UNDER THE COMMISSIONS OF INQUIRY ACT 1908

IN THE MATTER OF ROYAL COMMISSION OF INQUIRY INTO BUILDING FAILURE CAUSED BY CANTERBURY EARTHQUAKES

KOMIHANA A TE KARAUNA HEI TIROTIRO I NGA WHARE I HORO I NGA RUWHENUA O WAITAHA

AND IN THE MATTER OF THE CTV BUILDING COLLAPSE

SEVENTH STATEMENT OF EVIDENCE OF ASHLEY HENRY SMITH COMMENTS ON THE EVIDENCE OF DOUGLAS ALEXANDER LATHAM

DATE OF HEARING: COMMENCING 25 JUNE 2012

SEVENTH STATEMENT OF EVIDENCE OF ASHLEY HENRY SMITH
COMMENTS ON THE EVIDENCE OF DOUGLAS ALEXANDER LATHAM

INTRODUCTION

1. My name is Ashley Henry Smith. I live in Auckland. I am the director of StructureSmith Ltd, a consulting engineering company specialising in structural engineering.

QUALIFICATIONS AND EXPERIENCE

2. This is my Seventh Statement of Evidence. My qualifications and experience are outlined in my First Statement of Evidence dated 27 April 2012 [WIT.SMITH.0001.1].

EVIDENCE

3. This evidence comments on the Evidence of Douglas Alexander Latham [WIT.LATHAM.0002] and [WIT.LATHAM.003] and the Alan Reay Consultants Limited (ARCL) reports attached thereto. It also describes additional ERSA and displacement compatibility analyses that I have carried out to test the sensitivity of various assumptions about foundation stiffness and to calculate the elastic limit drift capacities of columns according to the codes NZS4203:1984 and NZS 3101:1982.

4. It should be noted that this evidence is my view. It does not necessarily reflect the view of the Department of Building and Housing (DBH), who engaged my company StructureSmith jointly with Hyland Consultants to investigate the CTV Building collapse; or the view of the DBH Expert Panel; or the view of my co-author for the CTV Collapse Investigation report Dr Clark Hyland.

5. I have read and agree to comply with the Code of Conduct for Expert Witnesses, a copy of which is attached and marked **"A"**.

6. I confirm that the matters I am giving evidence about are within my areas of expertise.

COMMENTS ON THE SECOND STATEMENT OF EVIDENCE OF DOUGLAS ALEXANDER LATHAM [WIT.LATHAM.0002]

ROYAL COMMISSION ERSA PANEL

PARAGRAPH 12

7. I, like Mr Latham, was also a member of the Elastic Response Spectra Analysis (ERSA) Panel constituted under the Royal Commission Order as to Directions in Relation to the Elastic Response Spectra Analysis Evidence dated 18 June 2012. The findings of that ERSA Panel are contained in a Joint Report prepared by Professor Athol Carr **[WIT.PANEL.0001.1]**.

PARAGRAPH 17(a)

8. I disagree with Mr Latham's statement that an ERSA was not required for the CTV Building design. I consider that the seismic resisting structure in the CTV building had a high degree of eccentricity and was also irregular due to the major re-entrant angles in the floor diaphragms adjacent to the north core. NZS4203:1984 clause 3.4.7.1(c) applies and therefore a three dimensional modal analysis (ERSA) was required. As noted in the ERSA Panel Joint Report the designer, Mr Harding did use a three dimensional ERSA. I have also learned from Mr Harding's evidence that the foundations were assumed to be rigid in the ERSA model that he used for design of the CTV building.

PARAGRAPH 19

9. The final paragraph in the ERSA Panel Joint Report reads as follows:

"Mr Latham requested a further ERSA analysis be undertaken using the Geotech soil stiffnesses and corrected masses. With little response from the panel to the discussions and given that different foundation stiffnesses would have compensating effects on the analyses, Dr Hyland, Mr Smith and Prof Car considered that a further ERSA was not warranted. The Compusoft ERSA input files were transmitted to ARCL so that ARCL could review the inputs and carry out further analyses with modified inputs if they wished."

10. As explained in the above extract from the ERSA Panel Joint Report, the only challenge to Compusoft's ERSA "reliability" came from Mr Latham. The majority of the Panel endorsed the Compusoft ERSA.

PARAGRAPH 20

11. I do not agree with Mr Latham's conclusion that it was feasible and appropriate for a static analysis to be run for the CTV Building, and that analysis was all that was required, as explained under paragraph 8 above.

PARAGRAPH 22 - Summary of attached Seismic Analysis report (by ARCL)

12. I do not agree with Mr Latham's conclusion (a) under paragraph 22 of his statement, as explained under paragraph 8 above.

PARAGRAPH 23

13. I do not agree with this statement by Mr Latham that the columns appear to comply with the code. As explained in my third statement of evidence, my opinion is that the CTV columns and beam column joints did not possess the required ductility, and did not contain the minimum shear reinforcement, or the minimum confinement reinforcement required by the code. In my opinion designing the columns (and the beam column joints) for at least limited ductility was required regardless of the magnitude of the design displacements.

14. As stated in paragraph 6 of my third statement of evidence:

"Interpretations of these code requirements varied between myself and Dr Clark Hyland, the co-authors of the CTV Building Collapse Investigation report dated 27 January 2012 (the Hyland/Smith report) as stated on page 12 of that report. The interpretation described on page 20 and page 109 of the Hyland/Smith report, and in the Appendix F titled 'Displacement Compatibility Analysis to Standards' was Dr Hyland's interpretation.

15. Dr Hyland assumed code recommended stiffnesses for the shear walls to calculate column drifts, but then assumed lower than code recommended stiffnesses for columns, which resulted, in my view, in an underestimate of the design column actions. Also, when calculating the capacity of the columns to withstand those design actions, Dr Hyland did not take into account the required strength reduction factor. I note this disagreement between myself and Dr Hyland relates only to the stiffness properties assumed for the columns and the capacity of the columns to withstand the code design actions. We are in agreement about the magnitude of the code displacement demands and corresponding column drifts.

16. As explained later in this evidence, I believe the methodology used by Mr Latham to determine the design column drifts is flawed and as a result he has underestimated the design drifts and corresponding column actions. I do agree, approximately, with Mr Latham's assessment of column flexural capacities.

COMMENTS ON THE ARCL SEISMIC ANALYSIS REPORT DATED 25 JULY 2012

PARAGRAPH 2.2 – MODIFICATIONS TO THE DBH MODEL

17. Regarding the four bullet pointed modifications that Mr Latham has applied to the Compusoft ERSA model, I consider that his bullet point items 2 and 3 only are significant in the present context as explained below:
   - *Seismic mass and centre of mass.* As reported under paragraph 7 of the ERSA Panel Joint Report, I had calculated the seismic mass independently and found it to differ by less than 1% from that shown in the original design calculations. Mr Latham had recently recalculated the seismic mass and he believed the mass may have been overestimated by approximately 5% in the original design. I cannot be certain of the seismic mass to the degree of accuracy that Mr Latham is now contemplating, and in my opinion neither can Mr Latham. This small potential difference in the estimated seismic mass is not significant in the present context and so I have not investigated this aspect any further.
   - *Foundation stiffness.* Mr Latham has used extremely soft foundation springs, based on data from the original Site Investigation Report that was intended for estimating long term settlements under gravity loads. As advised by Tim Sinclair of Tonkin and Taylor Limited (T&T), this data was not intended, and is not appropriate for use in dynamic analysis for earthquake loads. I note that Mr Latham's soil stiffnesses are approximately 5% of the 'likely' values that were recommended by T&T for seismic analysis purposes and would in any case, if applied correctly, lead to even greater column drifts than I have calculated herein. I have carried out further comparative ERSA analyses with the upper bound, likely and lower bound soil stiffnesses recommended T&T; and also with rigid foundations as used in the original design, to check the potential effects on seismic displacements as explained later in this statement. A summary of the modal periods, scaled base shears, centres of rigidity and point drifts from these comparative ERSA are attached and marked "**B**".
   - *Contribution of secondary elements.* In the additional ERSA that I have carried out and reported herein, I have applied moment releases to the column bases at each level, to eliminate their unintended minor contribution to lateral load

resistance. I found this correction had a negligible effect when comparing with previous results for column displacements.

- *Correction of ground floor storey height.* I consider that our original assumption, for the purposes of the analysis; that the ground floor structure height extended down to the top of the foundations is appropriate, and I would not be confident about the 'correction' carried out by Mr Latham to limit the ground floor structure height to the top of the ground floor slab. This is because, in reality, the ground slab may not have been fully effective in restraining lateral movements of the shear walls and columns.

SECTION 3 – MODELLING INPUTS

3.1 BUILDING MASS

18. Refer to comments under paragraph 17, bullet point 1 above.

3.2 BUILDING ELEMENT STIFFNESS

19. I understand the building element stiffnesses are largely unchanged from those in the Compusoft ERSA model transmitted to ARCL via the ERSA Panel. One exception is the uneven stiffness of the two piers in the south wall used by Mr Latham, which is not appropriate in my view for an ERSA where the seismic load may come from either direction. However, I expect this aspect will not affect the overall displacements to any great extent.

3.3 FOUNDATION STIFFNESS

20. Refer to my comments under paragraph 17, bullet point 2 above.

4.1 NATURAL PERIOD OF VIBRATION

21. The long modal periods of 2.07 seconds and 1.27 seconds from Mr Latham's analysis are, I believe, primarily the result of his extremely soft soil assumption which is not appropriate in my view.

4.2 CENTRE OF RIGIDITY AND 4.3 DEGREE OF ECCENTRICITY

22. I have recalculated the centre of rigidity from further comparative ERSA analyses that incorporated the upper bound, likely and lower bound soil stiffnesses recommended by T&T, and also with rigid foundations as used in the original design. I have plotted the centre of rigidity (COR) positions at level 4 together with that calculated by Mr Latham, with his extremely soft soil assumption, on the sketch plan included as page 3 of attachment "B". It can be seen from this sketch plan that the

COR positions for the upper bound and lower bound soil stiffness recommended by T&T are within a fairly narrow band and relatively close to the "fixed base" COR that assumed rigid foundations. Note that these COR positions are all outside the main floor plate, i.e. north of gridline 4.

23. The extremely soft soils assumed by Mr Latham have the effect of reducing the lateral stiffness of the north core to a larger extent than the south wall, thereby shifting the centre of rigidity closer to the middle of the floor plate. The resulting centre of rigidity position supports Mr Latham's conclusion (which is incorrect in my view) that the CTV building structure is only moderately eccentric. Mr Latham's interpretation also does not take into account the asymmetrical post-elastic effects referred to in NZS4203:1984 commentary clause C3.1.1 which states "*Geometrically dissimilar resisting elements are unlikely to develop their plastic hinges simultaneously, and ductility demand may also be increased by torsional effects.*" NZS4203:1984 clauses 1.2.5.1 and C1.2.5.1 are also relevant when considering the effects of asymmetry and foundation deformations. Commentary clause C1.2.5.1 states *"In practice the determination of relative stiffness is fraught with difficulties. The effect of foundation deformations can be considerable and should be minimised by aiming for geometrical similarity of resisting elements."*

5. EQUIVALENT STATIC ANALYSIS

24. As explained in paragraph 8 above I consider that an ERSA was required and so this Equivalent Static Analysis section is not relevant in my view.

6.1 REQUIREMENT FOR SPECTRAL MODAL ANALYSIS (ERSA)

25. Refer to my comments under paragraph 8 above.

6.5 SHEAR WALL DESIGN FORCES

26. I have not reviewed the shear wall design forces in detail; however I they are likely to have been underestimated by Mr Latham because of his extremely soft soil assumptions, which lead to the erroneous longer modal periods and corresponding reduced seismic loads.

6.6 BUILDING DEFLECTIONS

27. Mr Latham's Tables 21 to 23 show the 'ETABS Elastic Deflection' in the second column. Those values are higher than the deflections I have calculated, presumably due to the extremely soft foundation soils assumed by Mr Latham. In the second column Mr Latham has shown the 'ETABS Foundation Rotation' and he has

deducted that from the 'Elastic Deflection' before scaling by the required K/SM=2.75 factor to arrive at the 'K/SM Scaled Deflection' in the fourth column. This method is flawed because it neglects the displacement compatibility effects that the overall rotational displacements of the shear walls would have on the secondary structural elements including columns, which were on separate foundations. As a consequence of this flawed method I believe Mr Latham has underestimated the potential inter-storey drifts in the right hand column of Tables 21 to 23. My interpretation of NZS4203:1984 clause 3.8.1 is that the 'computed deformations' shall be those resulting from the application of the required horizontal actions to an analysis model that neglects foundation rotations. According to his evidence, Mr Harding did this when he carried out the ERSA for design of the CTV building. I have based my assessment of the code drift demands herein on the ERSA that I carried out with rigid foundations (i.e. with a 'fixed base') to the shear walls. This will give the smallest drift demands on columns based on the comparative analyses that I have carried out, as highlighted in the top right hand corner of the tables in attachment "**B**".

7. CONCLUSIONS

7.1 DEGREE OF ECCENTRICITY

28. Refer to my comments under paragraph 8 above.

7.2 COMPLIANCE WITH INTER-STOREY DRIFT LIMITS

29. Compliance with the overall inter-storey drift limit of 0.83% was reported in the Hyland-Smith report. I do not agree with the ERSA maximum inter-storey drift of 0.53% calculated by Mr Latham. With a fixed base ERSA model I calculated up to 0.72% drift along grid 1 in the east-west direction, which may also be accompanied by a drift up to 0.5% along grid F in the north-south direction. The maximum drifts on the columns near the corner at grid F1 could therefore be the vector sum of these two drift components or up to 0.88%.

7.3 FURTHER WORK

30. I disagree that the design forces and deflections determined by Mr Latham can be used as a basis for assessing whether structural elements were designed in accordance with relevant codes and standards, primarily because of the inappropriate soil stiffness and the flawed methodology of deducting the rotational displacements of the shear walls.

7.4 COLUMN COMPLIANCE

31. Refer to my comments under paragraphs 13 to 16 above.

COMMENTS ON THE THIRD STATEMENT OF EVIDENCE OF DOUGLAS ALEXANDER LATHAM [WIT.LATHAM.0003]

SUMMARY OF ATTACHED SECONDARY FRAME DESIGN REVIEW REPORT

5(a) CONSIDERATION OF GRAVITY FRAMES AS SECONDARY FRAMES

32. I agree that the beam and column frames could be considered to be secondary frames in accordance with NZS3101:1982 clause 3.5.14.

5(b) REQUIREMENT FOR DUCTILE DETAILING AND 5(c) COLUMN DESIGN

33. Refer to my comments under paragraphs 13 to 16 above and also my third statement of evidence.

5(d) BEAM DESIGN

34. The Hyland-Smith report did not identify the design of the beams as a design issue related to the collapse, except perhaps for the limited anchorage of beam bars into some supporting columns.

5(e) BEAM COLUMN JOINT DESIGN

35. I agree that the design of the beam column joints is not consistent with the provisions of the required codes and standards.

COMMENTS ON THE ARCL SECONDARY FRAME DESIGN REVIEW REPORT DATED 31 JULY 2012

1.2 REVIEW PROCEDURE

36. As explained in paragraphs 17 to 31 above I disagree with several aspects of the ARCL Seismic Analysis Report dated 25 July 2012 and I also do not agree with the lateral seismic design forces and deflections that were presented in that report.

2. BASIS OF SECONDARY FRAME CLASSIFICATION

37. I generally agree with the basis of classification that Mr Latham has described in paragraphs 2.1 to 2.4. Regarding the third paragraph in Mr Latham's section 2.2, I note that refers to "*... frames which are in parallel with <u>stiff</u> shear walls ...*" The word *stiff* may be a matter of contention because of the comparative flexibility of the south wall.

3. GRAVITY LOADINGS

38. The gravity loadings calculated by Mr Latham are in reasonable agreement with those that I have calculated, except that I calculated some variation in the loads on internal columns, and also some variation in the loads on perimeter columns; for example on the north and south sides compared with the east and west sides. Mr Latham's values appear to correspond, within approximately 5%, with the highest column loads that I calculated. In my analysis for determining the dependable drift capacity of the columns I used only one gravity load combination, namely 1.0D + 1.0Lr, based on NZS4203:1984 clause 3.3.2.4. The gravity loads 1.0D + 1.0Lr that I calculated would be somewhere between the two values calculated by Mr Latham using the 0.9D and 1.0D + 1.3Lr load combinations and this would have only a minor effect on the corresponding flexural capacity.

4. EARTHQUAKE LOADINGS

4.1 GENERAL

39. I agree with Mr Latham's interpretation about the classification of the secondary frames and that they were subjected to loadings induced by the deformation of the primary lateral load resisting elements.

4.2 DRIFT INDUCED ACTIONS ON SECONDARY FRAMES

40. I have calculated the drift induced actions on columns by relating the code computed deformations that were derived from my fixed base ERSA, as shown in the top right hand columns of the tables in attachment "**B**", to the displacements prescribed on two-dimensional frames as reported in Derek Bradley's email dated 31 July 2012, attached and marked "**C**". These two-dimensional frames were extracted from the Compusoft three-dimensional ERSA model, and were then subjected to the prescribed displacements and corresponding drifts noted in Mr Bradley's email. These two-dimensional frame analyses were carried out in accordance with the standard, and automatically took into account the relative flexibilities of the connecting beams and columns and the associated beam column joint rotations.

4.3 EFFECTIVE STIFFNESS OF MEMBERS

41. I have not reviewed Mr Latham's effective column stiffness calculations in detail, however I note that his Tables 5 to 14 have effective stiffness (Ie/Ig) values for columns ranging from approximately 0.4 at the top level to 1.0 at the bottom two or three levels. By comparison, I have applied the traditional simplified approach and

used Ie/Ig = 1.0 throughout, based on the recommendation in clause C3.5.5 of NZS3101:1982 which states "*... for columns carrying significant axial compression, 100% of the corresponding moment of inertia may be assumed...*" This approach may be somewhat conservative for the upper level columns if it is considered those columns carry less than 'significant' axial compression.

4.4 BEAM COLUMN JOINT DEMANDS

42. I have not reviewed Mr Harding's calculation of beam column joint demands. I note that he concluded that the beam column joints did not comply with the code, and I concur.

4.5 SECONDARY FRAME DESIGN ACTIONS SECONDARY

43. I have reviewed Mr Latham's column design actions and do not agree with the shear and bending moment demands that he has calculated, because they are based on his flawed assumptions and methodology including extremely soft foundation soils and the deduction of shear wall rotations.

5. MEMBER CAPACITIES

44. I have reviewed Mr Latham's column bending capacities, as listed in tables 15 to 18 and confirm that my calculations using similar methods are in close agreement, refer to attachment "**D**". I have not reviewed Mr Latham's column shear capacities at this stage, however as I have explained in my third statement of evidence; in my opinion at least the minimum shear reinforcement should have been provided in all the columns to ensure ductile behaviour.

6. DETAILING REQUIREMENTS

6.1 GENERAL

45. I generally agree with Mr Latham's classification of the columns as secondary elements and his assumption that the members are considered to remain elastic if demand does not exceed dependable capacity.

6.2 PERIMETER FRAMES AND 6.3 INTERNAL FRAMES

46. As explained above, I generally agree with the column flexural capacities (øMn values) listed in Mr Latham's Tables 20 and 22; however I do not agree with the column flexural demands (M* values) listed in those tables.

7. ASSESSMENT OF CONSISTENCY WITH DESIGN STANDARDS

7.2 COLUMNS

FLEXURAL STRENGTH

47. Refer to my comments under paragraph 46 above, which also apply to Mr Latham's Tables 24 and 25. In attachment "**E**", I have tabulated the computed drift demands from attachment **B,** and the dependable drift capacities from attachment **D**, and then I have calculated the corresponding demand/capacity ratios. These demand/capacity ratios range from 0.8 to 2.2 for the column at grid F2, and from 0.6 to 2.2 for the column at grid D2 (a ratio greater than 1.0 indicates that the computed drift demand exceeds the dependable flexural capacity drift of the column). At the right hand side of Tables E1 and E2 in attachment E, I have also recorded for comparison the effective column stiffnesses assumed by myself and by Mr Latham.

48. Demand/capacity ratios greater than 1.0 in Tables E1 and E2 indicate plastic behaviour at levels of deformation below $v\Delta$ as per NZS3101:1982 clause 3.5.14.3(b) and therefore the additional seismic requirements of the code should have been met.

CONFINEMENT

49. Refer to my third statement of evidence for my comments on why stronger and more closely spaced confining reinforcement (i.e. spiral) was required in the end zones of all the columns and in all the beam-column joints to ensure ductile behaviour.

MINIMUM SHEAR REINFORCEMENT

50. Refer to my third statement of evidence for explanation of my opinion that at least the minimum shear reinforcement was required over the full height of all the columns to ensure ductile behaviour.

COLUMN SUMMARY

51. I disagree with Mr Latham's statement that the columns appear to be consistent with the requirements of NZS3101:1982 for loading combinations involving earthquake loads.

7.3 BEAMS

52. I have not reviewed the beams, for the reasons explained under paragraph 34 above.

7.4 BEAM COLUMN JOINTS

53. I concur with Mr Latham's conclusion that the design of the beam column joints is not consistent with the requirements of NZS 3101:1982. My reasons are set out in my third statement of evidence.

8. CONCLUSIONS

8.1 CONSIDERATION OF GRAVITY FRAMES AS SECONDARY ELEMENTS

54. I agree with Mr Latham's classification of the frames as secondary elements, but not with the lateral analysis reported in the ARCL Seismic Analysis Report.

REQUIREMENT FOR DUCTILE DETAILING

55. Refer to my third statement of evidence for my explanation of why at least limited ductile detailing was required in the columns and beam column joints.

COLUMN DESIGN

56. I disagree with Mr Latham's statement that the design of the columns appears to be consistent with the standards for load combinations involving earthquake loadings. In my opinion the design of the columns did not comply, as explained in my third statement of evidence.

7.3 BEAMS

57. I have not reviewed the beams, for the reasons explained under paragraph 34 above.

7.4 BEAM COLUMN JOINTS

58. I concur with Mr Latham's conclusion that the design of the beam column joints is not consistent with the requirements of NZS 3101:1982. My reasons are set out in my third statement of evidence.

Signed: AH Smith

ASHLEY HENRY SMITH

Date: 3 August 2012

"A"

Statutes of New Zealand

High Court Rules

Schedule 4

Code of conduct for expert witnesses

r 9.43

Duty to the court

1 An expert witness has an overriding duty to assist the court impartially on relevant matters within the expert's area of expertise.

2 An expert witness is not an advocate for the party who engages the witness.

Evidence of expert witness

3 In any evidence given by an expert witness, the expert witness must—

(a) acknowledge that the expert witness has read this code of conduct and agrees to comply with it:

(b) state the expert witness' qualifications as an expert:

(c) state the issues the evidence of the expert witness addresses and that the evidence is within the expert's area of expertise:

(d) state the facts and assumptions on which the opinions of the expert witness are based:

(e) state the reasons for the opinions given by the expert witness:

(f) specify any literature or other material used or relied on in support of the opinions expressed by the expert witness:

(g) describe any examinations, tests, or other investigations on which the expert witness has relied and identify, and give details of the qualifications of, any person who carried them out.

4 If an expert witness believes that his or her evidence or any part of it may be incomplete or inaccurate without some qualification, that qualification must be stated in his or her evidence.

5 If an expert witness believes that his or her opinion is not a concluded opinion because of insufficient research or data or for any other reason, this must be stated in his or her evidence.

Duty to confer

6 An expert witness must comply with any direction of the court to—

(a) confer with another expert witness:

(b) try to reach agreement with the other expert witness on matters within the field of expertise of the expert witnesses:

(c) prepare and sign a joint witness statement stating the matters on which the expert witnesses agree and the matters on which they do not agree, including the reasons for their disagreement.

[7 In conferring with another expert witness, the expert witness must exercise independent and professional judgment, and must not act on the instructions or directions of any person to withhold or avoid agreement.**]**

History Note - Statutes of New Zealand

Clause 7 was substituted, as from 1 December 2009, by r 10 High Court Amendment Rules (No 2) 2009 (SR 2009/334).

History Note - Statutes of New Zealand

The High Court Rules were substituted, as from 1 February 2009, by s 8(1) Judicature (High Court Rules) Amendment Act 2008 (2008 No 90). See s 9 of that Act for the transitional provisions.

"B"

CTV BUILDING - COMPARISON OF ETABS PERIOD AND BASE SHEAR
WITH MASS OFFSET 0.1B TOWARDS SOUTH AND EAST

TABLE B1

ETABS PERIOD	DIRECTION	FIXED BASE PERIOD (sec)	TYPICAL DIRECTION u.n.o.	T&T STIFF PERIOD (sec)	T&T LIKELY PERIOD (sec)	T&T SOFT PERIOD (sec)	LATHAM / MCCAHON PERIOD (sec)
T1	Y1 (EAST-WEST)	0.93	X (NORTH-SOUTH)	1.24	1.31	1.39	2.09
T2	X (NORTH-SOUTH)	0.80	Y1 (EAST-WEST)	1.03	1.07	1.10	1.27
T3	Y2 (EAST-WEST)	0.20	Y2 (EAST-WEST)	0.32	0.35	0.40	0.84

ERSA SCALED BASE SHEAR	R=1, S=1, M=0.8	(SPEC#DUCTILE) BASE SHEAR (kN)		T&T STIFF BASE SHEAR (kN)	T&T LIKELY BASE SHEAR (kN)	T&T SOFT BASE SHEAR (kN)	LATHAM / MCCAHON BASE SHEAR (kN)
Vx	X (NORTH-SOUTH)	2715		1796	1796	1796	1712
Vy	Y (EAST-WEST)	2416		2187	2095	2026	1732

CTV BUILDING - COMPARISON OF CENTRE OF MASS (COM) AND CENTRE OF RIGIDITY (COR)
WITH MASS OFFSET 0.1B TOWARDS SOUTH AND EAST

TABLE B2

FIXED BASE

					CENTRE OF MASS						CENTRE OF RIGIDITY (COR)		ECCENTRICITY
LEVEL	Story	Diaphragm	MassX	MassY	XCM	YCM	CumMassX	CumMassY	XCCM	YCCM	XCR	YCR	XCM-XCR (m)
	STORY7	D1	72	72	24.6	10.9	72	72	24.6	10.9	27.5	12.7	
	STORY6	D1	64	64	25.0	13.0	136	136	24.8	11.9	27.0	13.4	
LEVEL 6	STORY5	D1	664	664	9.7	10.9	799	799	12.3	11.1	26.4	14.0	-16.7
LEVEL 5	STORY4	D1	631	631	10.2	10.8	1430	1430	11.3	11.0	26.2	14.0	-16.0
LEVEL 4	STORY3	D1	640	640	10.1	11.1	2070	2070	11.0	11.0	25.8	13.9	-15.6
LEVEL 3	STORY2	D1	650	650	10.1	11.3	2720	2720	10.8	11.1	24.9	13.8	-14.8
LEVEL 2	STORY1	D1	672	672	10.2	11.3	3392	3392	10.7	11.1	23.0	13.6	-12.8

T&T STIFF SOIL

LEVEL	Story	Diaphragm	MassX	MassY	XCM	YCM	CumMassX	CumMassY	XCCM	YCCM	XCR	YCR	XCM-XCR (m)
	STORY7	D1	72	72	24.6	10.9	72	72	24.6	10.9	26.9	12.5	
	STORY6	D1	64	64	25.0	13.0	136	136	24.8	11.9	26.2	13.2	
LEVEL 6	STORY5	D1	664	664	9.7	10.9	799	799	12.3	11.1	25.5	13.8	-15.7
LEVEL 5	STORY4	D1	631	631	10.2	10.8	1430	1430	11.3	11.0	25.1	13.8	-15.0
LEVEL 4	STORY3	D1	640	640	10.1	11.1	2070	2070	11.0	11.0	24.7	13.8	-14.6
LEVEL 3	STORY2	D1	650	650	10.1	11.3	2720	2720	10.8	11.1	24.0	13.8	-13.9
LEVEL 2	STORY1	D1	672	672	10.2	11.3	3392	3392	10.7	11.1	22.7	13.8	-12.5

T&T LIKELY SOIL

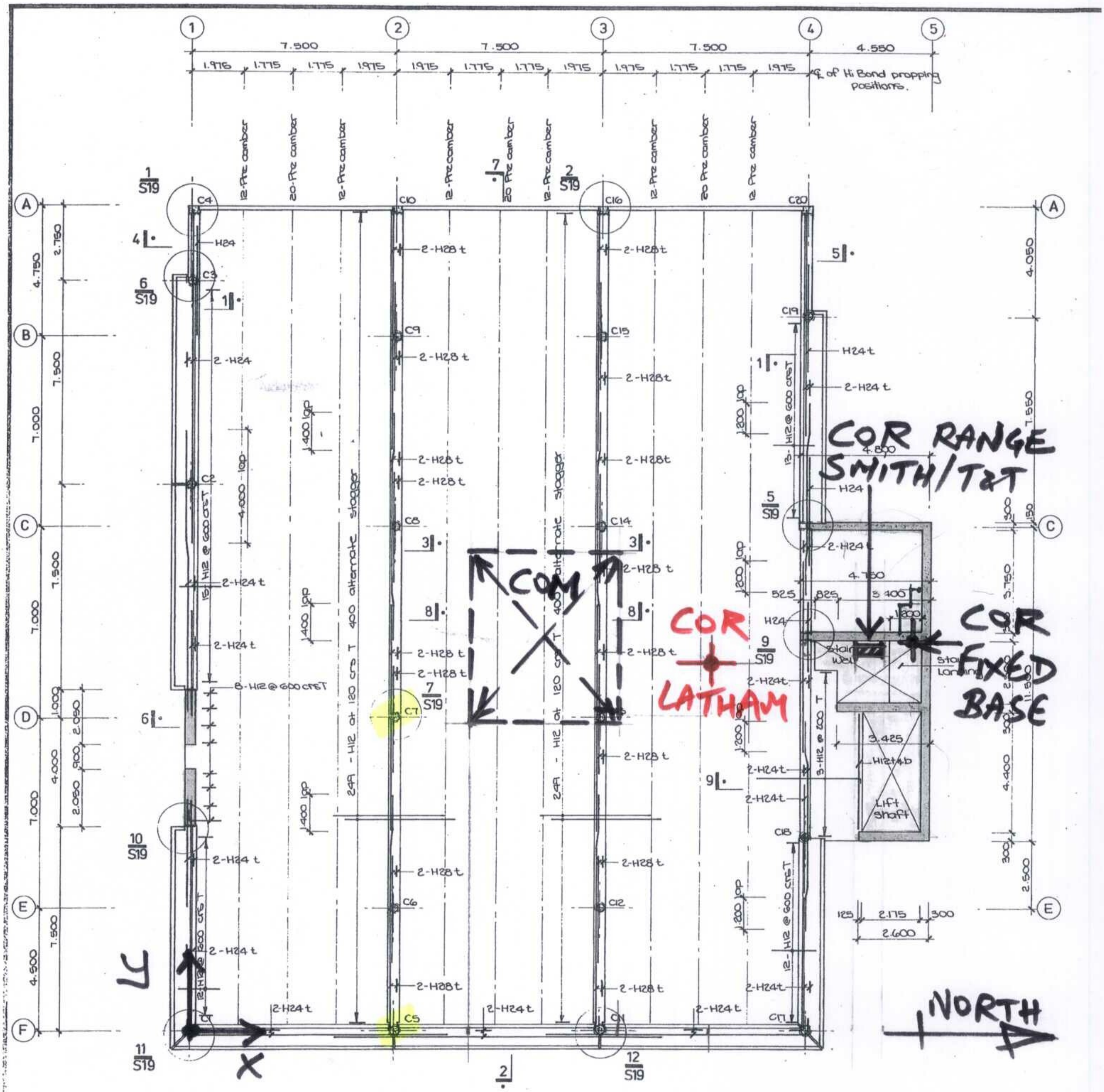
LEVEL	Story	Diaphragm	MassX	MassY	XCM	YCM	CumMassX	CumMassY	XCCM	YCCM	XCR	YCR	XCM-XCR (m)
	STORY7	D1	72	72	24.6	10.9	72	72	24.6	10.9	26.6	12.5	
	STORY6	D1	64	64	25.0	13.0	136	136	24.8	11.9	25.8	13.1	
LEVEL 6	STORY5	D1	664	664	9.7	10.9	799	799	12.3	11.1	25.1	13.7	-15.4
LEVEL 5	STORY4	D1	631	631	10.2	10.8	1430	1430	11.3	11.0	24.8	13.7	-14.6
LEVEL 4	STORY3	D1	640	640	10.1	11.1	2070	2070	11.0	11.0	24.3	13.7	-14.2
LEVEL 3	STORY2	D1	650	650	10.1	11.3	2720	2720	10.8	11.1	23.6	13.7	-13.5
LEVEL 2	STORY1	D1	672	672	10.2	11.3	3392	3392	10.7	11.1	22.4	13.7	-12.2

T&T SOFT SOIL

LEVEL	Story	Diaphragm	MassX	MassY	XCM	YCM	CumMassX	CumMassY	XCCM	YCCM	XCR	YCR	XCM-XCR (m)
	STORY7	D1	72	72	24.6	10.9	72	72	24.6	10.9	26.2	12.5	
	STORY6	D1	64	64	25.0	13.0	136	136	24.8	11.9	25.3	13.0	
LEVEL 6	STORY5	D1	664	664	9.7	10.9	799	799	12.3	11.1	24.5	13.5	-14.8
LEVEL 5	STORY4	D1	631	631	10.2	10.8	1430	1430	11.3	11.0	24.1	13.5	-14.0
LEVEL 4	STORY3	D1	640	640	10.1	11.1	2070	2070	11.0	11.0	23.6	13.5	-13.5
LEVEL 3	STORY2	D1	650	650	10.1	11.3	2720	2720	10.8	11.1	22.9	13.5	-12.8
LEVEL 2	STORY1	D1	672	672	10.2	11.3	3392	3392	10.7	11.1	21.7	13.5	-11.6

LATHAM / MCCAHON

LEVEL	Story	Diaphragm	MassX	MassY	XCM	YCM	CumMassX	CumMassY	XCCM	YCCM	XCR	YCR	XCM-XCR (m)
	LEVEL8	D1	48	48	24.8	11.2	48	48			20.6	13.6	
	LEVEL7	D1	94	94	15.6	10.7	142	142			19.8	13.5	
	LEVEL6	D1	597	597	9.8	10.7	739	739			19.5	13.4	-9.6
	LEVEL5	D1	606	606	9.9	10.7	1345	1345			19.0	13.3	-9.1
	LEVEL4	D1	613	613	9.9	10.9	1958	1958			18.5	13.1	-8.6
	LEVEL3	D1	627	627	9.9	11.3	2585	2585			17.9	13.0	-8.0
	LEVEL2	D1	648	648	10.1	11.3	3233	3233			17.2	12.9	-7.2



LEVELS 2,3,4,5 & 6

EXAMPLE LEVEL 4

CTV BUILDING - NORTH-SOUTH (X-DIRECTION) POINT DRIFTS ALONG GRID F
WITH MASS OFFSET 0.1B TOWARDS SOUTH AND EAST

TABLE B3

FIXED BASE

LEVEL	Story	Point (@ Grid F1)	Load	SPECXDUCTILE DispX (m)	SPECXDUCTILE DriftX (m/m)	Computed Deformations (DEMAND) Ref NZS 4203:1984 Clause 3.8.1 SPECXDUCTILE x 2.75 DispX (m)	DriftX (m/m)	DriftX (%)
LEVEL 5	STORY5	12	SPECXDUCTILE	0.023	0.0017	0.062	0.0047	0.47
LEVEL 4	STORY4	12	SPECXDUCTILE	0.017	0.0017	0.048	0.0046	0.46
LEVEL 3	STORY3	12	SPECXDUCTILE	0.012	0.0016	0.032	0.0043	0.43
LEVEL 2	STORY2	12	SPECXDUCTILE	0.007	0.0013	0.018	0.0035	0.35
LEVEL 1	STORY1	12	SPECXDUCTILE	0.003	0.0007	0.007	0.0018	0.18

T&T STIFF SOIL

LEVEL	Story	Point	Load	SPECXDUCTILE DispX (m)	SPECXDUCTILE DriftX (m/m)	Computed Deformations Ref NZS 4203:1984 Clause 3.8.1 SPECXDUCTILE x 2.75 DispX (m)	DriftX (m/m)	DriftX (%)
LEVEL 5	STORY5	12	SPECXDUCTILE	0.036	0.0024	0.099	0.0066	0.66
LEVEL 4	STORY4	12	SPECXDUCTILE	0.028	0.0024	0.078	0.0067	0.67
LEVEL 3	STORY3	12	SPECXDUCTILE	0.020	0.0023	0.056	0.0064	0.64
LEVEL 2	STORY2	12	SPECXDUCTILE	0.013	0.0021	0.035	0.0058	0.58
LEVEL 1	STORY1	12	SPECXDUCTILE	0.006	0.0016	0.017	0.0044	0.44

T&T LIKELY SOIL

LEVEL	Story	Point	Load	SPECXDUCTILE DispX (m)	SPECXDUCTILE DriftX (m/m)	Computed Deformations Ref NZS 4203:1984 Clause 3.8.1 SPECXDUCTILE x 2.75 DispX (m)	DriftX (m/m)	DriftX (%)
LEVEL 5	STORY5	12	SPECXDUCTILE	0.040	0.0027	0.111	0.0073	0.73
LEVEL 4	STORY4	12	SPECXDUCTILE	0.032	0.0027	0.087	0.0073	0.73
LEVEL 3	STORY3	12	SPECXDUCTILE	0.023	0.0026	0.063	0.0071	0.71
LEVEL 2	STORY2	12	SPECXDUCTILE	0.015	0.0023	0.040	0.0065	0.65
LEVEL 1	STORY1	12	SPECXDUCTILE	0.007	0.0018	0.019	0.0051	0.51

T&T SOFT SOIL

LEVEL	Story	Point	Load	SPECXDUCTILE DispX (m)	SPECXDUCTILE DriftX (m/m)	Computed Deformations Ref NZS 4203:1984 Clause 3.8.1 SPECXDUCTILE x 2.75 DispX (m)	DriftX (m/m)	DriftX (%)
LEVEL 5	STORY5	12	SPECXDUCTILE	0.045	0.0029	0.123	0.0080	0.80
LEVEL 4	STORY4	12	SPECXDUCTILE	0.035	0.0029	0.097	0.0081	0.81
LEVEL 3	STORY3	12	SPECXDUCTILE	0.026	0.0028	0.071	0.0078	0.78
LEVEL 2	STORY2	12	SPECXDUCTILE	0.017	0.0026	0.046	0.0072	0.72
LEVEL 1	STORY1	12	SPECXDUCTILE	0.008	0.0021	0.022	0.0058	0.58

CTV BUILDING - EAST-WEST (Y-DIRECTION) POINT DRIFTS ALONG GRID 1
WITH MASS OFFSET 0.1B TOWARDS SOUTH AND EAST

TABLE B4

FIXED BASE @ Grid F1

LEVEL	Story	Point	Load	SPECYDUCTILE DispY (m)	SPECYDUCTILE DriftY (m/m)	Computed Deformations (DEMAND) Ref NZS 4203:1984 Clause 3.8.1 SPECYDUCTILE x 2.75 DispY (m)	DriftY (m/m)	DriftY (%)
LEVEL 5	STORY5	12	SPECYDUCTILE	0.033	0.0026	0.090	0.0072	0.72
LEVEL 4	STORY4	12	SPECYDUCTILE	0.024	0.0026	0.067	0.0071	0.71
LEVEL 3	STORY3	12	SPECYDUCTILE	0.016	0.0023	0.045	0.0063	0.63
LEVEL 2	STORY2	12	SPECYDUCTILE	0.009	0.0018	0.024	0.0048	0.48
LEVEL 1	STORY1	12	SPECYDUCTILE	0.003	0.0008	0.008	0.0022	0.22

T&T STIFF SOIL

LEVEL	Story	Point	Load	SPECYDUCTILE DispY (m)	SPECYDUCTILE DriftY (m/m)	Computed Deformations Ref NZS 4203:1984 Clause 3.8.1 SPECYDUCTILE x 2.75 DispY (m)	DriftY (m/m)	DriftY (%)
LEVEL 5	STORY5	12	SPECYDUCTILE	0.039	0.0029	0.107	0.0080	0.80
LEVEL 4	STORY4	12	SPECYDUCTILE	0.030	0.0028	0.081	0.0078	0.78
LEVEL 3	STORY3	12	SPECYDUCTILE	0.020	0.0026	0.056	0.0072	0.72
LEVEL 2	STORY2	12	SPECYDUCTILE	0.012	0.0021	0.032	0.0059	0.59
LEVEL 1	STORY1	12	SPECYDUCTILE	0.005	0.0013	0.013	0.0035	0.35

T&T LIKELY SOIL

LEVEL	Story	Point	Load	SPECYDUCTILE DispY (m)	SPECYDUCTILE DriftY (m/m)	Computed Deformations Ref NZS 4203:1984 Clause 3.8.1 SPECYDUCTILE x 2.75 DispY (m)	DriftY (m/m)	DriftY (%)
LEVEL 5	STORY5	12	SPECYDUCTILE	0.039	0.0029	0.108	0.0080	0.80
LEVEL 4	STORY4	12	SPECYDUCTILE	0.030	0.0028	0.082	0.0078	0.78
LEVEL 3	STORY3	12	SPECYDUCTILE	0.021	0.0026	0.057	0.0072	0.72
LEVEL 2	STORY2	12	SPECYDUCTILE	0.012	0.0022	0.034	0.0060	0.60
LEVEL 1	STORY1	12	SPECYDUCTILE	0.005	0.0013	0.014	0.0037	0.37

T&T SOFT SOIL

LEVEL	Story	Point	Load	SPECYDUCTILE DispY (m)	SPECYDUCTILE DriftY (m/m)	Computed Deformations Ref NZS 4203:1984 Clause 3.8.1 SPECYDUCTILE x 2.75 DispY (m)	DriftY (m/m)	DriftY (%)
LEVEL 5	STORY5	12	SPECYDUCTILE	0.040	0.0029	0.109	0.0080	0.80
LEVEL 4	STORY4	12	SPECYDUCTILE	0.030	0.0028	0.083	0.0078	0.78
LEVEL 3	STORY3	12	SPECYDUCTILE	0.021	0.0026	0.058	0.0072	0.72
LEVEL 2	STORY2	12	SPECYDUCTILE	0.013	0.0022	0.034	0.0061	0.61
LEVEL 1	STORY1	12	SPECYDUCTILE	0.005	0.0014	0.015	0.0039	0.39

CTV BUILDING - EAST-WEST (Y-DIRECTION) POINT DRIFTS ALONG GRID 2
WITH MASS OFFSET 0.1B TOWARDS SOUTH AND EAST

TABLE B5

FIXED BASE @ Grid D2

				SPECYDUCTILE		Computed Deformations (DEMAND) Ref NZS 4203:1984 Clause 3.8.1 SPECYDUCTILE x 2.75		
LEVEL	Story	Point	Load	DispY (m)	DriftY (m/m)	DispY (m)	DriftY (m/m)	DriftY (%)
LEVEL 5	STORY5	6	SPECYDUCTILE	0.024	0.0019	0.066	0.0053	0.53
LEVEL 4	STORY4	6	SPECYDUCTILE	0.018	0.0019	0.049	0.0052	0.52
LEVEL 3	STORY3	6	SPECYDUCTILE	0.012	0.0017	0.032	0.0046	0.46
LEVEL 2	STORY2	6	SPECYDUCTILE	0.006	0.0013	0.018	0.0035	0.35
LEVEL 1	STORY1	6	SPECYDUCTILE	0.002	0.0006	0.006	0.0016	0.16

T&T STIFF SOIL

				SPECYDUCTILE		Computed Deformations Ref NZS 4203:1984 Clause 3.8.1 SPECYDUCTILE x 2.75		
LEVEL	Story	Point	Load	DispY (m)	DriftY (m/m)	DispY (m)	DriftY (m/m)	DriftY (%)
LEVEL 5	STORY5	6	SPECYDUCTILE	0.029	0.0022	0.079	0.0059	0.59
LEVEL 4	STORY4	6	SPECYDUCTILE	0.022	0.0021	0.060	0.0058	0.58
LEVEL 3	STORY3	6	SPECYDUCTILE	0.015	0.0019	0.042	0.0053	0.53
LEVEL 2	STORY2	6	SPECYDUCTILE	0.009	0.0016	0.024	0.0044	0.44
LEVEL 1	STORY1	6	SPECYDUCTILE	0.004	0.0010	0.010	0.0026	0.26

T&T LIKELY SOIL

				SPECYDUCTILE		Computed Deformations Ref NZS 4203:1984 Clause 3.8.1 SPECYDUCTILE x 2.75		
LEVEL	Story	Point	Load	DispY (m)	DriftY (m/m)	DispY (m)	DriftY (m/m)	DriftY (%)
LEVEL 5	STORY5	6	SPECYDUCTILE	0.029	0.0021	0.080	0.0059	0.59
LEVEL 4	STORY4	6	SPECYDUCTILE	0.022	0.0021	0.061	0.0058	0.58
LEVEL 3	STORY3	6	SPECYDUCTILE	0.015	0.0019	0.042	0.0053	0.53
LEVEL 2	STORY2	6	SPECYDUCTILE	0.009	0.0016	0.025	0.0045	0.45
LEVEL 1	STORY1	6	SPECYDUCTILE	0.004	0.0010	0.011	0.0028	0.28

T&T SOFT SOIL

				SPECYDUCTILE		Computed Deformations Ref NZS 4203:1984 Clause 3.8.1 SPECYDUCTILE x 2.75		
LEVEL	Story	Point	Load	DispY (m)	DriftY (m/m)	DispY (m)	DriftY (m/m)	DriftY (%)
LEVEL 5	STORY5	6	SPECYDUCTILE	0.030	0.0022	0.081	0.0059	0.59
LEVEL 4	STORY4	6	SPECYDUCTILE	0.023	0.0021	0.062	0.0058	0.58
LEVEL 3	STORY3	6	SPECYDUCTILE	0.016	0.0020	0.043	0.0054	0.54
LEVEL 2	STORY2	6	SPECYDUCTILE	0.009	0.0016	0.026	0.0045	0.45
LEVEL 1	STORY1	6	SPECYDUCTILE	0.004	0.0011	0.011	0.0029	0.29

-----Original Message-----
From: Derek Bradley [mailto:derek@compusoftengineering.com]
Sent: Tuesday, 31 July 2012 4:43 p.m.
To: Ashley Smith (StructureSmith)
Subject: Fw: CTV Frame actions

-----Original Message-----
From: Derek Bradley
Sent: Wednesday, May 23, 2012 11:07 AM
To: [redacted]
Subject: Re: CTV Frame actions

Murray,

Attached are the frame actions that I sent you yesterday. I have also included the actions if a fixed base model was considered instead i.e. no soil springs.

If you have any queries, give me a call.

Regards,
Derek

-----Original Message-----
From: [redacted]
Sent: Tuesday, May 22, 2012 6:13 PM
To: Derek Bradley
Subject: Re: CTV Frame actions

thanks Derek
Kind Regards
Murray Jacobs
---- Derek Bradley <derek@compusoftengineering.com> wrote:
> Murray,
>
> Please find attached the frame actions for the GL 2 and GL F frames.
> I have imposed the displacements from the 3-D ETABS analysis (that
> does not have frame action incorporated) onto a 2-D frame as per the
> 1984 code requirements. The displacements are the worst case for the
> various 0.1B models, and are elastic displacements. The results can
> be linearly scaled to get the various proportions of elastic actions
> required for the 1982 concrete code.
>
> Note that the results assume Igross on all columns and have spring
> supports. I will have a look at a fixed base model to get a feel for
> the difference in actions.
>
> If you need anything else let me know.
>
> Cheers,
> Derek

The following is the frame F actions for the imposed displacements determined for the the model that has springs at the base, does not have any frame contribution, and includes 0.1B eccentricity effects. The imposed elastic displacements are;

Level 6 - 176.84mm

Level 5 - 138.43mm

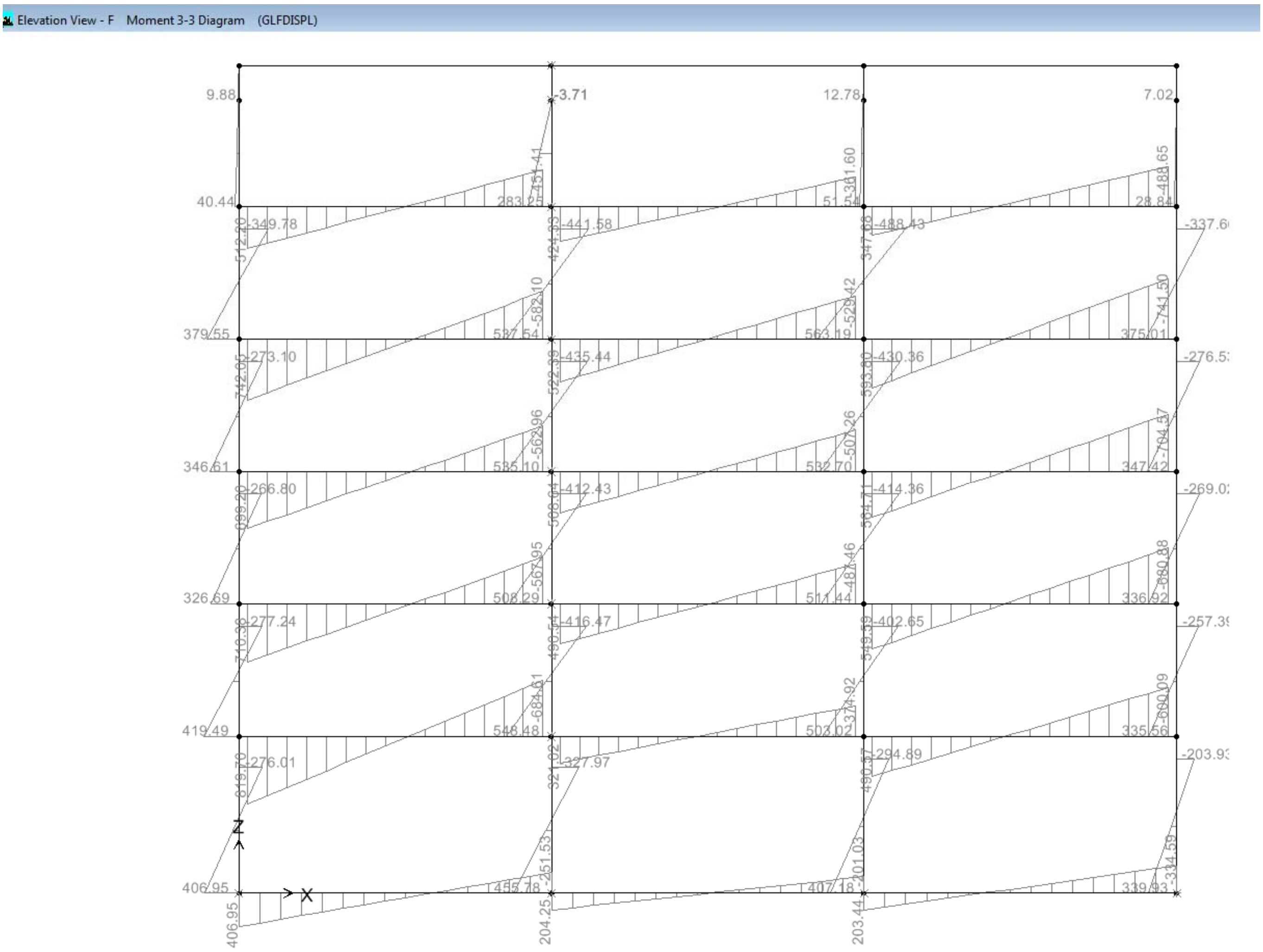
Level 4 - 99.93mm

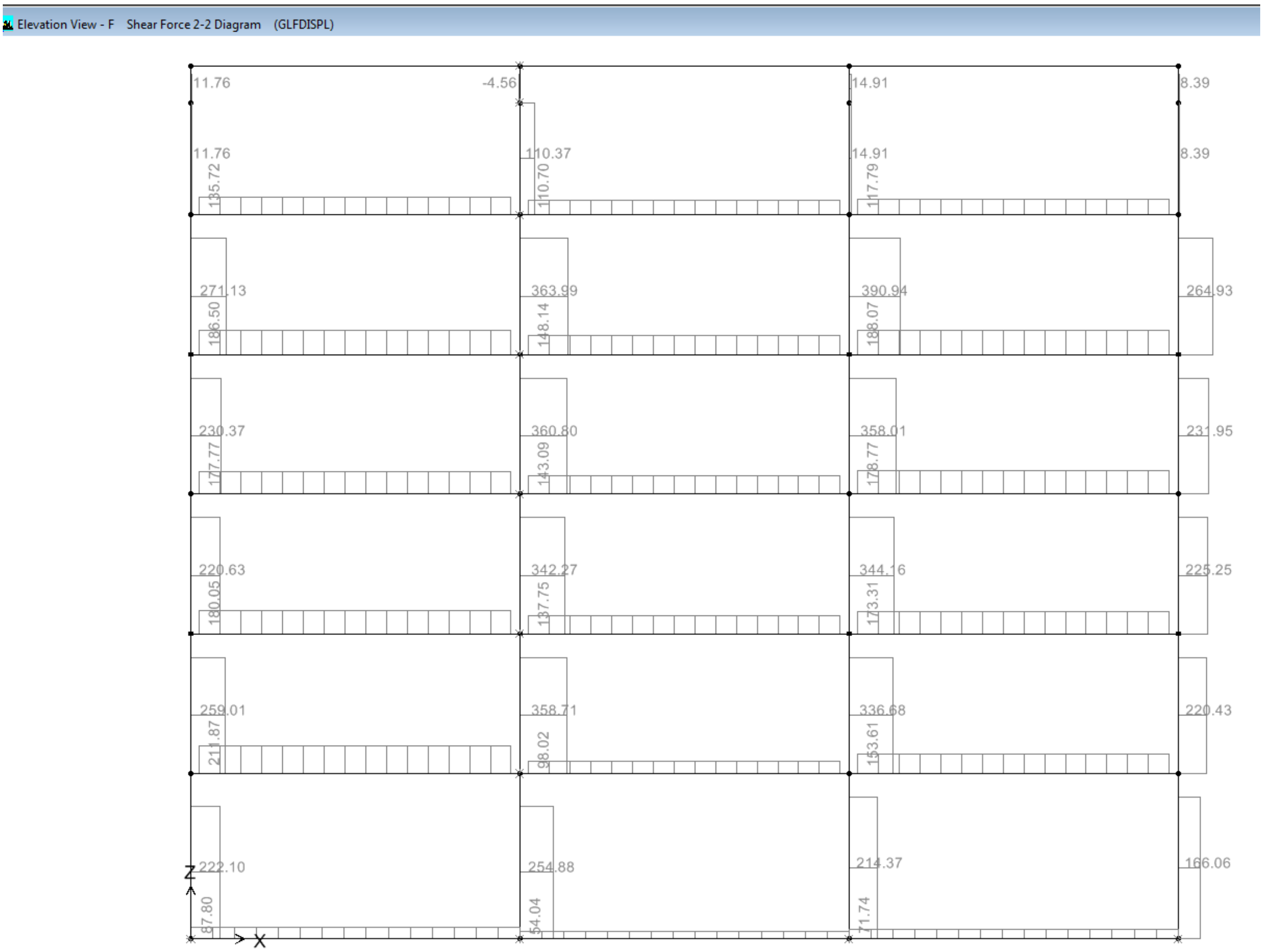
Level 3 - 63.01mm

Level 2 - 29.64mm

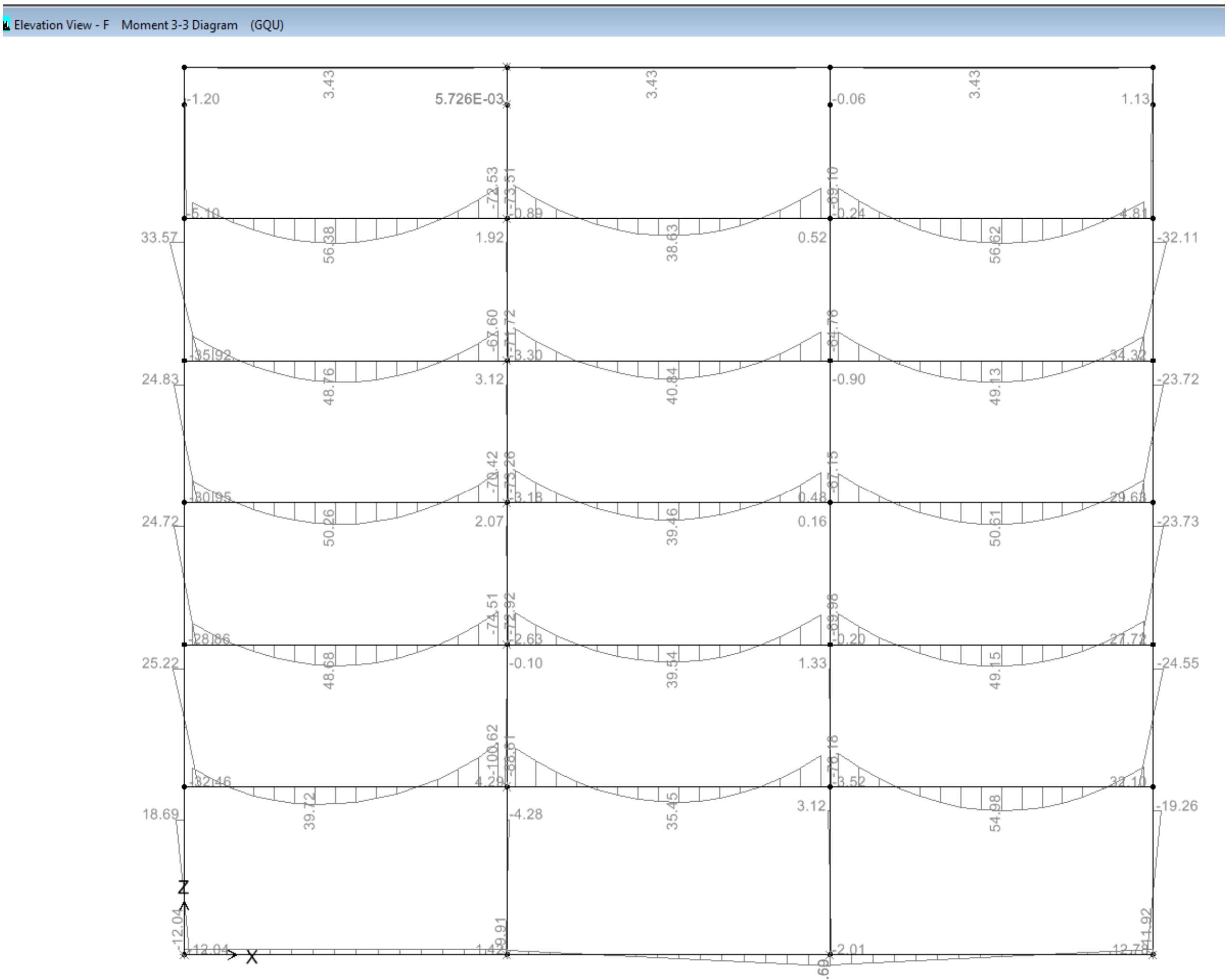
Results are for elastic actions (Mp = 0.8, R = 1, S = 5) and Igross for the columns

Note that these are just the componet of displacement for the N/S direction - due to torsional effects there will be a concurrent component in the E/W direction however it is not possible to tell what this from a response spectrum analysis.





Below are the G+Qu DL moments for GL F.



Wall on GL C N/S displ

Level 6 - 160.1mm

Level 5 - 125.6mm

Level 4 - 91.3mm

Level 3 - 58.3mm

Level 2 - 27.9mm

Wall on GL D/E N/S displ

Level 6 - 163.1mm

Level 5 - 127.9mm

Level 4 - 92.8mm

Level 3 - 58.9mm

Level 2 - 28.0mm

The following is for the case where the model has a fixed base i.e. no springs. The imposed elastic displacements are;

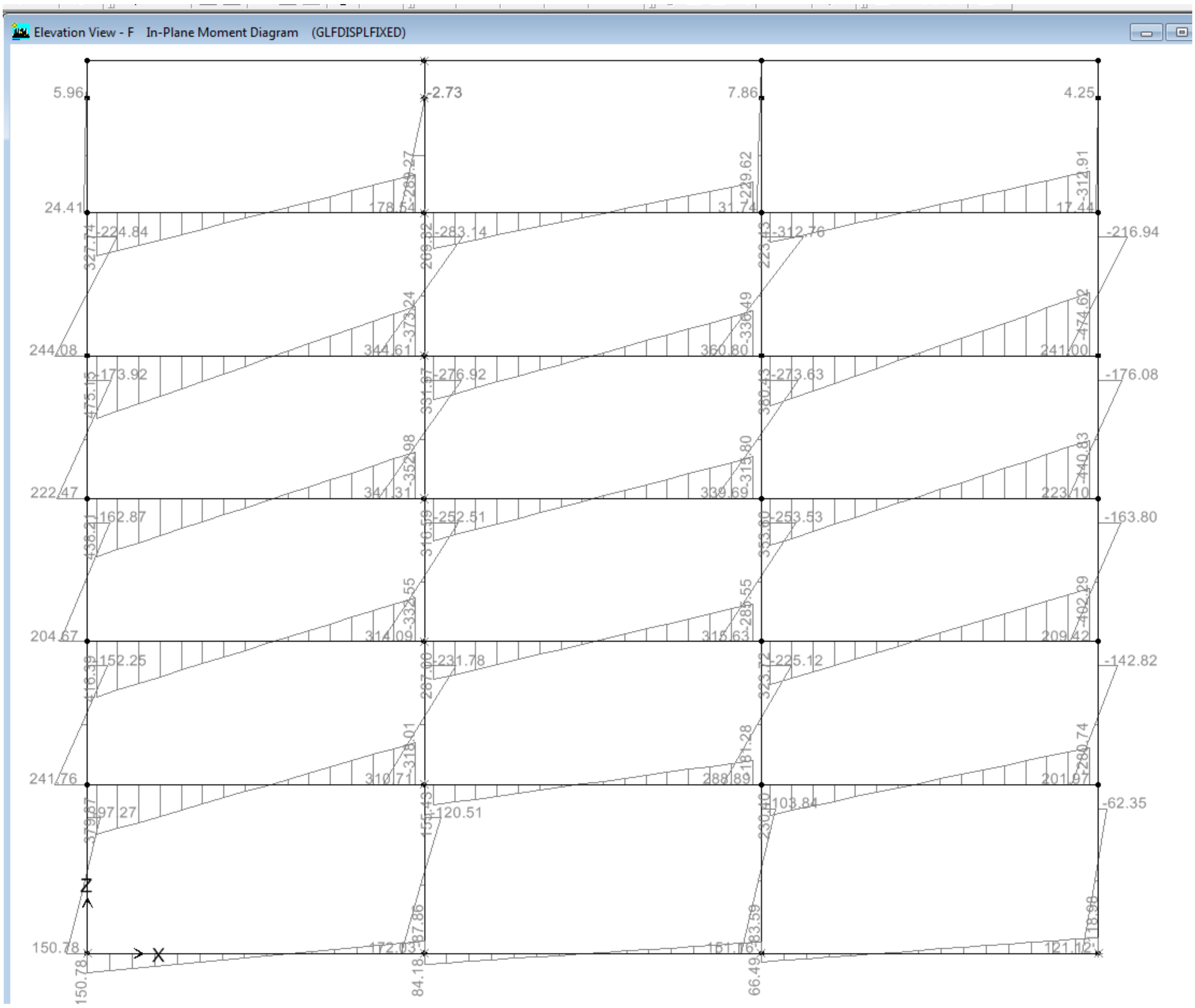
Level 6 - 101.81mm

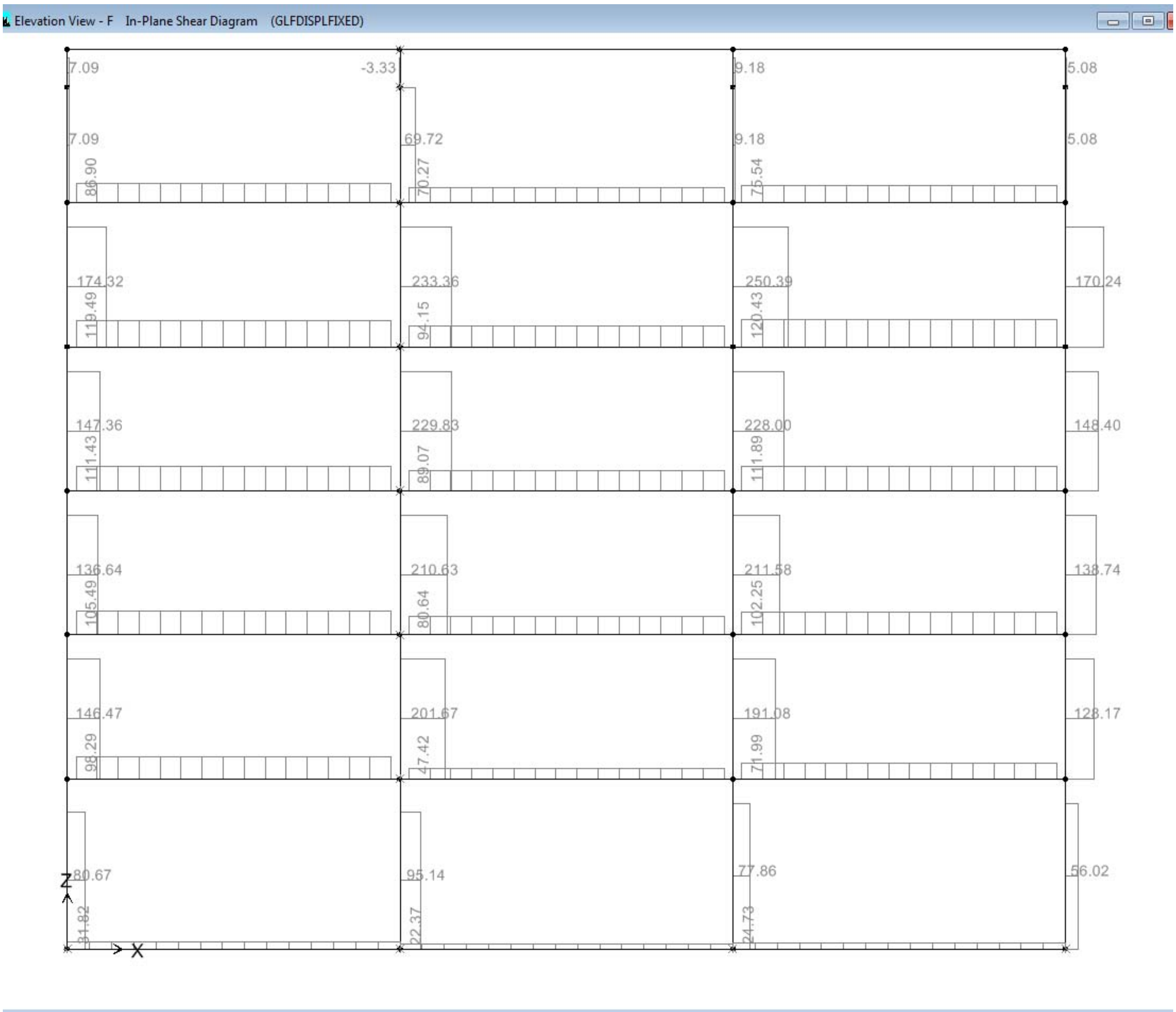
Level 5 - 77.23mm

Level 4 - 52.77mm

Level 3 - 30.15mm

Level 2 - 11.47mm




Elevation View - F In-Plane Shear Diagram (GLFDISPLFIXED)
7.09
-3.33
9.18
5.08
7.09
69.72
9.18
5.08
86.90
70.27
75.54
174.32
233.36
250.39
170.24
119.49
94.15
120.43
147.36
229.83
228.00
148.40
111.43
89.07
111.89
136.64
210.63
211.58
138.74
105.49
80.64
102.25
146.47
201.67
191.08
128.17
98.29
47.42
71.99
80.67
95.14
77.86
56.02
31.82
22.37
24.73
Z
X

The following is the frame 2 actions for the imposed displacements determined for the the model that has springs at the base, does not have any frame contribution, and includes 0.1B eccentricity effects. The imposed elastic displacements are;

Level 6 - 144.56mm

Level 5 - 109.64mm

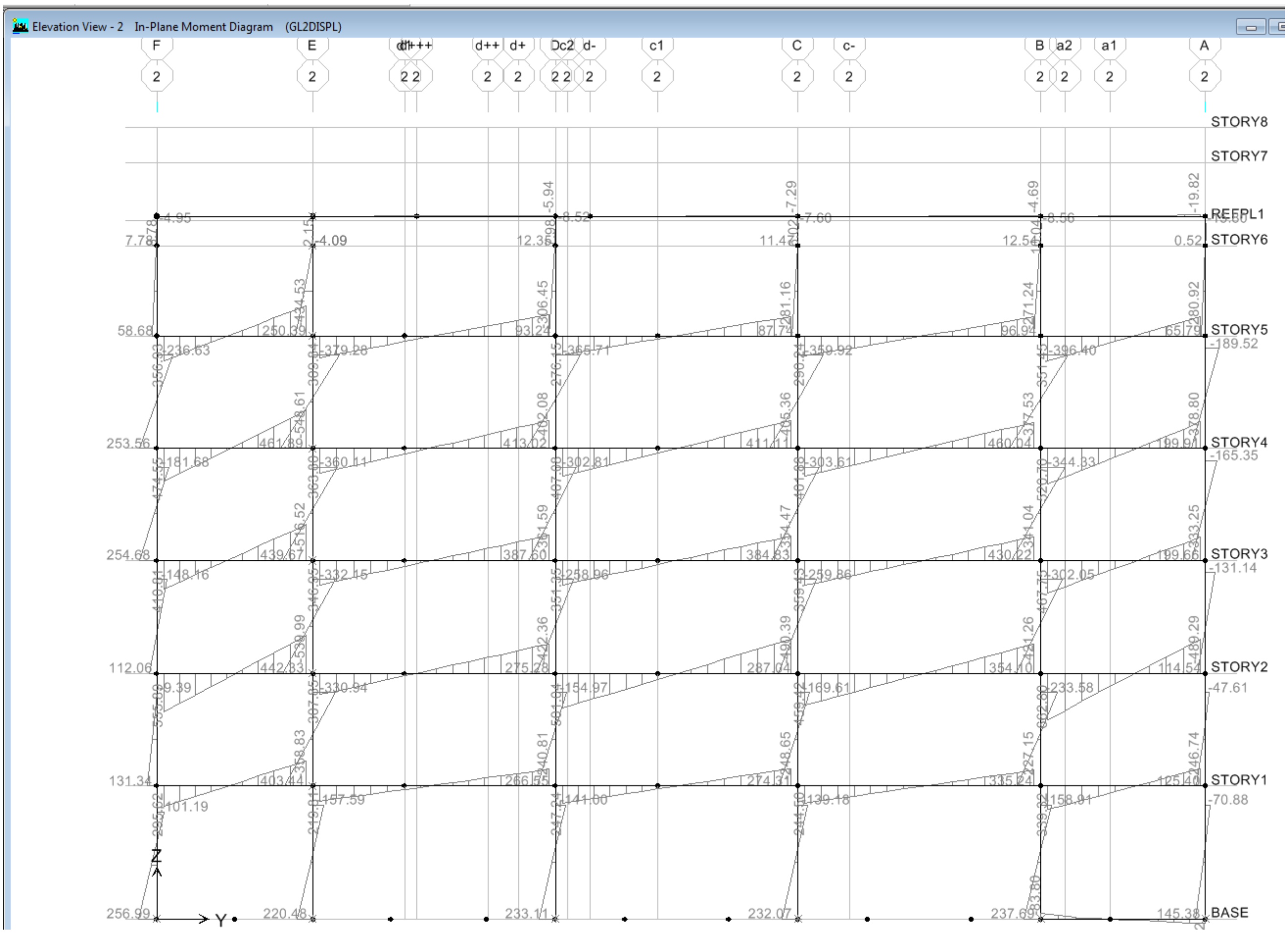
Level 4 - 75.43mm

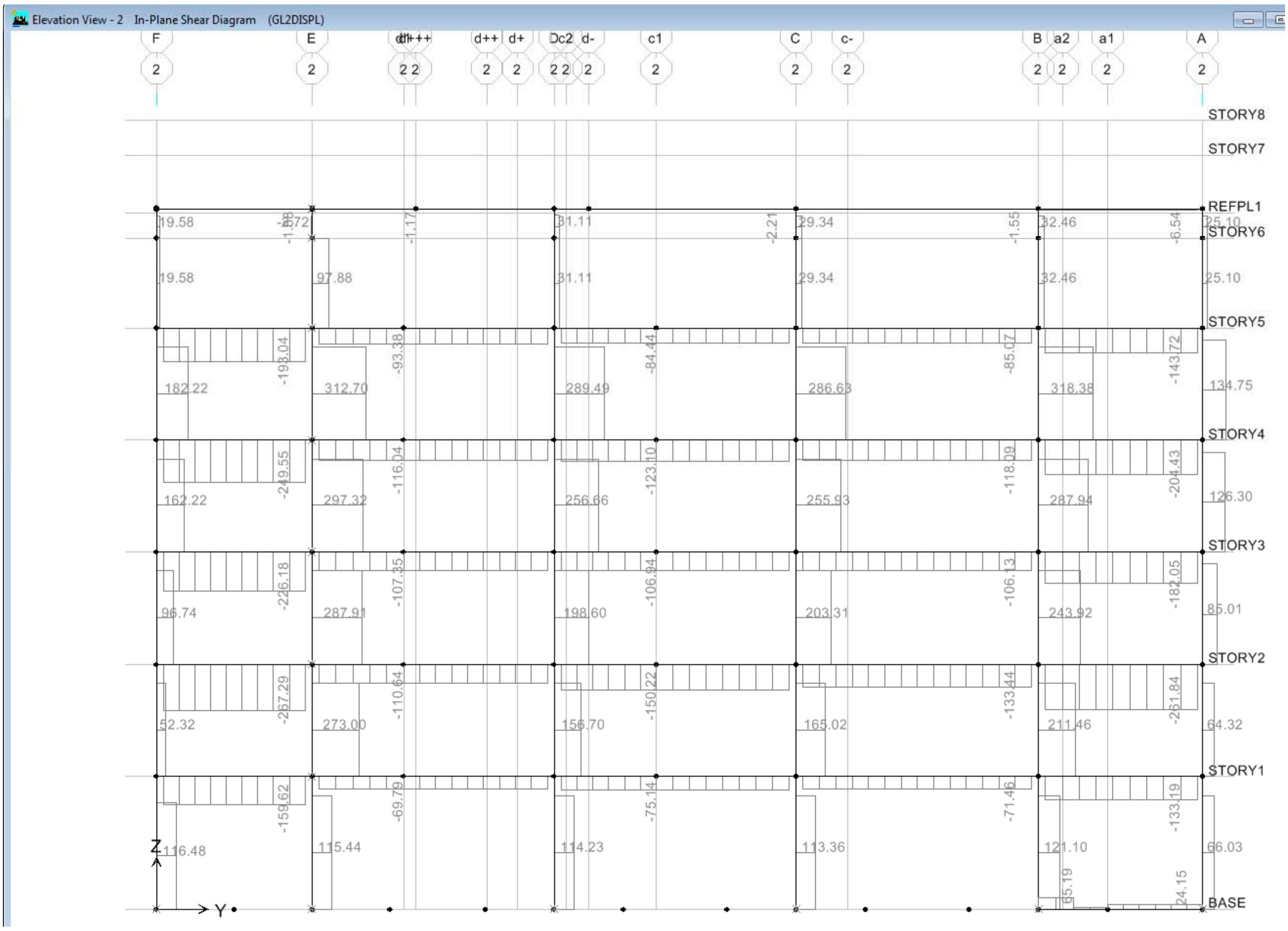
Level 3 - 44.05mm

Level 2 - 18.18mm

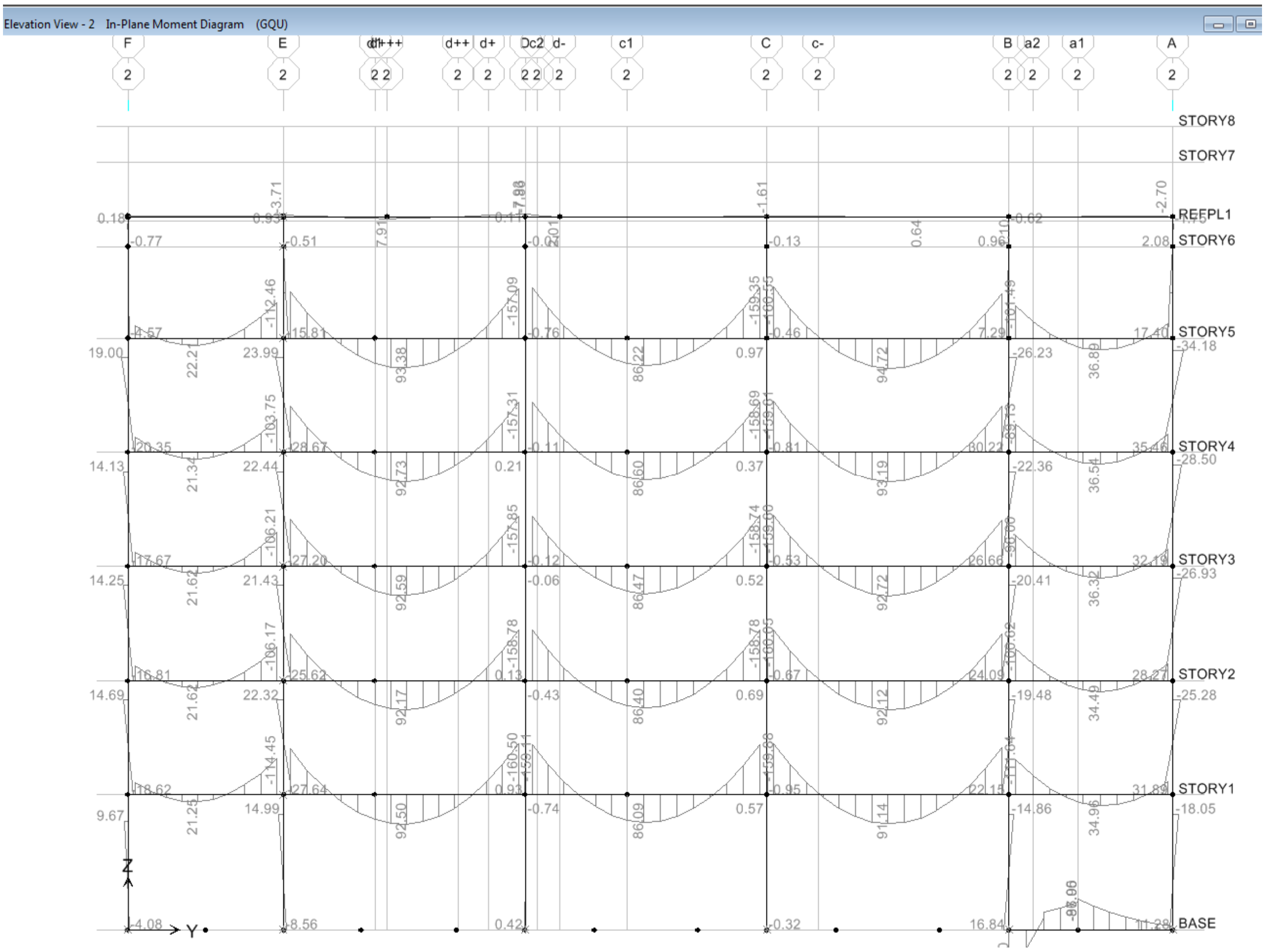
Results are for elastic actions (Mp = 0.8, R = 1, S = 5) and Igross for the columns

Note that these are just the componet of displacement for the E/W direction - due to torsional effects there will be a concurrent component in the N/S direction however it is not possible to tell what this from a response spectrum analysis.





Below are the G+Qu DL moments for GL 2.



Wall on GL 1 E/W displ

Level 6 - 194.7mm

Level 5 - 147.6mm

Level 4 - 101.4mm

Level 3 - 59.1mm

Level 2 - 24.3mm

Wall on GL 5 E/W displ

Level 6 - 34.4mm

Level 5 - 27.3mm

Level 4 - 20.2mm

Level 3 - 13.3mm

Level 2 - 6.7mm

The following is for the case where the structure has a fixed base i.e. no springs. The imposed elastic displacements are;

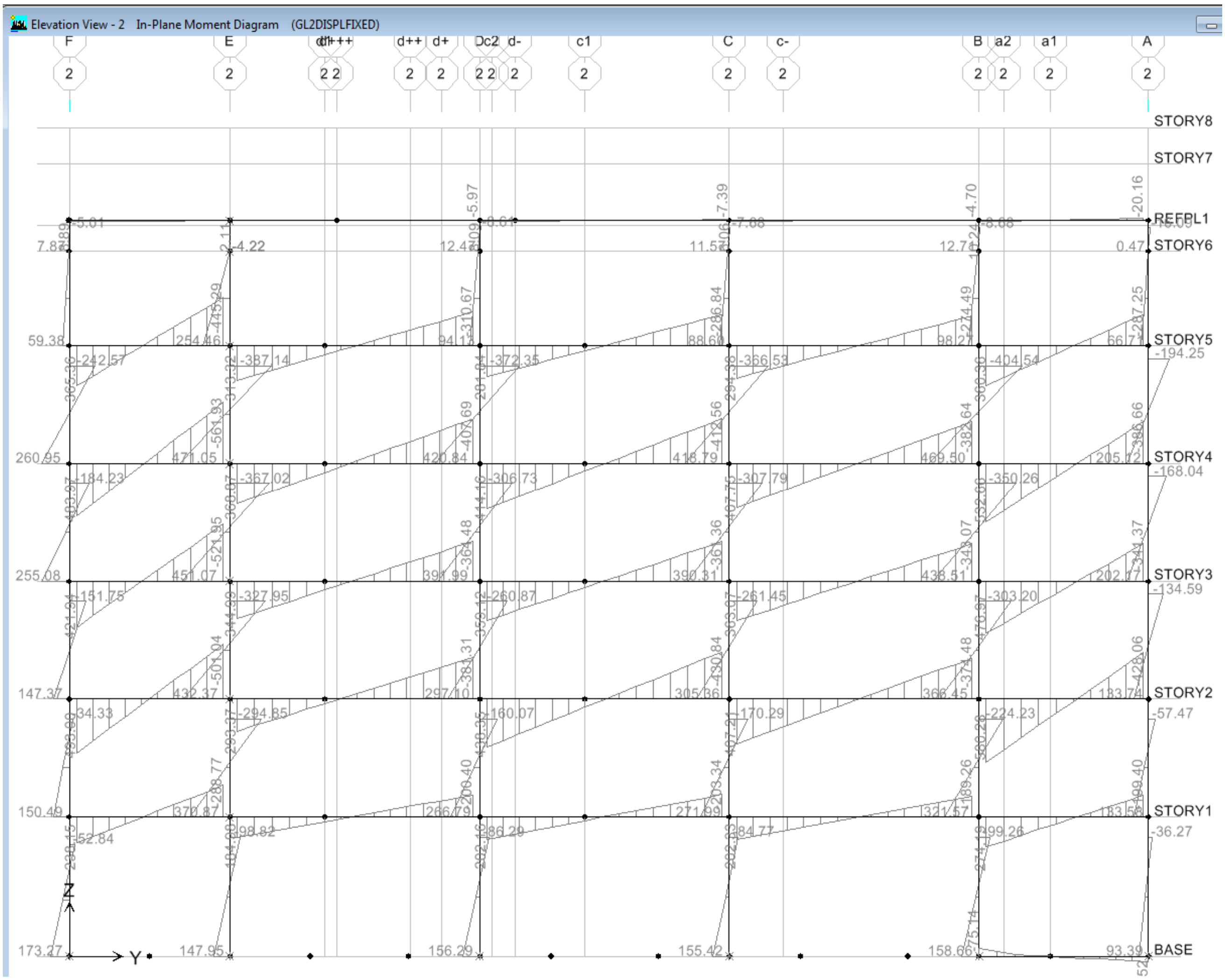
Level 6 - 137.96mm

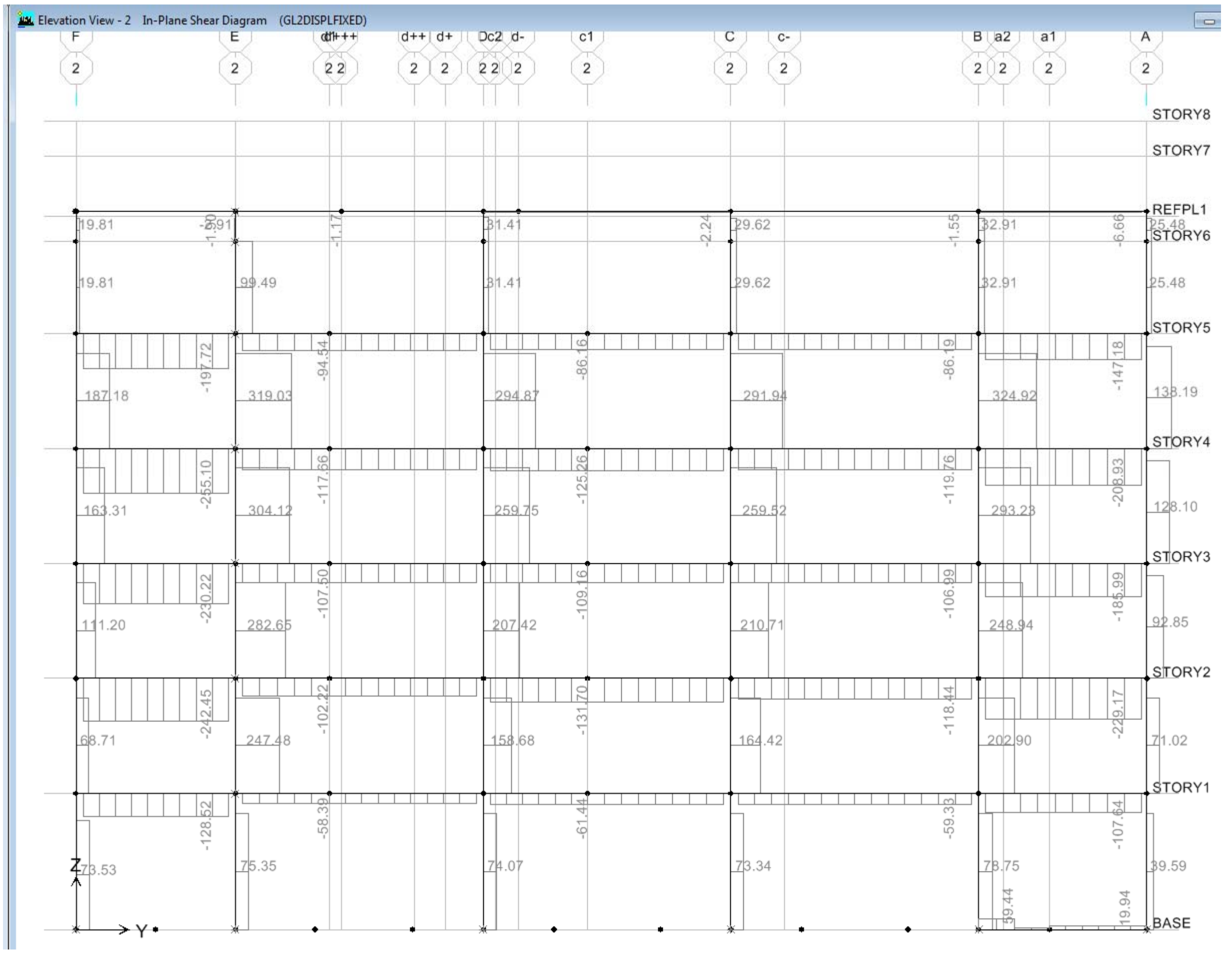
Level 5 - 102.40mm

Level 4 - 67.67mm

Level 3 - 36.60mm

Level 2 - 12.75mm




Elevation View - 2 In-Plane Shear Diagram (GL2DISPLFIXED)
STORY8
STORY7
REFPL1
STORY6
STORY5
STORY4
STORY3
STORY2
STORY1
BASE

"D"

Members in combined bending and axial load

Introduction

This part of the handbook deals with members subjected to combined bending and axial loads in an uniaxial state. The tables produced in this section have been based upon the design requirements of NZS 3101P*.

After considerable discussion on the relative merits of including ϕ factors, two complete sets of tables are published. The white set continues the overall policy of the handbook by including the relevant ϕ factors. The buff set has been produced for those designers wishing to work with P_i in the calculation method and these charts do not include capacity reduction factors. In effect therefore they represent the ϕ = 1 condition.

Acknowledgement

In the preparation of this part of the handbook the assistance of the following is gratefully recorded:

Beca Carter Hollings and Ferner, Auckland.

Cement and Concrete Association of Australia, Sydney.

Civil Engineering Division, Ministry of Works and Development, Wellington.

Control Data Australia Pty. Ltd, Sydney.

Department of Civil Engineering, University of Canterbury, Christchurch.

Computer services were provided by the University of Canterbury and the Ministry of Works and Development.

The continued advice of Morrison Cooper and Partners as consultants to the publication committee is acknowledged.

Tables and charts

*Except where the provisions of the draft NZ Standard DZ 3101: Parts 1 and 2, "Code of practice for the design of concrete structures", are more applicable.

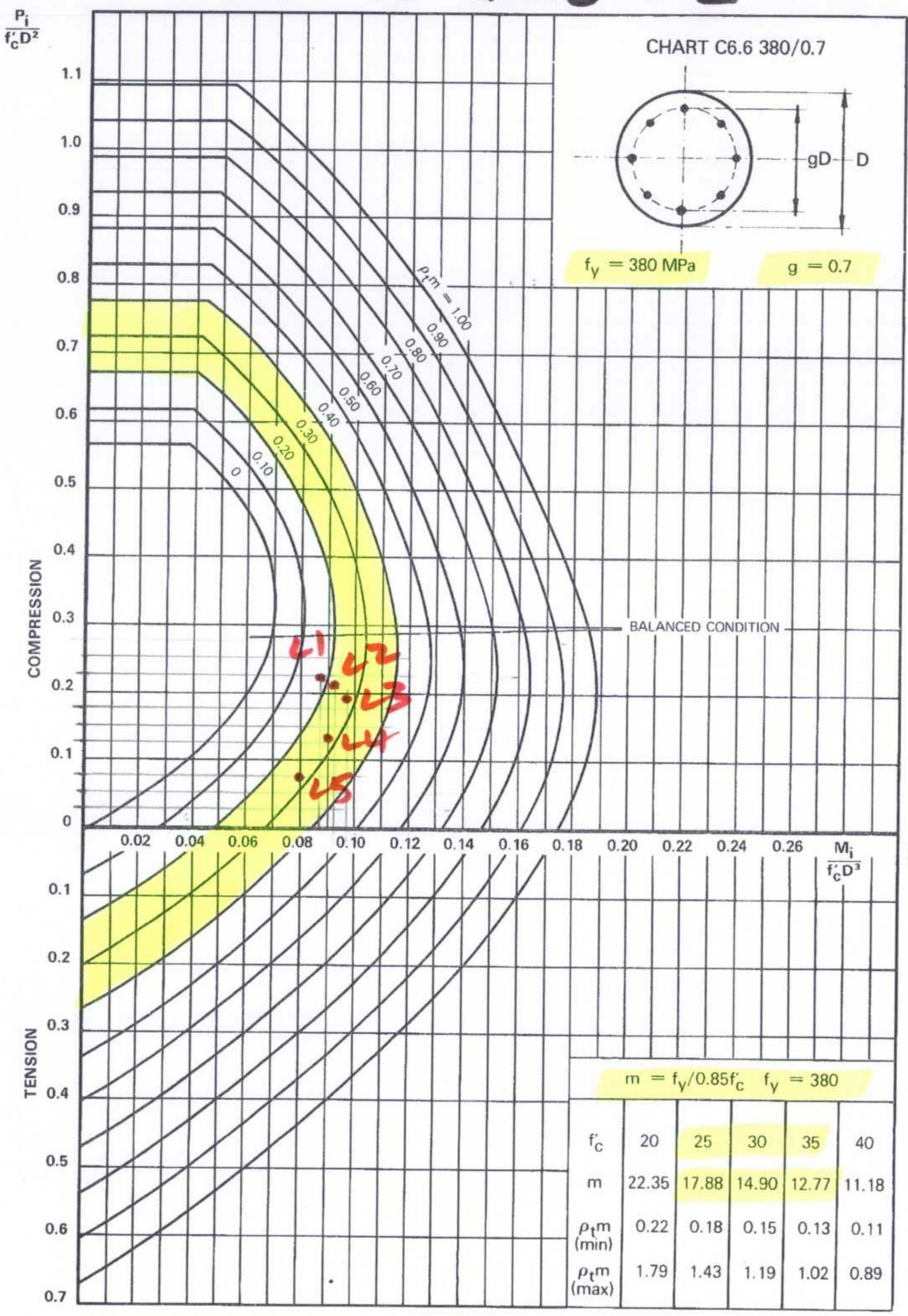
Notation

Notations are in addition to those contained in Members in Pure Bending.

A_c	=	area of core of spirally reinforced compression member measured to outside diameter of spiral,	mm
A_g	=	gross area of section,	mm^2
A_{st}	=	total area of longitudinal reinforcement	mm^2
A_{sh}	=	total effective area of hoop bars and supplementary ties in direction under consideration	mm^2
a	=	depth of equivalent rectangular stress block	mm
a_b	=	depth of equivalent rectangular stress block at balanced strain conditions	mm
C_m	=	a factor relating actual moment diagram to an equivalent uniform moment diagram	
d_b	=	diameter of reinforcing bar	mm
d_c	=	thickness of concrete cover measured from extreme tension fibre to centre of bar or wire located closest thereto	mm
d_s	=	distance from extreme tension fibre to centroid of tension reinforcement	mm
D	=	overall diameter of circular member	mm
EI	=	flexural stiffness of compression member	
e	=	eccentricity of load parallel to axis of member measured from centroid of gross section	mm
f_{yh}	=	specified yield strength of spiral reinforcement	MPa
g	=	factor where distance between centreline of reinforcement in opposite faces of a member is gh or gD	
h″	=	dimension of concrete core measured perpendicular to direction of hoop bars	mm^2
I_g	=	moment of inertia of gross concrete section about centroidal axis, neglecting reinforcement	m^4 or mm^4
I_{se}	=	moment of inertia of reinforcement about centroidal axis of member cross section	m^4 or mm^4

C6.6 COLUMN DESIGN CHART

COLUMN AT GRID F2



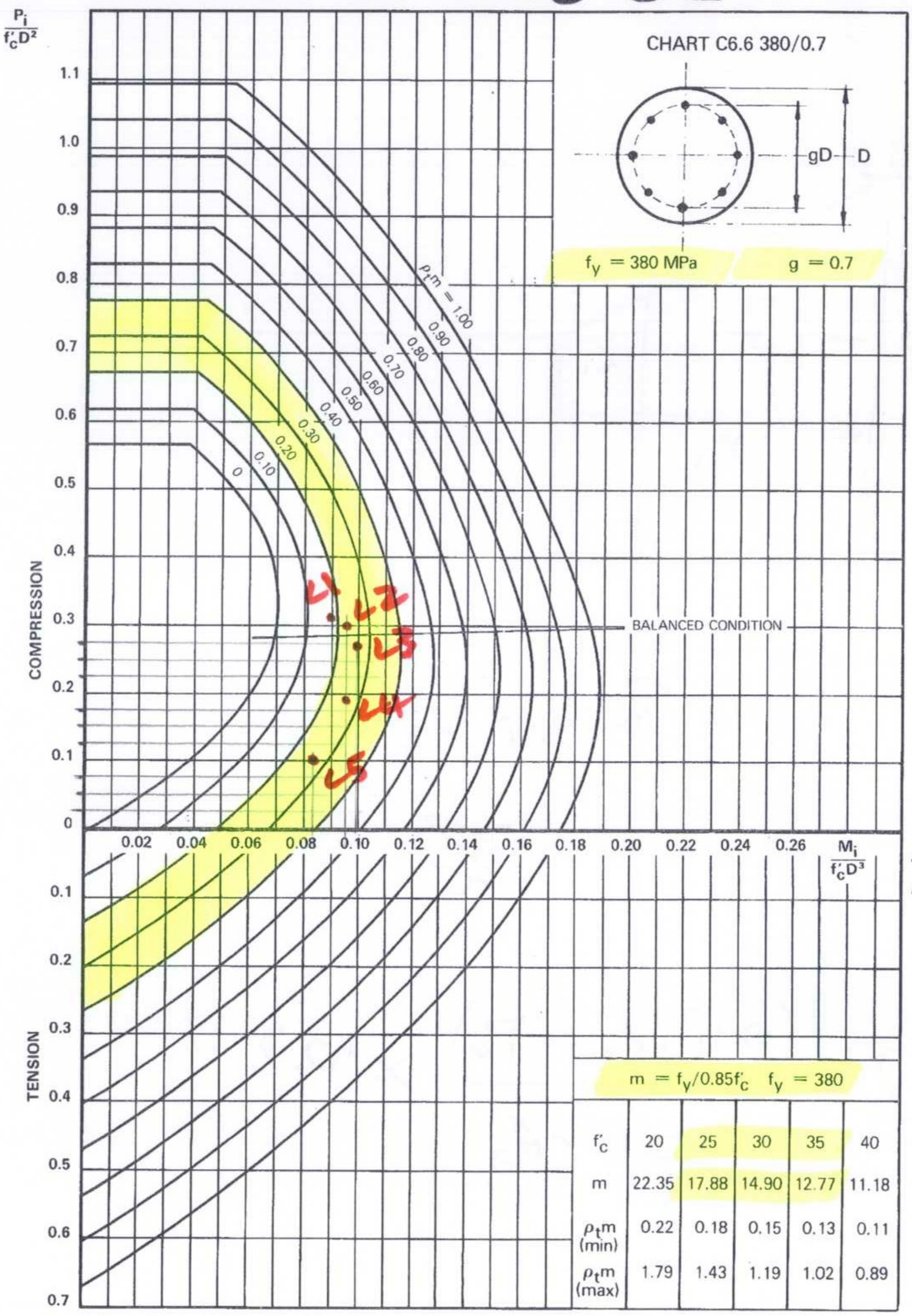
Ø=1

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

f'_c	20	25	30	35	40
m	22.35	17.88	14.90	12.77	11.18
$\rho_t m$ (min)	0.22	0.18	0.15	0.13	0.11
$\rho_t m$ (max)	1.79	1.43	1.19	1.02	0.89

C6.6 COLUMN DESIGN CHART

COLUMN AT GRID D2



$\phi = 1$

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

f'_c	20	25	30	35	40
m	22.35	17.88	14.90	12.77	11.18
$\rho_t m$ (min)	0.22	0.18	0.15	0.13	0.11
$\rho_t m$ (max)	1.79	1.43	1.19	1.02	0.89

CTV BUILDING - CODE DEPENDABLE DRIFT CAPACITIES

TABLE D1

DEPENDABLE DRIFT CAPACITY OF COLUMN AT GRID F2 (= DWG S14 COLUMN C5 / ETABS COLUMN C10) IN NORTH-SOUTH DIRECTION

400 dia COLUMN Ag = 125664 mm2, fy = 380 MPa, pt = 0.015

LEVEL	GRAVITY LOAD Pe = D + Lr (kN)	CONCRETE STRENGTH f'c (MPa)	0.1f'cAg (kN)	phi	m	ptm	Pe / f'cD^2	Mi / f'cD^3 from chart C6.6	phi.Mi Dependable (kNm)	From drift compatibility analysis by DB 31-07-2012: Column Moment (kNm)	Drift (mm)	Drift (%)	Dependable Drift CAPACITY (%)
LEVEL 5	269	25	314	0.73	17.88	0.27	0.07	0.080	93	537	38.4	1.19	0.21
LEVEL 4	513	25	314	0.70	17.88	0.27	0.13	0.090	101	535	38.5	1.19	0.22
LEVEL 3	754	25	314	0.70	17.88	0.27	0.19	0.098	110	508	36.9	1.14	0.25
LEVEL 2	995	30	377	0.70	14.90	0.22	0.21	0.092	124	548	33.4	1.03	0.23
LEVEL 1	1245	35	440	0.70	12.77	0.19	0.22	0.089	140	456	29.6	0.77	0.24

TABLE D2

DEPENDABLE DRIFT CAPACITY OF COLUMN AT GRID D2 (= DWG S14 COLUMN C7 / ETABS COLUMN C6) IN EAST-WEST DIRECTION

400 dia COLUMN Ag = 125664 mm2, fy = 380 MPa, pt = 0.015

LEVEL	GRAVITY LOAD Pe = D + Lr (kN)	CONCRETE STRENGTH f'c (MPa)	0.1f'cAg (kN)	phi	m	ptm	Pe / f'cD^2	Mi / f'cD^3 from chart C6.6	phi.Mi Dependable (kNm)	From drift compatibility analysis by DB 31-07-2012: Column Moment (kNm)	Drift (mm)	Drift (%)	Dependable Drift CAPACITY (%)
LEVEL 5	400	25	314	0.70	17.88	0.27	0.10	0.083	93	413	34.9	1.08	0.24
LEVEL 4	745	25	314	0.70	17.88	0.27	0.19	0.096	108	388	34.2	1.06	0.29
LEVEL 3	1085	25	314	0.70	17.88	0.27	0.27	0.100	112	275	31.4	0.97	0.39
LEVEL 2	1422	30	377	0.70	14.90	0.22	0.30	0.096	129	267	25.9	0.80	0.39
LEVEL 1	1759	35	440	0.70	12.77	0.19	0.31	0.090	141	233	18.2	0.47	0.29

"E"

CTV BUILDING - COLUMN DRIFT DEMAND VS DEPENDABLE FLEXURE CAPACITY RATIO

TABLE E1 - PERIMETER COLUMN AT GRID F2

LEVEL	DRIFT DEMAND NORTH-SOUTH (%)	'ELASTIC' OR DEPENDABLE CAPACITY		SMITH - RATIO DEMAND / CAPACITY	COLUMN STIFFNESS ASSUMPTION	
		MOMENT phi.Mi (kNm)	EQUIV. DRIFT		SMITH Ie/Ig COLUMN	LATHAM Ie/Ig COLUMN
5	0.47	93	0.21	2.2	1.00	0.38
4	0.46	101	0.22	2.1	1.00	0.50
3	0.43	110	0.25	1.7	1.00	0.71
2	0.35	124	0.23	1.5	1.00	0.98
1	0.18	140	0.24	0.8	1.00	1.00

TABLE E2 - INTERNAL COLUMN AT GRID D2

LEVEL	DRIFT DEMAND EAST-WEST (%)	'ELASTIC' OR DEPENDABLE CAPACITY		SMITH - RATIO DEMAND / CAPACITY	COLUMN STIFFNESS ASSUMPTION	
		MOMENT phi.Mi (kNm)	EQUIV. DRIFT		SMITH Ie/Ig COLUMN	LATHAM Ie/Ig COLUMN
5	0.53	93	0.24	2.2	1.00	0.38
4	0.52	108	0.29	1.8	1.00	0.50
3	0.46	112	0.39	1.2	1.00	0.71
2	0.35	129	0.39	0.9	1.00	0.98
1	0.16	141	0.29	0.6	1.00	1.00