axial Load (kN)	N*	1230	1789
Column Effective Stiffness	Ie/Ig	1.00	1.00
Beam Effective Stiffness	Ie/Ig	0.61	0.58
Column Shear (kN)	V*	20	20
Column Moment (kNm)	M*c	32	31
Beam EQ Moment (kNm)	M*b	35	34
Beam Gravity Moment (kNm)	M*b	114	186
Total Beam Moment (kNm)	M*b	149	220
Horizontal Joint Shear (kN)	V*jh	138	135

Further details of the determination of the effective stiffness for the beam and column elements have been presented in Appendix B.

5. Member Capacities

5.1 *Column Flexural and Shear Capacities*

The capacities of the columns have been determined in accordance with NZS 3101:1982. Capacities are based on the gravity axial loads determined in Section 3 above. Flexural capacities have been determined using the column charts from the New Zealand Reinforced Concrete Design Handbook – Ultimate Strength Design in accordance with NZS 3101:1982 Code of Practice for Design of Concrete Structures. Shear capacities have been determined using the equations provided in NZS 3101:1982. A summary of the capacities is presented in Tables 15 to 18 below:

Table 15: Typical Perimeter Column Capacity, 0.9D

Level	N* (kN)	f'c (MPa)	0.5Vc (kN)	ΦVi (kN)	ΦMn (kNm)
Level 6-roof	16	25	45	89	69
Level 5-6	252	25	55	106	85
Level 4-5	489	25	65	123	97
Level 3-4	725	25	75	140	109
Level 2-3	961	30	85	158	122
Level 1-2	1198	35	95	174	136

Table 16: Typical Perimeter Column Capacity, 1.0D+1.3L_R

Level	N* (kN)	f'c (MPa)	0.5Vc (kN)	ΦVi (kN)	ΦMn (kNm)
Level 6-roof	29	25	45	90	71
Level 5-6	365	25	59	114	80
Level 4-5	671	25	72	136	104
Level 3-4	974	25	85	157	112
Level 2-3	1274	30	97	178	128
Level 1-2	1573	35	108	197	141

Table 17: Typical Internal Column Capacity, 0.9D

Level	N* (kN)	f'c (MPa)	0.5Vc (kN)	ΦVi (kN)	ΦMn (kNm)
Level 6-roof	19	25	45	89	69
Level 5-6	261	25	55	106	84
Level 4-5	504	25	65	124	99
Level 3-4	746	25	75	141	109
Level 2-3	988	30	86	159	122
Level 1-2	1230	35	96	176	138

Table 18: Typical Internal Column Capacity, 1.0D+1.3L_R

Level	N* (kN)	f'c (MPa)	0.5Vc (kN)	ΦVi (kN)	ΦMn (kNm)
Level 6-roof	38	25	46	90	71
Level 5-6	424	25	62	118	94
Level 4-5	772	25	77	143	109
Level 3-4	1114	25	91	167	112
Level 2-3	1453	30	104	190	128
Level 1-2	1789	35	116	210	141

Further details of the determination of the member capacities have been presented in Appendix C.

5.2 *Beam Flexural Capacities*

The capacities of the beams have been determined in accordance with NZS 3101:1982, and are presented in Table 19 below:

Table 19: Beam Flexural Capacities

Beam	As (top)	As (bot)	ΦMn+ (kNm)	ΦMn- (kNm)
Perimeter 960x550 beams, at column face	3-H24 + 664 mesh	2-H24 (63%)	109	281
Perimeter 960x550 beams, midspan	4-H24 + 664 mesh	4-H24	350	298
Internal 400x550 beams, at column face	4-H28 + 664 mesh	2-H28 (54%)	112	444
Internal 400x550 beams, midspan	4-H28 + 664 mesh	4-H28	385	448

The embedment length of the bottom bars into the joint was less than the development length of bars with a standard hook specified in NZS 3101:1982. The capacity of these bars has been factored to allow for this in accordance with the procedures outlined in NZS 3101:1982.

6. Detailing Requirements

6.1 General

The secondary frames were classified as Group 2 secondary elements, as they were not detailed for separation and were therefore subject to inertia loadings and to loadings induced by the deformation of the primary elements.

The detailing requirements of Group 2 secondary elements were outlined in Clause 3.5.14.3 of NZS 3101:1982. The clause stated: *"Additional seismic requirements of this Code need not be satisfied when the design loadings are derived from the imposed deformations vΔ, specified in NZS 4203, and the assumptions of elastic behaviour. Additional seismic requirements of this Code shall be met when plastic behaviour is assumed at levels of deformation below vΔ."*

It is assumed that the members remain elastic if the demand on the member does not exceed the dependable capacity of the member. If the dependable capacity is exceeded, then plastic behaviour is assumed.

6.2 Perimeter Frames

The demands on the perimeter beams and columns and the capacities of these members have been determined in sections 4 and 5 of this report. A summary of these demands and capacities is presented in Tables 20 and 21 below:

Table 20: Perimeter Column Flexure (kNm)

	0.9D		$1.0D+1.3L_R$	
Level	**M***	**ΦMn**	**M***	**ΦMn**
Level 5-6	70	85	75	80
Level 4-5	86	97	97	104
Level 3-4	101	109	112	112
Level 2-3	114	122	107	128
Level 1-2	67	136	61	141

Table 21: Perimeter Beam Flexure (kNm)

	0.9D		$1.0D+1.3L_R$	
Level	**M***	**ΦMn**	**M***	**ΦMn**
Level 5-6	183	281	238	281
Level 4-5	201	281	264	281
Level 3-4	219	281	281	281
Level 2-3	233	281	275	281
Level 1-2	177	281	221	281

The above tables show that the flexural demand on the beam or column elements does not exceed the dependable capacity of the elements at any floor level. In accordance with NZS 3101:1982 and the secondary elements clause noted above, the additional seismic requirements did not need to be satisfied for the perimeter frame elements.

6.3 Internal Frames

The demands on the internal beams and columns and the capacities of these members have been determined in sections 4 and 5 of this report. A summary of these demands and capacities is presented in Tables 22 and 23 below:

Table 22: Internal Column Flexure (kNm)

	0.9D		$1.0D+1.3L_R$	
Level	**M***	**ΦMn**	**M***	**ΦMn**
Level 5-6	53	84	67	94
Level 4-5	72	99	86	109
Level 3-4	78	109	76	112
Level 2-3	66	122	65	128
Level 1-2	32	138	31	141

Table 23: Internal Beam Flexure (kNm)

	0.9D		$1.0D+1.3L_R$	
Level	**M***	**ΦMn**	**M***	**ΦMn**
Level 5-6	174	444	262	444
Level 4-5	196	444	284	444
Level 3-4	203	444	273	444
Level 2-3	189	444	259	444
Level 1-2	149	444	220	444

The above tables show that the flexural demand on the beam or column elements does not exceed the dependable capacity of the elements at any floor level. In accordance with NZS 3101:1982 and the secondary elements clause noted above, the additional seismic requirements did not need to be satisfied for the internal frame elements.

7. Assessment of Consistency with Design Standards

7.1 General

An assessment has been made to determine if the design of the various secondary frame elements was consistent with the requirements of NZS 3101:1982. In accordance with the conclusion reached in Section 6 previously, the additional seismic requirements of NZS 3101:1982 were not required to be satisfied. The assessment undertaken in this section is not comprehensive, and only the most relevant and critical clauses and requirements have been considered in detail.

7.2 Columns

Flexural Strength

The flexural strength has been determined in accordance with the charts provided in the New Zealand Reinforced Concrete Design Handbook – Ultimate Strength Design in accordance with NZS 3101:1982 Code of Practice for Design of Concrete Structures. A comparison of demand versus capacity is presented in Tables 24 and 25 below:

Table 24: Perimeter Column Flexure (kNm)

	0.9D		1.0D+1.3L$_R$	
Level	**M***	**ΦMn**	**M***	**ΦMn**
Level 5-6	70	85	75	80
Level 4-5	86	97	97	104
Level 3-4	101	109	112	112
Level 2-3	114	122	107	128
Level 1-2	67	136	61	141

Table 25: Internal Column Flexure (kNm)

	0.9D		1.0D+1.3L$_R$	
Level	**M***	**ΦMn**	**M***	**ΦMn**
Level 5-6	53	84	67	94
Level 4-5	72	99	86	109
Level 3-4	78	109	76	112
Level 2-3	66	122	65	128
Level 1-2	32	138	31	141

The longitudinal reinforcement of six H20 bars is sufficient to provide adequate strength to meet the design demands.

Axial Load Limits

The axial load limits have not been specifically calculated, as the earthquake load combinations have a smaller level of axial load than the maximum gravity load combinations.

Confinement

Clause 6.4.7.2 of NZS 3101:1982 outlined the requirements for the minimum column confinement. The clause stated *"Centre to centre spacing of hoop or tie sets along the member shall not exceed the smaller of the* *least lateral dimension of the cross section of the member, 16 longitudinal bar diameters, or 48 transverse bar diameters."*

Least lateral dimension of cross section = 400mm
16 longitudinal bar diameters = 320mm
48 transverse bar diameters = 288mm

The R6 transverse reinforcement spiral provided at 250mm pitch is consistent with these minimum requirements.

<u>Shear Strength</u>

Clauses 7.3.2 and 7.3.6 of NZS 3101:1982 outlined the requirements for determining the concrete and reinforcement shear capacities respectively. A comparison of demand versus capacity is presented in Tables 26 and 27 below:

Table 26: Perimeter Column Shear (kN)

	0.9D		$1.0D+1.3L_R$	
Level	**V***	**ΦVi**	**V***	**ΦVi**
Level 5-6	52	106	55	114
Level 4-5	64	123	72	136
Level 3-4	75	140	83	157
Level 2-3	85	158	80	178
Level 1-2	42	174	39	197

Table 27: Internal Column Shear (kN)

	0.9D		$1.0D+1.3L_R$	
Level	**V***	**ΦVi**	**V***	**ΦVi**
Level 5-6	40	106	50	118
Level 4-5	53	124	64	143
Level 3-4	58	141	57	167
Level 2-3	49	159	48	190
Level 1-2	20	176	20	210

The R6 transverse reinforcement spiral provided at 250mm pitch is sufficient to provide adequate strength to meet the design demands.

<u>Minimum Shear Reinforcement</u>

Clause 7.3.4 of NZS 3101:1982 outlined the requirements for the minimum area of shear reinforcement required. A minimum area of shear reinforcement was not required if the shear demand did not exceed half the concrete shear strength. A comparison of demand versus half the concrete shear capacity is presented in Tables 28 and 29 below:

Table 28: Perimeter Column Shear (kN)

	0.9D		$1.0D+1.3L_R$	
Level	**V***	**0.5Vc**	**V***	**0.5Vc**
Level 5-6	52	55	55	59
Level 4-5	64	65	72	72
Level 3-4	75	75	83	85
Level 2-3	85	85	80	97
Level 1-2	42	95	39	108

Table 29: Internal Column Shear (kN)

	0.9D		$1.0D+1.3L_R$	
Level	**V***	**0.5Vc**	**V***	**0.5Vc**
Level 5-6	40	55	50	62
Level 4-5	53	65	64	77
Level 3-4	58	75	57	91
Level 2-3	49	86	48	104
Level 1-2	20	96	20	116

A minimum area of shear reinforcement was not required.

Column Summary

The columns appear to be consistent with the requirements of NZS 3101:1982 for loading combinations involving earthquake loads.

7.3 Beams

Flexural Strength

The flexural strength of the beams has been determined in accordance with NZS 3101:1982. A comparison of demand versus capacity is presented in Tables 30 and 31 below:

Table 30: Perimeter Beam Flexure (kNm)

	0.9D		$1.0D+1.3L_R$	
Level	**M***	**ΦMn**	**M***	**ΦMn**
Level 5-6	183	281	238	281
Level 4-5	201	281	264	281
Level 3-4	219	281	281	281
Level 2-3	233	281	275	281
Level 1-2	177	281	221	281

Table 31: Internal Beam Flexure (kNm)

	0.9D		$1.0D+1.3L_R$	
Level	**M***	**ΦMn**	**M***	**ΦMn**
Level 5-6	174	444	262	444
Level 4-5	196	444	284	444
Level 3-4	203	444	273	444
Level 2-3	189	444	259	444
Level 1-2	149	444	220	444

The beam longitudinal reinforcement is sufficient to provide adequate strength to meet the design demands.

Shear Strength

Clauses 7.3.2 and 7.3.6 of NZS 3101:1982 outlined the requirements for determining the concrete and reinforcement shear capacities respectively. A comparison of demand versus capacity is presented in Tables 32 below:

Table 32: Beam Shear, Maximum over any Level (kN)

Level	$V^*_{1.0D+1.3Lr}$	V^*_E	V^*_{total}	**ΦVi**
Perimeter beams	157	36	193	348
Internal beams	181	30	211	427

The R12 stirrups provided are sufficient to provide adequate strength to meet the design demands.

Beam Summary

The beams appear to be consistent with the requirements of NZS 3101:1982 for loading combinations involving earthquake loads.

7.4 Beam Column Joints

Confinement

Clause 9.4.8 of NZS 3101:1982 outlined the requirements for the minimum joint confinement. The clause required the *"horizontal transverse confinement reinforcement in beam-column joints shall not be less than that required by 6.4.7…, but in no case shall the stirrup-tie spacing in the joint core exceed ten times the diameter of the column bar or 200mm, whichever is less."* Note that Clause 6.4.7 refers to the requirement for confinement of columns.

The R6 at 250mm pitch transverse reinforcement provided meets the confinement requirements of Clause 6.4.7 as shown in the Section 7.2 of this report, however is not consistent with the maximum spacing requirements.

Horizontal Joint Shear Strength

It is unlikely that the R6 transverse reinforcement spiral provided at 250mm pitch would have sufficient strength to meet the design demands.

Beam Column Joint Summary

The beam column joints do not appear to be consistent with the requirements of NZS 3101:1982 for loading combinations involving earthquake loads.

8. Conclusions

8.1 Consideration of Gravity Frames as Secondary Frames

As discussed in this report, it is reasonable to expect the gravity elements of the CTV building, such as the beams and columns, to be considered as secondary elements and detailed accordingly to the requirements of NZS 3101:1982 outlined in Clause 3.5.14. This was the basis of the lateral analysis carried out in the SAR.

8.2 Requirement of Ductile Detailing

As shown in this report, and based on the equivalent static drifts determined in the SAR, the additional seismic requirements of NZS 3101:1982 were not required to be satisfied, as the imposed deformations on the secondary frame elements did not result in plastic behaviour.

8.3 Column Design

As shown in this report, the design of the columns appears to be consistent with the provisions of the required codes and standards for load combinations involving earthquake loadings. Other load combinations have not been specifically reviewed as part of this report.

8.4 Beam Design

As shown in this report, the design of the beams appears to be consistent with the provisions of the required codes and standards for load combinations involving earthquake loadings. Other load combinations have not been specifically reviewed as part of this report.

8.5 Beam Column Joint Design

As shown in this report, the design of the beam column joints does not appear to be consistent with the provisions of the required codes and standards for load combinations involving earthquake loadings. Other load combinations have not been specifically reviewed as part of this report.

APPENDIX A – GRAVITY LOADING CALCULATION

Secondary Frame Gravity Loads

Refer to the Calculation of Seismic Mass presented in the SAR report for generic weights

Perimeter Frame (eg Grid 1) Typical span = 7.5m

Element	**x**	**y**	**z**	**Unit Weigł**	**Weight**	
Roof Level						
Roof rafter	7.5			0.22	1.7	
Roof (purlins, cladding)	7.5	4.62		0.14	4.9	
Roof SDL	7.5	3.425		0.15	3.9	
Cladding	7.5		1.89	0.2	2.8	
Column self		0.126	1.5	23.5	4.4	
Total Roof, D					**17.6**	
Live Load Roof	7.5	4.62		0.25	8.7	
Total Roof, L					**8.7**	
						Beam UDL Loadings
Typical Floor level						(kN/m)
Beam	7.5	0.96	0.55	23.5	93.1	12.4
200 HiBond	7.5	3.48		4.0	104.4	13.9
SDL	7.5	3.425		0.5	12.8	1.7
Spandrel	7.08		0.25	23.5	41.6	**
Cladding	7.5		1.89	0.2	2.8	0.4
Column self		0.126	2.69	23.5	7.9	N/A
Total Typical Floor, D					**262.7**	**28.4**
Live Load Floor	7.5	3.425		2.5	64.2	8.6
Total Typical Floor, L					**64.2**	**8.6**

** Spandrel panels were supported at each end and were self supporting, therefore the weight is not applied uniformly along the beam, instead is applied at each end as two point loads

Summary beam loads

	D	**L**	**ΣA**	**R**	**Lr**	**0.9D**	**1.0D+1.3Lr**
UDL	28.4	8.6	25.7	0.89	7.6	25.6	38.3
Point loads	2x20.8					2x18.7	2x20.8

Beam bending moment demands

		0.9D	**1.0D+1.3Lr**	
M*b centreline	$M = wL^2/12$	125	185	kNm
M*b column face	$M = wL^2/12 - Vdc$	103	153	kNm
M*b midspan	$M = wL^2/24$	60	90	kNm

Summary column loads

	D	**L**	**ΣA**	**R**	**Lr**	**0.9D**	**1.0D+1.3Lr**
Level 6-roof	17.6	8.7	N/A	1.00	8.7	16	29
Level 5-6	280.3	72.9	25.7	0.89	65.0	252	365
Level 4-5	543.0	137.1	51.4	0.72	98.5	489	671
Level 3-4	805.7	201.3	77.1	0.64	129.2	725	974
Level 2-3	1068.3	265.5	102.8	0.60	158.2	961	1274
Level 1-2	1331.0	329.8	128.4	0.56	186.2	1198	1573

Internal Frame (eg Grid 2 or 3) Typical span = 7.0m

Element	x	y	z	Unit Weigł	Weight		Beam UDL Loadings (kN/m)
Roof Level							
Roof rafter	7.0			0.22	1.5		
Roof	7.0	7.5		0.14	7.4		
Roof SDL	7.0	7.5		0.15	7.9		
Column self		0.126	1.5	23.5	4.4		
Total Roof, D					**21.2**		
Live Load Roof	7.0	7.5		0.25	13.1		
Total Roof, L					**13.1**		
Typical Floor level							
Beam	7.0	0.4	0.55	23.5	36.2		5.2
200 HiBond	7.0	7.1		4.0	198.8		28.4
SDL	7.0	7.5		0.5	26.3		3.8
Column self		0.126	2.69	23.5	7.9		N/A
Total Typical Floor, D					**269.2**		**37.3**
Live Load Floor	7.0	7.5		2.5	131.3		18.8
Total Typical Floor, L					**131.3**		**18.8**

Summary beam loads

	D	L	ΣA	R	Lr	0.9D	1.0D+1.3Lr
UDL	37.3	18.8	52.5	0.71	13.4	33.6	54.7

Beam bending moment demands

		0.9D	1.0D+1.3Lr	
M*b centreline	$M = wL^2/12$	137	223	kNm
M*b column face	$M = wL^2/12 - Vdc$	114	186	kNm
M*b midspan	$M = wL^2/24$	69	112	kNm

Summary column loads

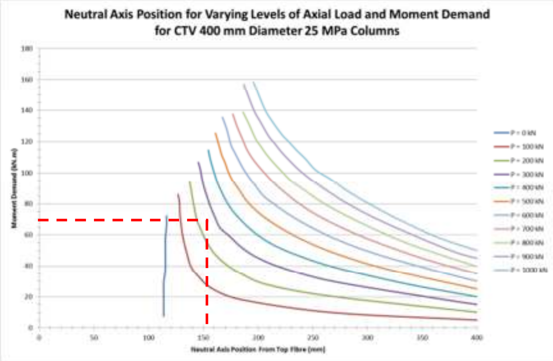
	D	L	ΣA	R	Lr	0.9D	1.0D+1.3Lr
Level 6-roof	21.2	13.1	N/A	1.00	13.1	19	38
Level 5-6	290.4	144.4	52.5	0.71	103.1	261	424
Level 4-5	559.6	275.6	105.0	0.59	163.4	504	772
Level 3-4	828.7	406.9	157.5	0.54	219.3	746	1114
Level 2-3	1097.9	538.1	210.0	0.51	272.8	988	1453
Level 1-2	1367.1	669.4	262.5	0.49	324.8	1230	1789

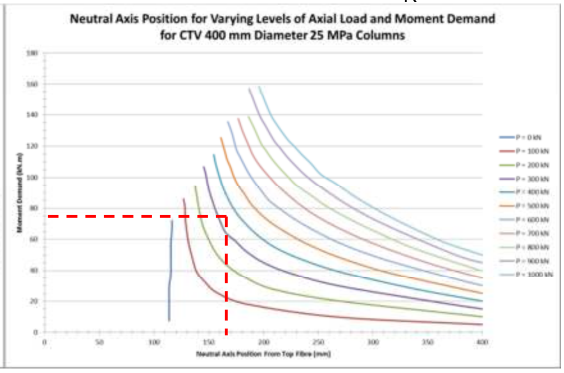
APPENDIX B – DETERMINATION OF EFFECTIVE STIFFNESS FOR ELEMENTS

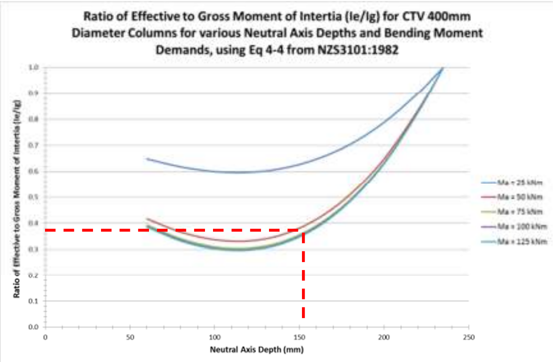
Perimeter Frame – Level 5-6

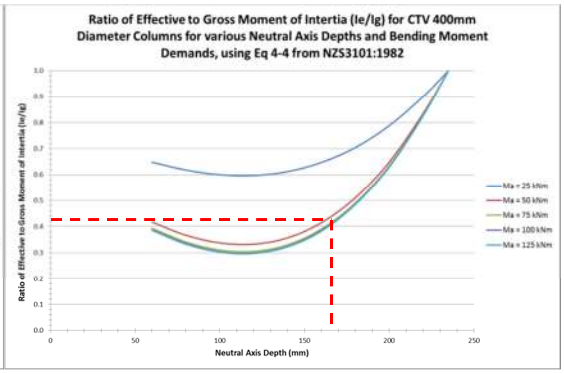
N*=0.9D

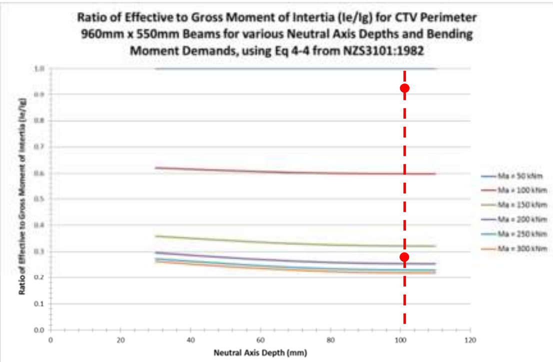
N*=1.0D+1.3L_R

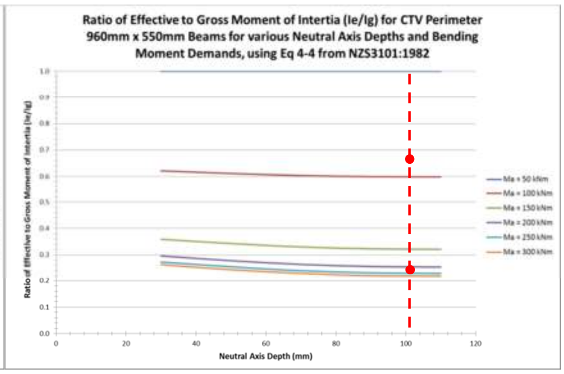






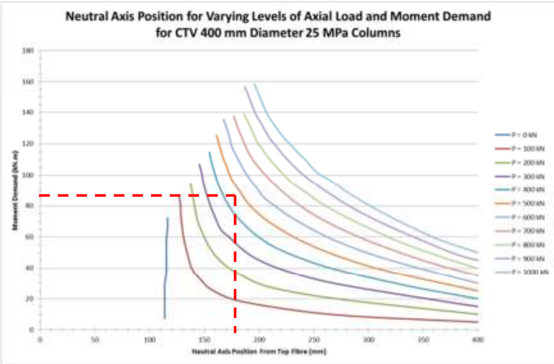


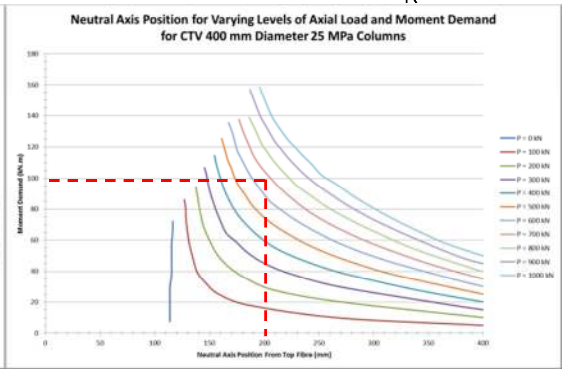


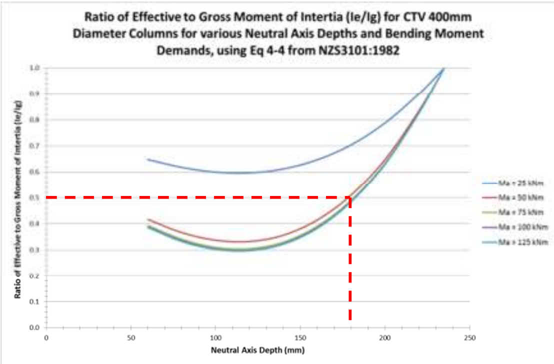


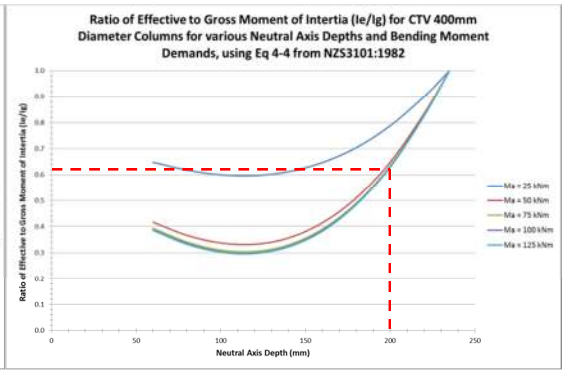
Perimeter Frame – Level 4-5

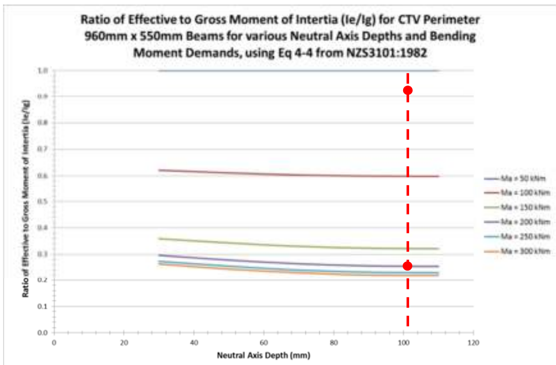
N*=0.9D | N*=1.0D+1.3L_R

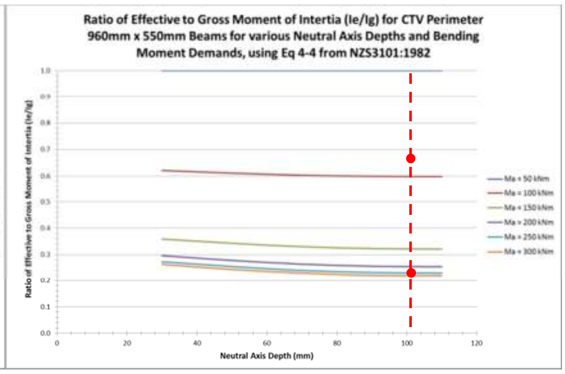






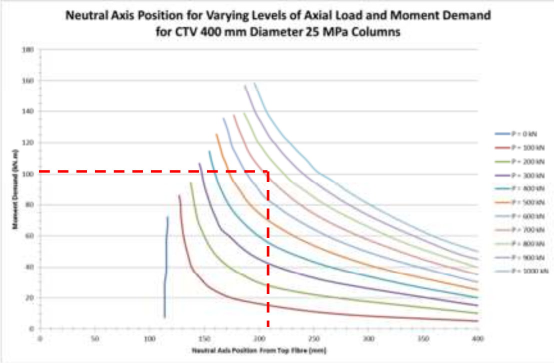


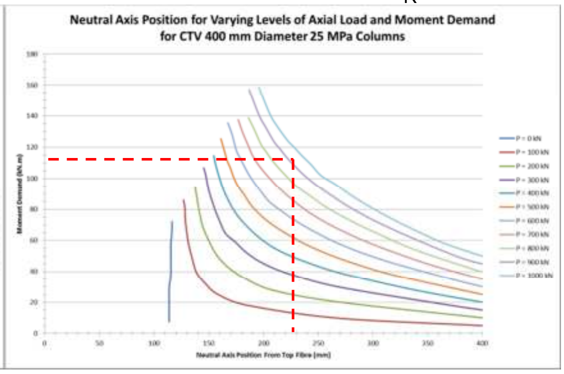


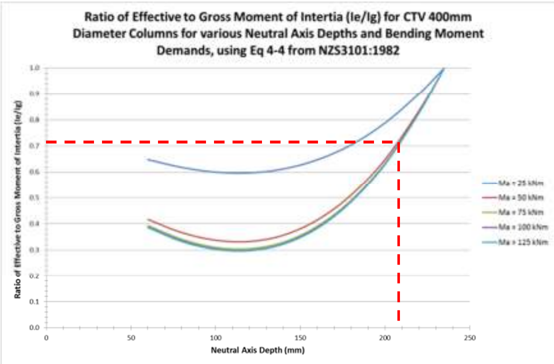


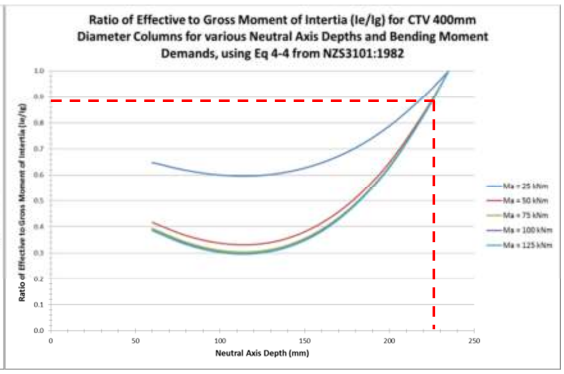
Perimeter Frame – Level 3-4

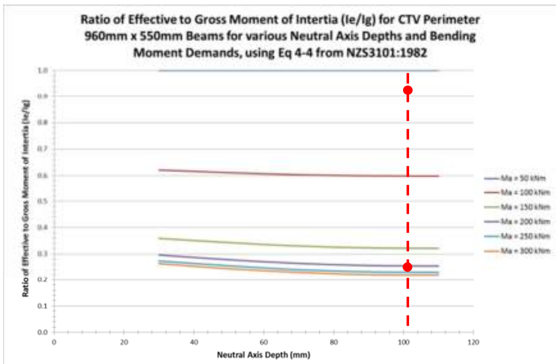
$N^*=0.9D$ | $N^*=1.0D+1.3L_R$

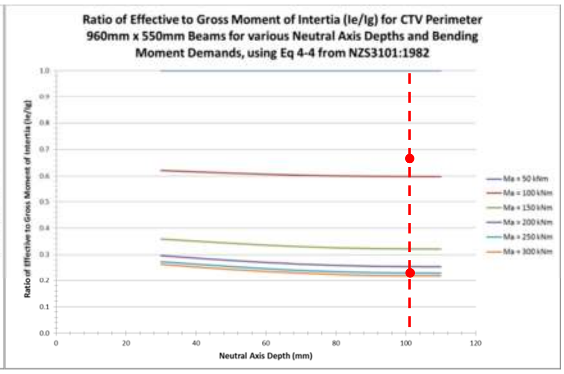








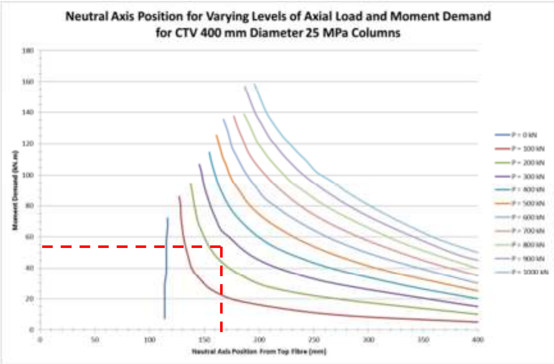


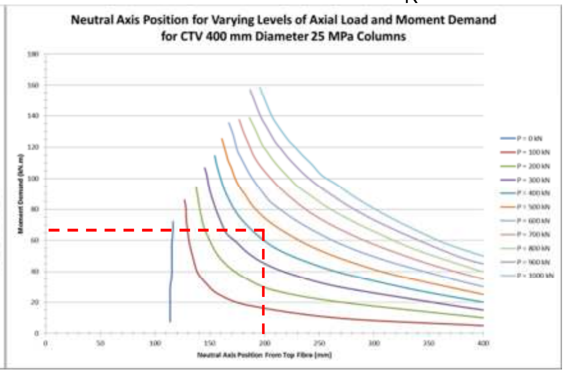


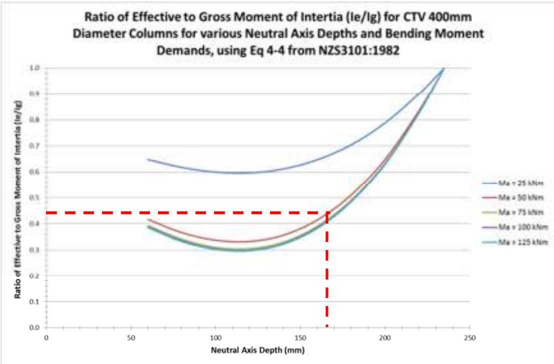
Internal Frame – Level 5-6

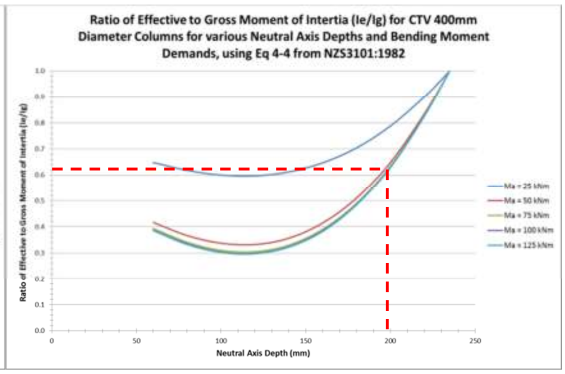
N*=0.9D

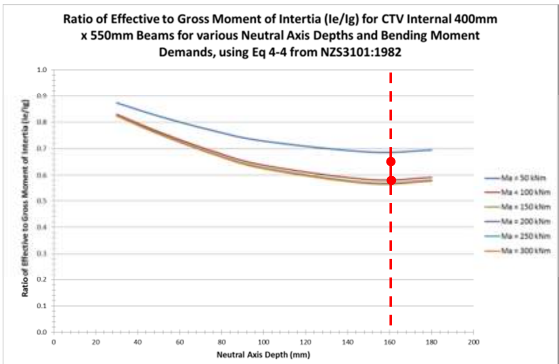
N*=1.0D+1.3L_R

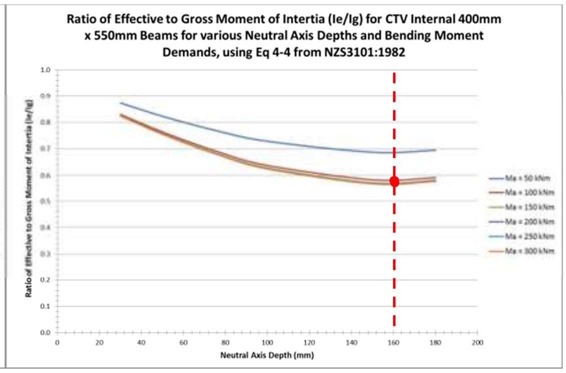












Internal Frame – Level 4-5

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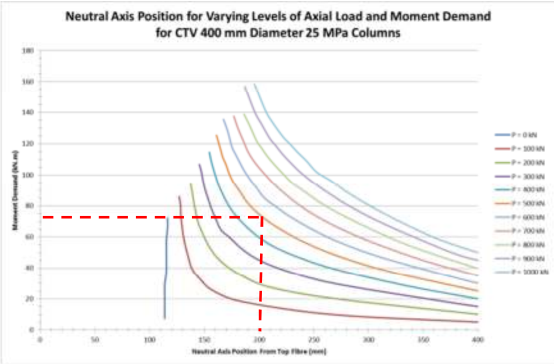
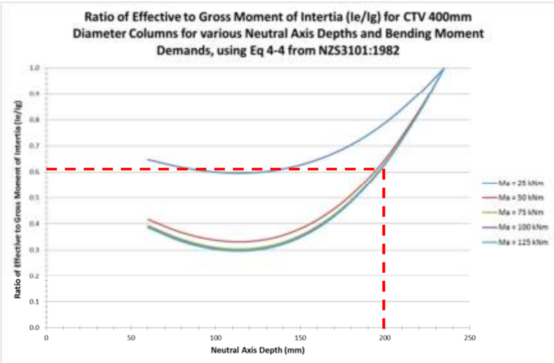
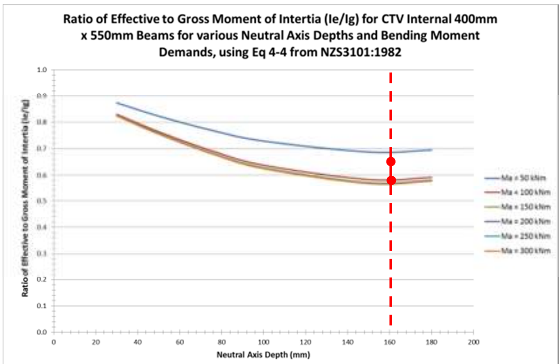
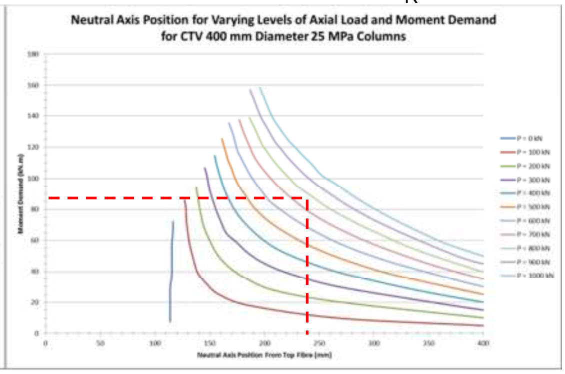
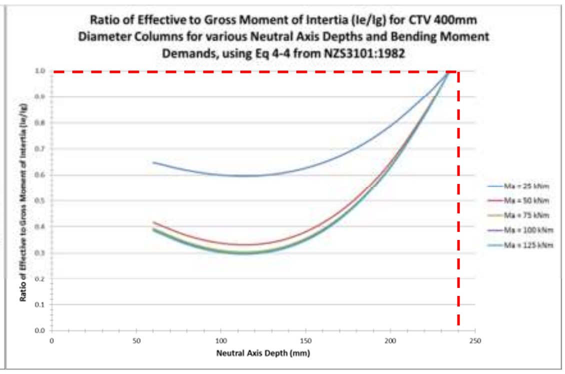


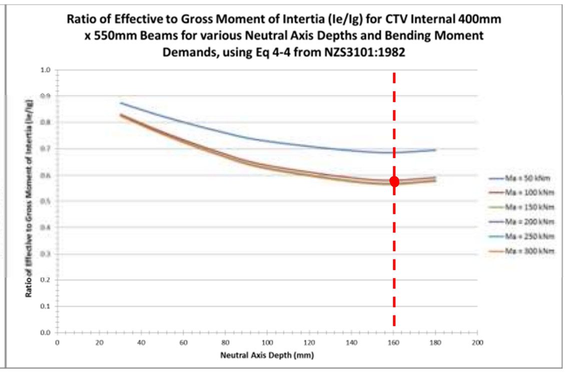




N*=1.0D+1.3L_R







APPENDIX C – MEMBER CAPACITY CALCULATION

Capacity of columns

Perimeter

	N*=0.9D	f'c	vc	Vc	0.5Vc	Vs	ΦV	Pi/f'cD2	Mi/f'cD3*	Mn	ΦMn
Level 6	16	25	0.763	90	45	15	89	0.004	0.062	99	69
Level 5	252	25	0.932	109	55	15	106	0.063	0.076	122	85
Level 4	489	25	1.102	129	65	15	123	0.122	0.087	139	97
Level 3	725	25	1.272	149	75	15	140	0.181	0.097	155	109
Level 2	961	30	1.453	170	85	15	158	0.200	0.091	175	122
Level 1	1198	35	1.616	190	95	15	174	0.214	0.087	195	136

	N*=D+1.3Lr	f'c	vc	Vc	0.5Vc	Vs	ΦV	Pi/f'cD2	Mi/f'cD3*	Mn	ΦMn
Level 6	29	25	0.772	91	45	15	90	0.007	0.063	101	71
Level 5	365	25	1.013	119	59	15	114	0.091	0.071	114	80
Level 4	671	25	1.233	145	72	15	136	0.168	0.093	149	104
Level 3	974	25	1.450	170	85	15	157	0.243	0.100	160	112
Level 2	1274	30	1.658	195	97	15	178	0.265	0.095	182	128
Level 1	1573	35	1.843	216	108	15	197	0.281	0.090	202	141

Internal

	N*=0.9D	f'c	vc	Vc	0.5Vc	Vs	ΦV	Pi/f'cD2	Mi/f'cD3*	Mn	ΦMn
Level 6	19	25	0.765	90	45	15	89	0.005	0.062	99	69
Level 5	261	25	0.939	110	55	15	106	0.065	0.075	120	84
Level 4	504	25	1.113	131	65	15	124	0.126	0.088	141	99
Level 3	746	25	1.287	151	75	15	141	0.186	0.097	155	109
Level 2	988	30	1.470	173	86	15	159	0.206	0.091	175	122
Level 1	1230	35	1.635	192	96	15	176	0.220	0.088	197	138

	N*=D+1.3Lr	f'c	vc	Vc	0.5Vc	Vs	ΦV	Pi/f'cD2	Mi/f'cD3*	Mn	ΦMn
Level 6	38	25	0.779	91	46	15	90	0.010	0.063	101	71
Level 5	424	25	1.056	124	62	15	118	0.106	0.084	134	94
Level 4	772	25	1.305	153	77	15	143	0.193	0.097	155	109
Level 3	1114	25	1.551	182	91	15	167	0.278	0.100	160	112
Level 2	1453	30	1.775	208	104	15	190	0.303	0.095	182	128
Level 1	1789	35	1.974	232	116	15	210	0.320	0.090	202	141

* Refer to attached column charts

Capacity of Beams

Perimeter

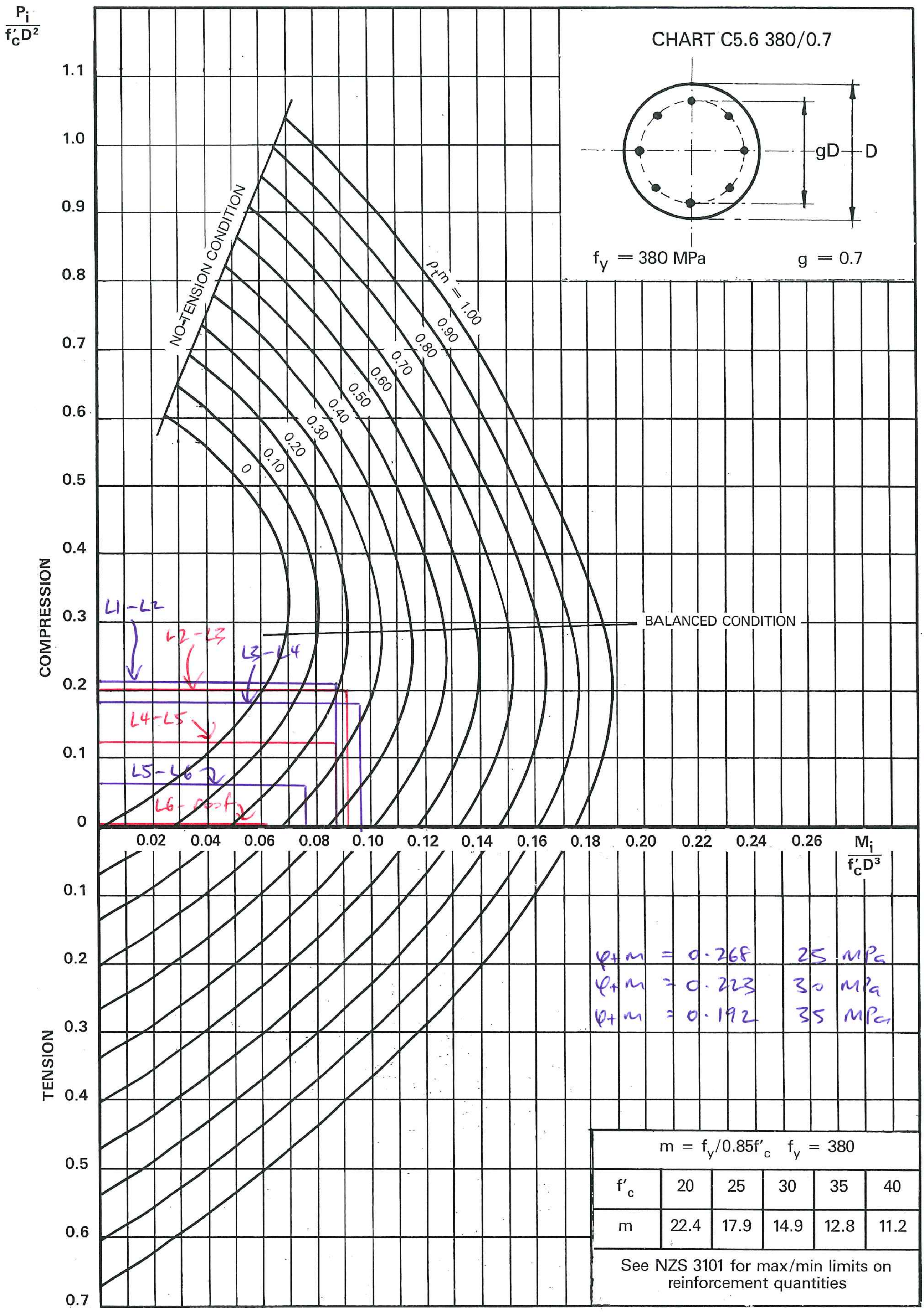
ΦMn+	109
ΦMn-	281
vb	0.54
vs	0.32
vi	0.86
Vi	409
ΦVi	348

Internal

ΦMn+	112
ΦMn-	444
vb	0.97
vs	1.55
vi	2.52
Vi	502
ΦVi	427

WIT.LATHAM.0003.38

PERIMETER COLUMNS, $N^* = 0.9D$



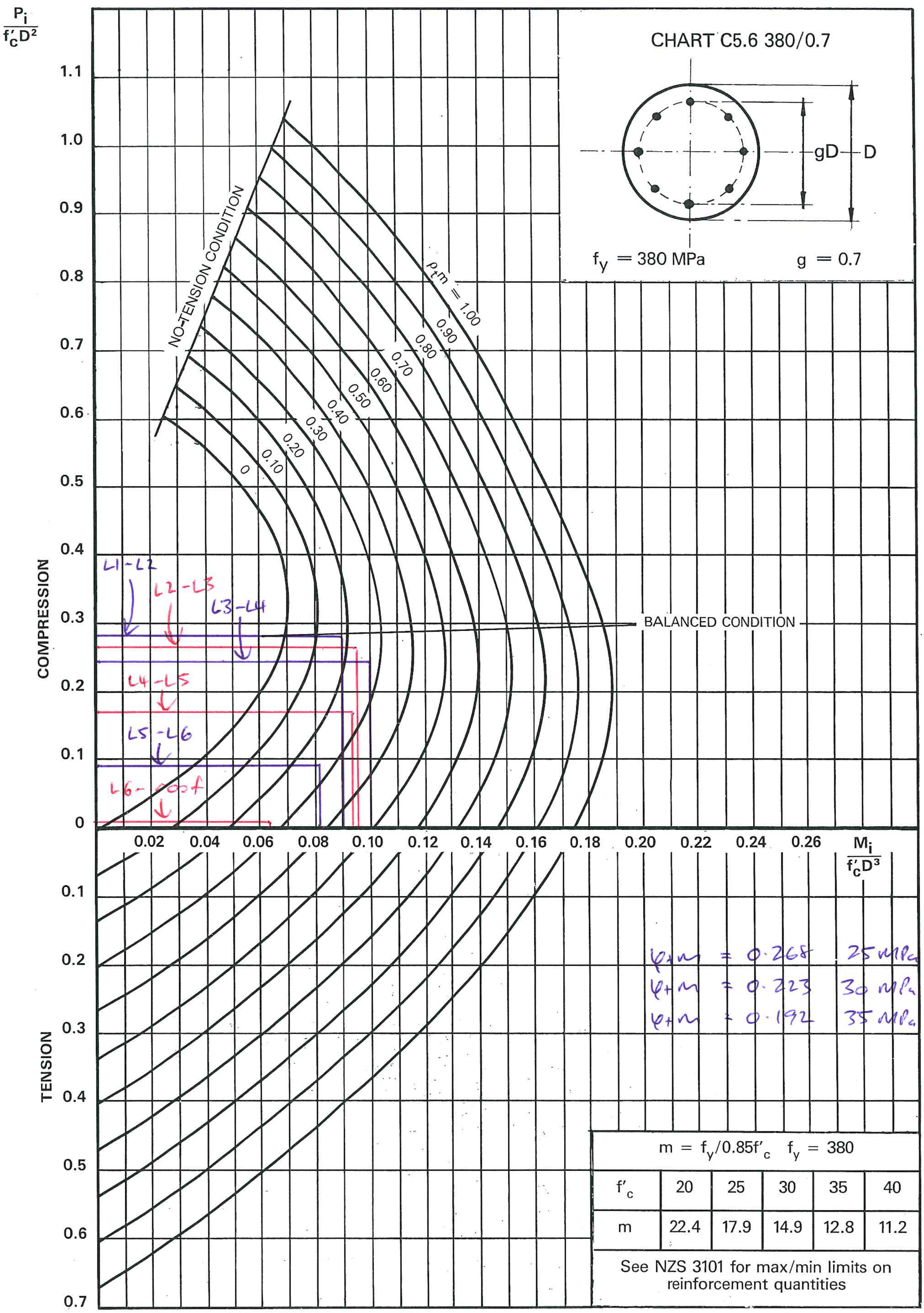
$m = f_y/0.85f'_c$ $f_y = 380$

f'_c	20	25	30	35	40
m	22.4	17.9	14.9	12.8	11.2

See NZS 3101 for max/min limits on reinforcement quantities

C5.6 COLUMN DESIGN CHART

PERIMETER COLUMNS, $N^{*} = 1{\cdot}0D + 1{\cdot}3L_R$



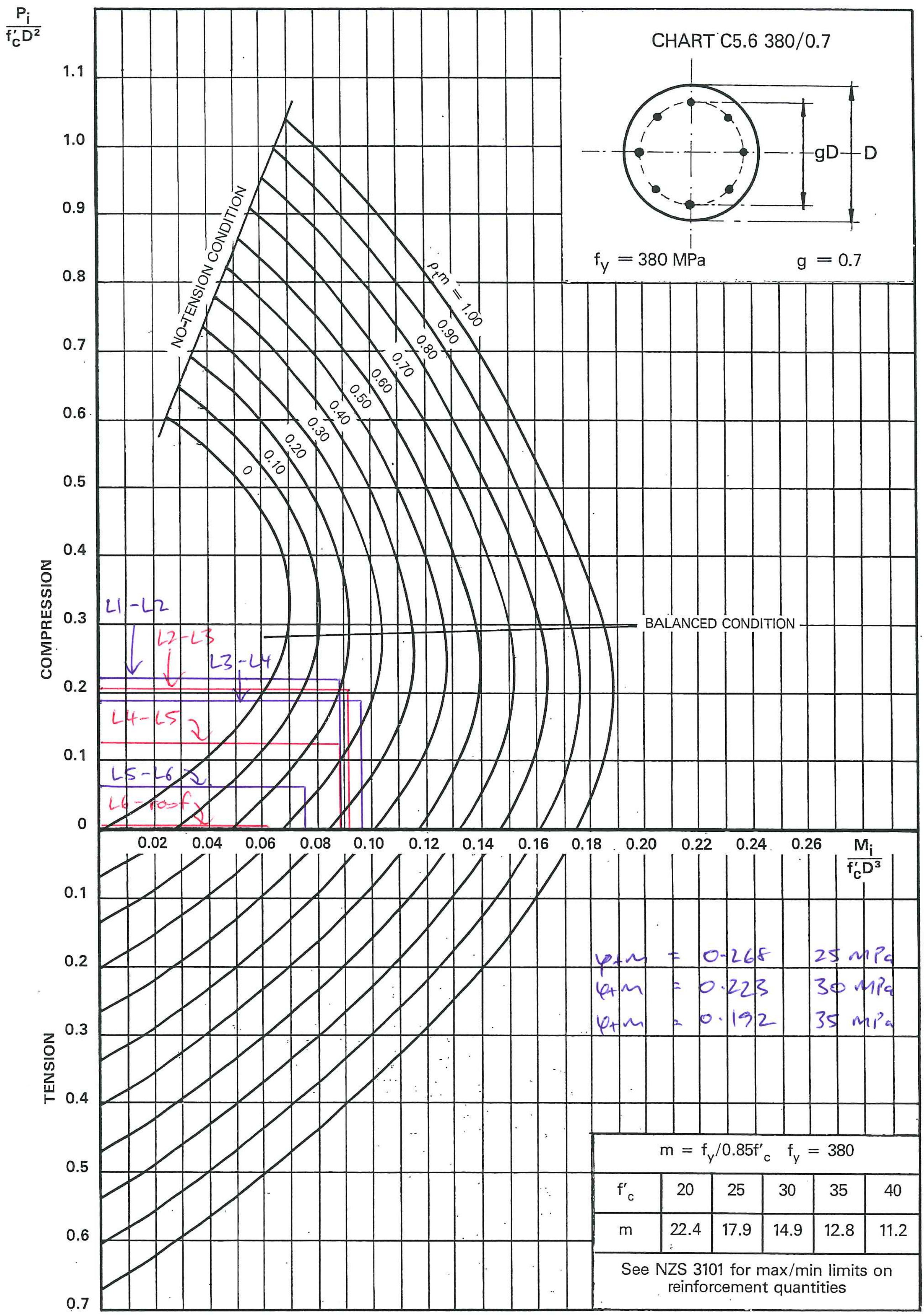
m = $f_y/0.85f'_c$ f_y = 380

f'_c	20	25	30	35	40
m	22.4	17.9	14.9	12.8	11.2

See NZS 3101 for max/min limits on reinforcement quantities

WIT.LATHAM.0003.40

INTERNAL COLUMNS, $N^* = 0.9D$

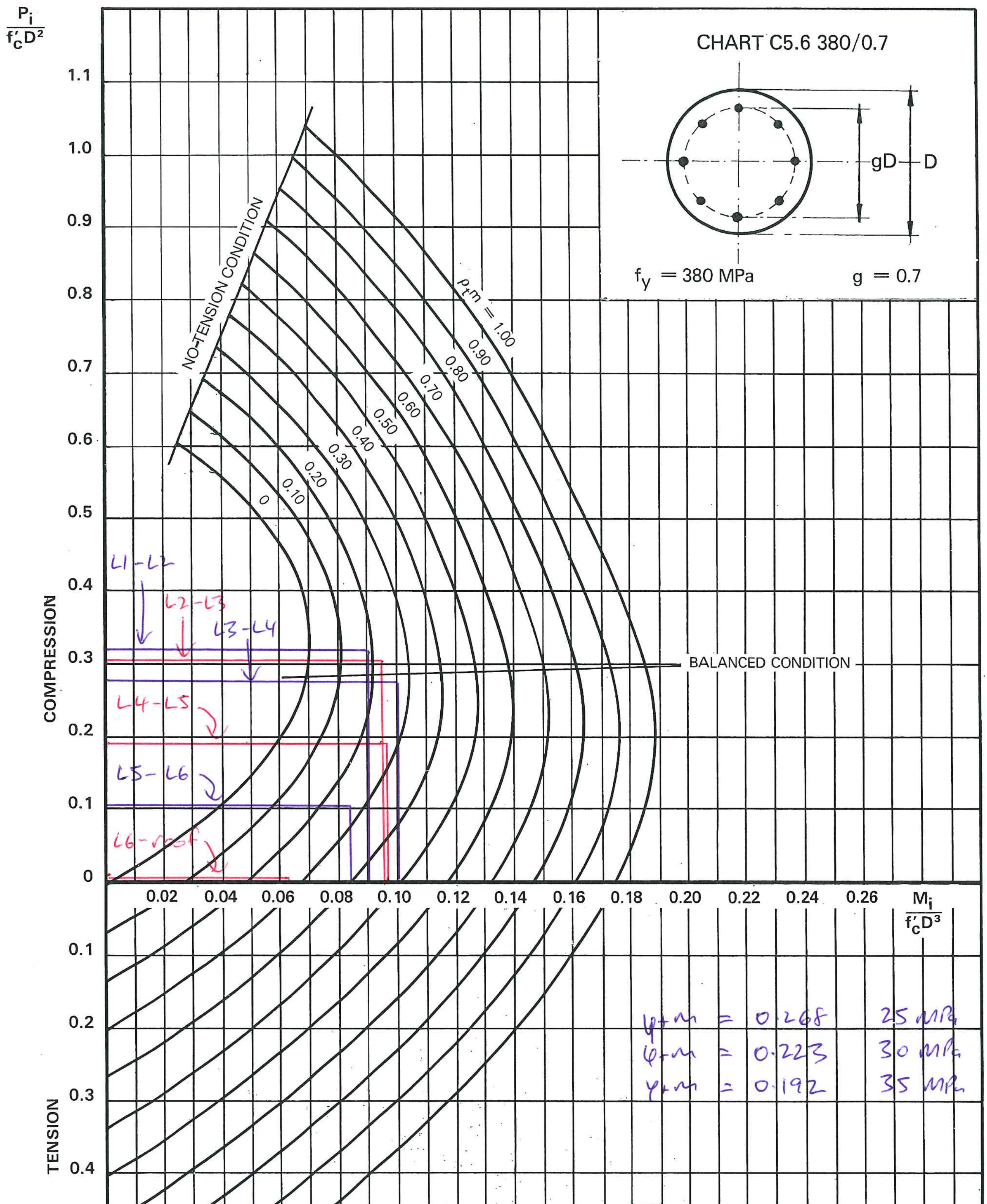


$m = f_y/0.85f'_c$ $f_y = 380$					
f'_c	20	25	30	35	40
m	22.4	17.9	14.9	12.8	11.2

See NZS 3101 for max/min limits on reinforcement quantities

C5.6 COLUMN DESIGN CHART

INTERNAL COLUMNS, $N^* = 1.0D + 1.3L_R$



m = $f_y/0.85f'_c$ f_y = 380

f'_c	20	25	30	35	40
m	22.4	17.9	14.9	12.8	11.2

See NZS 3101 for max/min limits on reinforcement quantities