FATIGUE+EARTHQUAKE ENGINEERING

Appendix to 4th Brief of Evidence in Response to WIT.LATHAM.0002 and WIT.LATHAM.0003

Line 2 and F Group 2 Frame Design Compliance Checks for the CTV Building

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Dr Clark Hyland

Phone 09 262 0203, Fax 09 262 0243
PO Box 97282, Manukau City 2241, New Zealand, www.fatigueandfracture.com

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GROUP 2 FRAME DESIGN COMPLIANCE CHECKS

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Summary of Findings:

1. Sensitivity analyses were undertaken using rigid base, stiff soil, soft soil subgrade stiffnesses recommended by Tonkin & Taylor.
   Analysis was also done for soft soil and increased superimposed dead load of 0.95 kPa.

2. It was found that the most critical condition was for soft soil subgrade with the existing 0.50 kPa superimposed dead load. The stiff soil condition had been checked in BCR Appendix F.

3. Columns C/1, D/2 and F/2 were checked using the Elastic Theory limits of reinforcing stress of 0.55 f_y and concrete compression strain of $\varepsilon_c = 0.001$. In accordance with cl 3.5.14.3. Elastic theory being as defined by Park & Paulay, "Reinforced Concrete Structures", 1975 p.1
   "... elastic theory has been the basis of reinforced concrete design for many years."
   In describing the renewed interest in ultimate strength theory they note "... elastic theory (working stress) design method ..."
   (refer PP-1 to 3).

4. Elastic theory or the working stress design method was described in Appendix B of NZS3101:1982
Setting the tensile stress in reinforcing steel to 150 MPa for Grade 275 reinforcement or 0.55 fy as set more generally in an amendment when reinforcing steel grades changed. (refer PP-5).

5. Concrete stress limits were 0.45 f'c compared to 0.85 f'c for strength design.
The equivalent strain for attainment of 0.45 f'c based on Hognestad stress-strain curve for f'_c = 27.5 MPa was ε_c = 0.0006. However using Roy-Sozen straight line curve this would be ε_c = 0.001 which has been used for this assessment (Refer P&P fig 2.3, p.13 & p.27).

6. For drift compatability checks on Group 2 frames foundation effects on primary frame displacements must be accounted for as these directly affect interstorey drift demands on the Group 2

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elements.

The effect of foundation rotations on the South wall and North Core interstorey drifts may be discounted as relative drift demands between floors will be unaffected.

7. It was found that all the columns on lines 2 & F triggered the additional seismic design requirements of NZS 3101:1982 as set by cl 3.5.14.3(a) for elastic behaviour using the definition of elastic theory criteria.

8. In clause 3.5.14.3(b) 'plastic behaviour' could be interpreted as the point where steel reinfg reaches yield fy and concrete strain reaches $\varepsilon_c = 0.002$ as was used in the BCR Appendix F assessment. Fewer of the colms triggered this criteria.

9. An alternative interpretation is that 'plastic behaviour' is what occurs after 'elastic behaviour' based on the principles of elastic theory (cl 3.5.14.3(e)).

10. Either interpretation leads to the conclusion that the frames required careful treatment

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in terms of drift compatability design and provision of limited ductile design requirements at least as per BCR Appendix F.

11. It was shown that even if the additional seismic design requirements were not applied the frames still required additional spiral reinforcing in the line 2 columns as the minimum shear reinforcing requirements were triggered.

12. Minimum spirals of R6 @ 55 mm centres were required irrespective of the additional seismic requirements (Refer p. L2-4). This was for L4 & L5 on line 2 if cracked column properties were used in the analysis. It was required for all levels in uncracked column properties were used in the analysis (Refer p. L2-2 & L2-4) for line 2.

13. It was found that all the columns on line 2 exceeded their dependable flexural strength whether cracked or uncracked section properties were used in the analysis. This was with the exception of L2 column

which would have satisfied the flexural demands if cracked section properties were used. (refer p. LZ-5). This is irrespective of the additional seismic design requirements.

14. Along line F it was found that if cracked column properties were assumed in the analysis minimum spiral reinforcing of R6 @ 55 c/c was required irrespective of the additional seismic requirements. (refer p. LF-2)

15. If the recommended uncracked column properties were used in the analysis then R10 @ 55 c/c was required for columns from L3 to L5 and R6 @ 55 c/c below that

16. Along line F it was found that all the columns did not have sufficient dependable flexural compressive strength to cope with the design demands. This was irrespective of them being modelled as cracked or uncracked in the analyses.
This is irrespective of the additional seismic design requirements.

ERSA has been used to identify VS drift profile for design of the line 2 & line F frames in combination with design live and dead loads.
Cases include super dead loads as used in the BCR and also with superimposed dead load of 0.95 kPa

Analyses were undertaken using rigid, stiff & soft soil conditions for case of BCR loads and soft soil only for the case with 0.95 kPa superimposed dead load

The drifts were found to be highest by a small margin for the case with soft soils and the assumed BCR loads

These were then used to determine the demands on line 2 & 4 group 2 frames

The drifts were also compared to the Group 2 drift limit criteria for triggering the additional seismic design requirements and were found in nearly every case to trigger these for the Elastic Theory limits of $0.55 f_y$ reinforcing steel tensile stress and $\varepsilon_c = 0.001$ concrete compression strain

F/2 ERSA Inter-storey North-South Drifts

10% eccentricty of mass south and east of centre.
Values for the Elastic Theory Limit of fs= 0.55fy reinforcing steel tension also includes ec=0.002 concrete compression strain bound.
Elastic Deformation limit is fy reinforcing steel tension and ec=0.002 concrete compression strain.
The NZS 4203:1984 55% ULS limit is the $\nu\delta$ drift determined by factoring the S=1 ERSA drift by K/SM=2.75 .
Stiff and soft soil results use foundation subgrade stiffnesses recommended by the Tonkin and Taylor Ltd report.

Key:

(pink)	Does not satisfy fs=0.55fy limit
(lilac)	Does not satisfy ec=0.001 Limit

	Column Drift Limits					Stiff Soil No Line A Masonry Infill Contact		Soft Soil No Line A Masonry Infill Contact		Soft Soil with 0.95 kPa SDL No Line A Masonry Infill Contact	
Level	Axial Compression	Elastic Theory Limits		Elastic Deformation Limit	Failure Limit	NZS 4203:1984		NZS 4203:1984		NZS 4203:1984	
	kN	ec=0.001	fs=0.55fy		ec=0.004	55% ULS	ULS	55% ULS	ULS	55% ULS	ULS
North -South Earthquake											
L5 - L6	269	0.40%	0.32%	0.62%	1.58%	0.65%	1.19%	0.79%	1.44%	0.76%	1.38%
L4	513	0.33%	0.36%	0.73%	1.45%	0.66%	1.19%	0.79%	1.44%	0.76%	1.38%
L3	754	0.28%	0.44%	0.69%	1.30%	0.63%	1.15%	0.77%	1.40%	0.74%	1.34%
L2	995	0.23%	0.48%	0.61%	1.20%	0.57%	1.04%	0.71%	1.30%	0.69%	1.25%
L1	1245	0.20%	0.51%	0.55%	1.13%	0.44%	0.79%	0.58%	1.05%	0.56%	1.02%
East-West Earthquake											
L5 - L6	269	0.40%	0.32%	0.62%	1.58%	0.28%	0.51%				
L4	513	0.33%	0.36%	0.73%	1.45%	0.28%	0.50%				
L3	754	0.28%	0.44%	0.69%	1.30%	0.25%	0.46%				
L2	995	0.23%	0.48%	0.61%	1.20%	0.20%	0.37%				
L1	1245	0.20%	0.51%	0.55%	1.13%	0.13%	0.23%				

Similar to Table 14 in BCR with Elastic Theory Limits added along with 'Soft' soil drifts

DA-2 cont

C/1 ERSA Inter-storey East-West Drifts

10% eccentricty of mass south and east of centre.
Values for the Elastic Theory Limit of fs= 0.55fy reinforcing steel tension also includes ec=0.002 concrete compression strain bound.
Elastic Deformation limit is fy reinforcing steel tension and ec=0.002 concrete compression strain.
The NZS 4203:1984 55% ULS limit is the νδ drift determined by factoring the S=1 ERSA drift by K/SM=2.75 .
Stiff and soft soil results use foundation subgrade stiffnesses recommended by the Tonkin and Taylor Ltd report.

Key:

(pink)	Does not satisfy fs=0.55fy limit
(lavender)	Does not satisfy ec=0.001 Limit

	Column Drift Limits					Stiff Soil No Line A Masonry Infill Contact		Soft Soil No Line A Masonry Infill Contact		Soft Soil with 0.95 kPa SDL No Line A Masonry Infill Contact	
Level	Axial Compression kN	Elastic Theory Limits ec=0.001	fs=0.55fy	Elastic Deformation Limit	Failure Limit ec=0.004	NZS 4203:1984 55% ULS	ULS	NZS 4203:1984 55% ULS	ULS	NZS 4203:1984 55% ULS	ULS
North -South Earthquake											
L5 - L6	336	0.30%	0.40%	0.65%	1.55%	0.29%	0.52%				
L4	623	0.25%	0.47%	0.73%	1.37%	0.28%	0.52%				
L3	906	0.20%	0.50%	0.64%	1.23%	0.26%	0.48%				
L2	1189	0.20%	0.50%	0.58%	1.15%	0.21%	0.39%				
L1	1478	0.18%	0.50%	0.50%	1.10%	0.13%	0.23%				
East-West Earthquake											
L5 - L6	336	0.30%	0.40%	0.65%	1.55%	0.79%	1.43%	0.80%	1.46%	0.69%	1.25%
L4	623	0.25%	0.47%	0.73%	1.37%	0.77%	1.40%	0.79%	1.43%	0.68%	1.23%
L3	906	0.20%	0.50%	0.64%	1.23%	0.71%	1.29%	0.73%	1.33%	0.63%	1.14%
L2	1189	0.20%	0.50%	0.58%	1.15%	0.58%	1.06%	0.61%	1.11%	0.53%	0.96%
L1	1478	0.18%	0.50%	0.50%	1.10%	0.35%	0.63%	0.39%	0.71%	0.34%	0.61%

Similar to Table 13 in BCR with elastic theory limits + soft soil drifts added

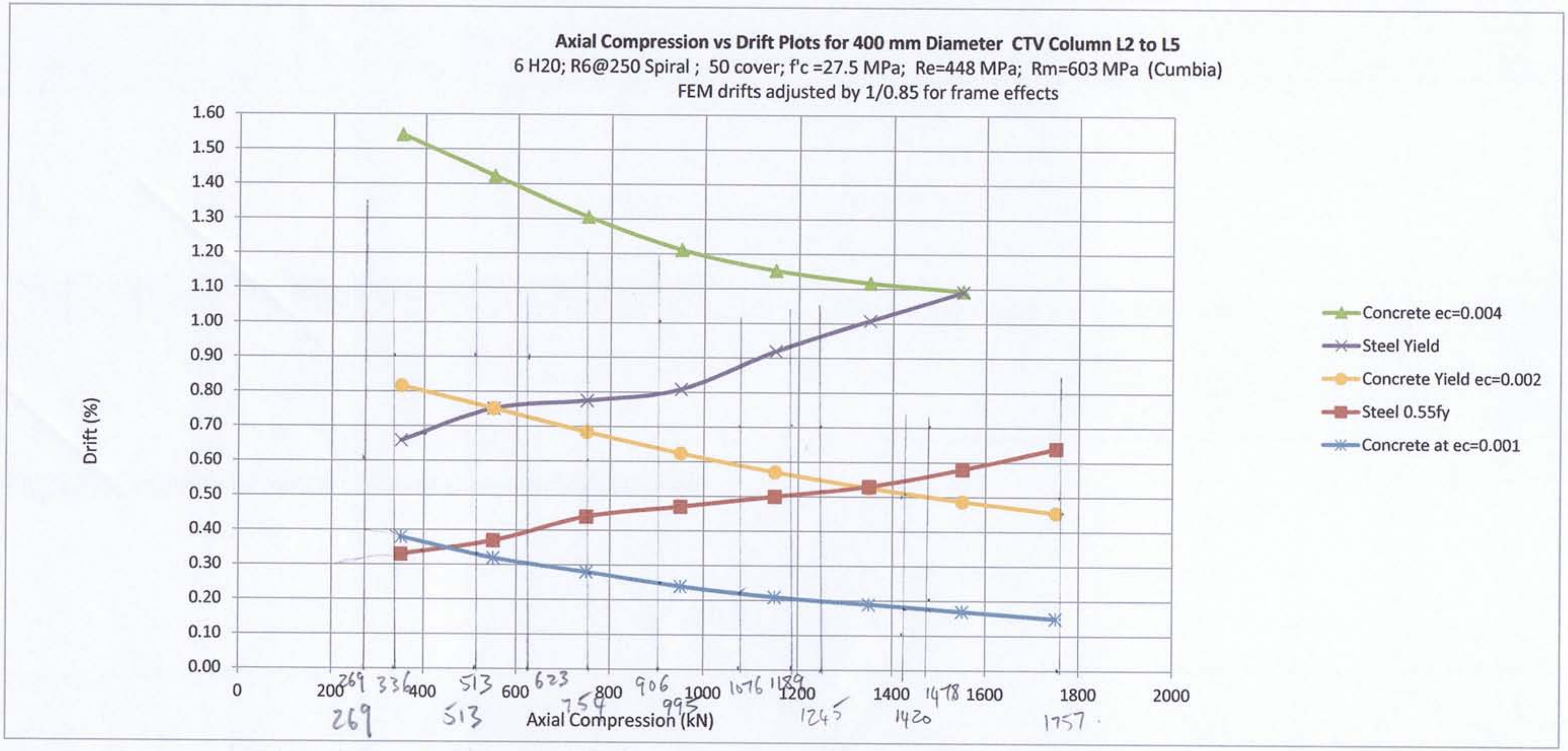
DA-3 Cont

D/2 ERSA Inter-storey East-West Drifts

10% eccentricty of mass south and east of centre.
Values for the Elastic Theory Limit of fs= 0.55fy reinforcing steel tension also includes ec=0.002 concrete compression strain bound.
Elastic Deformation limit is fy reinforcing steel tension and ec=0.002 concrete compression strain.
The NZS 4203:1984 55% ULS limit is the νδ drift determined by factoring the S=1 ERSA drift by K/SM=2.75 .
Stiff and soft soil results use foundation subgrade stiffnesses recommended by the Tonkin and Taylor Ltd report.

Key:
- (pink) Does not satisfy fs=0.55fy limit
- (lilac) Does not satisfy ec=0.001 Limit

	Column Drift Limits					Stiff Soil		Soft Soil		Soft Soil with 0.95 kPa SDL	
						No Line A Masonry Infill Contact		No Line A Masonry Infill Contact		No Line A Masonry Infill Contact	
Level	Axial Compression	Elastic Theory Limits		Elastic Deformation Limit	Failure Limit	NZS 4203:1984		NZS 4203:1984		NZS 4203:1984	
	kN	ec=0.001	fs=0.55fy		ec=0.004	55% ULS	ULS	55% ULS	ULS	55% ULS	ULS
North -South Earthquake											
L5 - L6	400	0.36%	0.34%	0.68%	1.55%	0.21%	0.38%				
L4	745	0.28%	0.43%	0.69%	1.36%	0.21%	0.38%				
L3	1085	0.22%	0.49%	0.59%	1.23%	0.19%	0.35%				
L2	1422	0.18%	0.51%	0.51%	1.13%	0.16%	0.28%				
L1	1759	0.16%	0.46%	0.46%	1.10%	0.09%	0.16%				
East-West Earthquake											
L5 - L6	400	0.36%	0.34%	0.68%	1.50%	0.59%	1.08%	0.60%	1.09%	0.51%	0.93%
L4	745	0.28%	0.43%	0.69%	1.32%	0.58%	1.06%	0.59%	1.07%	0.50%	0.91%
L3	1085	0.22%	0.49%	0.59%	1.18%	0.53%	0.97%	0.54%	0.99%	0.47%	0.85%
L2	1422	0.18%	0.51%	0.51%	1.10%	0.44%	0.80%	0.46%	0.83%	0.34%	0.62%
L1	1759	0.16%	0.46%	0.46%	1.10%	0.26%	0.48%	0.29%	0.53%	0.26%	0.46%



Similar to Fig 161 in BCR with steel at 0.55fy and concrete at $\varepsilon_c = 0.001$. Elastic Theory limits added.

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

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)	0.92	X (NORTH-SOUTH)	1.23	1.31	1.39	2.09
T2	X (NORTH-SOUTH)	0.77	Y1 (EAST-WEST)	1.04	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)	2844		1798	1796	1793	1712
Vy	Y (EAST-WEST)	2454		2153	2095	2037	1732

DA-6

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

FIXED BASE

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

DA-7

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

FIXED BASE **@ Grid F1**

				SPECXDUCTILE		Computed Deformations Ref NZS 4203:1984 Clause 3.8.1 SPECXDUCTILE x 2.75		
LEVEL	Story	Point	Load	DispX (m)	DriftX (m/m)	DispX (m)	DriftX (m/m)	DriftX (%)
LEVEL 5	STORY5	12	SPECXDUCTILE	0.0215	0.0016	0.0592	0.0044	0.44
LEVEL 4	STORY4	12	SPECXDUCTILE	0.0163	0.0016	0.0449	0.0044	0.44
LEVEL 3	STORY3	12	SPECXDUCTILE	0.0111	0.0015	0.0306	0.0041	0.41
LEVEL 2	STORY2	12	SPECXDUCTILE	0.0063	0.0012	0.0174	0.0033	0.33
LEVEL 1	STORY1	12	SPECXDUCTILE	0.0024	0.0006	0.0065	0.0017	0.17

T&T STIFF SOIL

				SPECXDUCTILE		Computed Deformations Ref NZS 4203:1984 Clause 3.8.1 SPECXDUCTILE x 2.75		
LEVEL	Story	Point	Load	DispX (m)	DriftX (m/m)	DispX (m)	DriftX (m/m)	DriftX (%)
LEVEL 5	STORY5	12	SPECXDUCTILE	0.0357	0.0024	0.0981	0.0065	0.65
LEVEL 4	STORY4	12	SPECXDUCTILE	0.0280	0.0024	0.0769	0.0066	0.66
LEVEL 3	STORY3	12	SPECXDUCTILE	0.0202	0.0023	0.0556	0.0063	0.63
LEVEL 2	STORY2	12	SPECXDUCTILE	0.0128	0.0021	0.0352	0.0057	0.57
LEVEL 1	STORY1	12	SPECXDUCTILE	0.0061	0.0016	0.0167	0.0044	0.44

EDLimit	ETLimit .55fy	ec=0.001	N* kN
0.62	0.32	0.4	269
0.73	0.36	0.33	513
0.69	0.44	0.28	754
0.61	0.48	0.23	995
0.55	0.51	0.2	1245

T&T LIKELY SOIL

				SPECXDUCTILE		Computed Deformations Ref NZS 4203:1984 Clause 3.8.1 SPECXDUCTILE x 2.75		
LEVEL	Story	Point	Load	DispX (m)	DriftX (m/m)	DispX (m)	DriftX (m/m)	DriftX (%)
LEVEL 5	STORY5	12	SPECXDUCTILE	0.0402	0.0027	0.1106	0.0073	0.73
LEVEL 4	STORY4	12	SPECXDUCTILE	0.0316	0.0027	0.0869	0.0073	0.73
LEVEL 3	STORY3	12	SPECXDUCTILE	0.0230	0.0026	0.0633	0.0071	0.71
LEVEL 2	STORY2	12	SPECXDUCTILE	0.0147	0.0023	0.0404	0.0065	0.65
LEVEL 1	STORY1	12	SPECXDUCTILE	0.0070	0.0018	0.0193	0.0051	0.51

T&T SOFT SOIL

				SPECXDUCTILE		Computed Deformations Ref NZS 4203:1984 Clause 3.8.1 SPECXDUCTILE x 2.75		
LEVEL	Story	Point	Load	DispX (m)	DriftX (m/m)	DispX (m)	DriftX (m/m)	DriftX (%)
LEVEL 5	STORY5	12	SPECXDUCTILE	0.0442	0.0029	0.1215	0.0079	0.79
LEVEL 4	STORY4	12	SPECXDUCTILE	0.0348	0.0029	0.0958	0.0079	0.79
LEVEL 3	STORY3	12	SPECXDUCTILE	0.0255	0.0028	0.0701	0.0077	0.77
LEVEL 2	STORY2	12	SPECXDUCTILE	0.0164	0.0026	0.0451	0.0071	0.71
LEVEL 1	STORY1	12	SPECXDUCTILE	0.0080	0.0021	0.0221	0.0058	0.58

EDLimit	ETLimit .55fy	ec=0.001	N* kN
0.62	0.32	0.4	269
0.73	0.36	0.33	513
0.69	0.44	0.28	754
0.61	0.48	0.23	995
0.55	0.51	0.2	1245

T&T SOFT SOIL with 0.95 kPa SDL

				SPECXDUCTILE		Computed Deformations Ref NZS 4203:1984 Clause 3.8.1 SPECXDUCTILE x 2.75		
LEVEL	Story	Point	Load	DispX (m)	DriftX (m/m)	DispX (m)	DriftX (m/m)	DriftX (%)
LEVEL 5	STORY5	12	SPECXDUCTILE	0.0425	0.0028	0.1169	0.0076	0.76
LEVEL 4	STORY4	12	SPECXDUCTILE	0.0336	0.0028	0.0923	0.0076	0.76
LEVEL 3	STORY3	12	SPECXDUCTILE	0.0246	0.0027	0.0677	0.0074	0.74
LEVEL 2	STORY2	12	SPECXDUCTILE	0.0159	0.0025	0.0438	0.0069	0.69
LEVEL 1	STORY1	12	SPECXDUCTILE	0.0078	0.0020	0.0214	0.0056	0.56

EDLimit	ETLimit .55fy	ec=0.001	N* kN
0.62	0.32	0.4	269
0.73	0.36	0.33	513
0.69	0.44	0.28	754
0.61	0.48	0.23	995
0.55	0.51	0.2	1245

SA-8 Cul

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

FIXED BASE @ Grid F1

				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	12	SPECYDUCTILE	0.0331	0.0026	0.091	0.0073	0.73
LEVEL 4	STORY4	12	SPECYDUCTILE	0.0245	0.0026	0.067	0.0071	0.71
LEVEL 3	STORY3	12	SPECYDUCTILE	0.0162	0.0023	0.044	0.0064	0.64
LEVEL 2	STORY2	12	SPECYDUCTILE	0.0087	0.0018	0.024	0.0049	0.49
LEVEL 1	STORY1	12	SPECYDUCTILE	0.0030	0.0008	0.008	0.0022	0.22

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 (%)	EDLimit	ETLimit .55fy	ec=0.001	N* kN
LEVEL 5	STORY5	12	SPECYDUCTILE	0.0383	0.0029	0.105	0.0079	0.79	0.65	0.4	0.3	336
LEVEL 4	STORY4	12	SPECYDUCTILE	0.0290	0.0028	0.080	0.0077	0.77	0.73	0.47	0.25	623
LEVEL 3	STORY3	12	SPECYDUCTILE	0.0200	0.0026	0.055	0.0071	0.71	0.64	0.5	0.2	906
LEVEL 2	STORY2	12	SPECYDUCTILE	0.0117	0.0021	0.032	0.0058	0.58	0.58	0.5	0.2	1189
LEVEL 1	STORY1	12	SPECYDUCTILE	0.0048	0.0013	0.013	0.0035	0.35	0.5	0.5	0.18	1478

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	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

				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 (%)	EDLimit	ETLimit .55fy	ec=0.001	N* kN
LEVEL 5	STORY5	12	SPECYDUCTILE	0.0399	0.0029	0.110	0.0080	0.80	0.65	0.4	0.3	336
LEVEL 4	STORY4	12	SPECYDUCTILE	0.0305	0.0029	0.084	0.0079	0.79	0.73	0.47	0.25	623
LEVEL 3	STORY3	12	SPECYDUCTILE	0.0212	0.0027	0.058	0.0073	0.73	0.64	0.5	0.2	906
LEVEL 2	STORY2	12	SPECYDUCTILE	0.0126	0.0022	0.035	0.0061	0.61	0.58	0.5	0.2	1189
LEVEL 1	STORY1	12	SPECYDUCTILE	0.0054	0.0014	0.015	0.0039	0.39	0.5	0.5	0.18	1478

T&T SOFT SOIL with 0.95 kPa SDL

				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 (%)	EDLimit	ETLimit .55fy	ec=0.001	N* kN
LEVEL 5	STORY5	12	SPECYDUCTILE	0.0343	0.0025	0.0943	0.0069	0.69	0.65	0.4	0.3	336
LEVEL 4	STORY4	12	SPECYDUCTILE	0.0262	0.0025	0.0721	0.0068	0.68	0.73	0.47	0.25	623
LEVEL 3	STORY3	12	SPECYDUCTILE	0.0183	0.0023	0.0503	0.0063	0.63	0.64	0.5	0.2	906
LEVEL 2	STORY2	12	SPECYDUCTILE	0.0109	0.0019	0.0300	0.0053	0.53	0.58	0.5	0.2	1189
LEVEL 1	STORY1	12	SPECYDUCTILE	0.0047	0.0012	0.0129	0.0034	0.34	0.5	0.5	0.18	1478

DA-9

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

FIXED BASE @ Grid D2

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	6	SPECYDUCTILE	0.0242	0.0019	0.067	0.0053	0.53
LEVEL 4	STORY4	6	SPECYDUCTILE	0.0180	0.0019	0.049	0.0052	0.52
LEVEL 3	STORY3	6	SPECYDUCTILE	0.0118	0.0017	0.033	0.0047	0.47
LEVEL 2	STORY2	6	SPECYDUCTILE	0.0064	0.0013	0.018	0.0036	0.36
LEVEL 1	STORY1	6	SPECYDUCTILE	0.0022	0.0006	0.006	0.0016	0.16

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	6	SPECYDUCTILE	0.0284	0.0021	0.078	0.0058	0.58
LEVEL 4	STORY4	6	SPECYDUCTILE	0.0216	0.0021	0.059	0.0057	0.57
LEVEL 3	STORY3	6	SPECYDUCTILE	0.0149	0.0019	0.041	0.0052	0.52
LEVEL 2	STORY2	6	SPECYDUCTILE	0.0087	0.0016	0.024	0.0043	0.43
LEVEL 1	STORY1	6	SPECYDUCTILE	0.0036	0.0009	0.010	0.0026	0.26

EDLimit	ETLimit .55fy	ec=0.001	N* kN
0.68	0.34	0.36	399
0.69	0.43	0.28	744
0.59	0.49	0.22	1076
0.51	0.51	0.18	1420
0.46	0.46	0.16	1757

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	6	SPECYDUCTILE	0.0291	0.002148	0.080	0.0059	0.59
LEVEL 4	STORY4	6	SPECYDUCTILE	0.0222	0.002108	0.061	0.0058	0.58
LEVEL 3	STORY3	6	SPECYDUCTILE	0.0154	0.001943	0.042	0.0053	0.53
LEVEL 2	STORY2	6	SPECYDUCTILE	0.0091	0.001619	0.025	0.0045	0.45
LEVEL 1	STORY1	6	SPECYDUCTILE	0.0039	0.00101	0.011	0.0028	0.28

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	6	SPECYDUCTILE	0.0297	0.0022	0.082	0.0060	0.60
LEVEL 4	STORY4	6	SPECYDUCTILE	0.0227	0.0021	0.062	0.0059	0.59
LEVEL 3	STORY3	6	SPECYDUCTILE	0.0158	0.0020	0.044	0.0054	0.54
LEVEL 2	STORY2	6	SPECYDUCTILE	0.0094	0.0017	0.026	0.0046	0.46
LEVEL 1	STORY1	6	SPECYDUCTILE	0.0041	0.0011	0.011	0.0029	0.29

EDLimit	ETLimit .55fy	ec=0.001	N* kN
0.68	0.34	0.36	399
0.69	0.43	0.28	744
0.59	0.49	0.22	1076
0.51	0.51	0.18	1420
0.46	0.46	0.16	1757

T&T SOFT SOIL with 0.95kPa SDL

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	6	SPECYDUCTILE	0.0256	0.0019	0.0704	0.0051	0.51
LEVEL 4	STORY4	6	SPECYDUCTILE	0.0196	0.0018	0.0539	0.0050	0.50
LEVEL 3	STORY3	6	SPECYDUCTILE	0.0137	0.0017	0.0376	0.0047	0.47
LEVEL 2	STORY2	6	SPECYDUCTILE	0.0082	0.0012	0.0226	0.0034	0.34
LEVEL 1	STORY1	6	SPECYDUCTILE	0.0036	0.0009	0.0098	0.0026	0.26

EDLimit	ETLimit .55fy	ec=0.001	N* kN
0.68	0.34	0.36	399
0.69	0.43	0.28	744
0.59	0.49	0.22	1076
0.51	0.51	0.18	1420
0.46	0.46	0.16	1757

DA-10
[illegible]

Dead & live loads on line 2.

Gravity

D : 200 HB : 4.15kPa.
Ceiling : 0.20kPa.
SDL : 0.50

4.85kPa × 7.5 = 36.4
Beam: 350×400×24 kN/m³ = 3.4

D = 39.8 kN/m.

L_R: Live load

2.5kPa for offices Table 2 NZS4203:1984

Live load reduction factor R Fig C2.

A_1 = 7.5m × 7.0m = 52.5 m²/ bay

$\Rightarrow R = 0.3 + \frac{3.0}{\sqrt{52.5}} = 0.71$

$\Rightarrow L_R$ = 2.5kPa × 0.71 × 7.5m = 13.4 kN/m.
Applied uniformly over all relevant floor areas (cl. 1.3.2.2 NZS4203:1984).

E: Drifts are from DA_2 - 10

Column Shear Strength at D/2 : Cracked M/C Columns.

Concrete Shear Stress V_c cl 7.3.2.1(b) reinforcing.

$$\left(1 + \frac{3P_u}{A_g f_c'}\right)(0.07 + 10\rho_w)\sqrt{f_c'} \ngtr 3.5\sqrt{f_c'}\sqrt{1 + 0.002\frac{N}{A_g}}\,(psi)$$

(P&P Eqn 7.35b)

	f_c'	P_u	V_c	$V_{c\,max}$	V^*	V^*/ϕ	V_i reqd.
L5	25	399	1.38 MPa	2.01 MPa	119.6	140.7	1.01 ≮ 0.5
L4	25	744	1.75	2.40	109.4	128.7	0.92 ≮ 0.5
L3	25	1076	2.07	2.71	101.6	119.5	0.85
L2	30	1420	2.39	3.29	84.5	99.4	0.71
L1	35	1757	2.66	3.87	84.0	98.8	0.71

$\phi = 0.85$ cl. 4.3.1.2

$$\rho_w = \frac{A_s}{b_w d} = \frac{6 \times 314}{400 \times 350} = 0.0134$$

D = Diameter of column

c = cover to reinforcing steel

$$A_g = \frac{\pi D^2}{4} = \frac{\pi \times 400^2}{4} = 125664\,mm^2$$

$$v_c \ngtr \left(3.5\sqrt{\frac{f_c'}{0.00689}}\sqrt{1 + 0.002\left(\frac{P_u}{A_g}\right)\Big/0.00689}\right) \times 0.00689$$

(Eqn 7.35b P&P)

(1 psi = 0.00689 MPa)

$d = 400 - 50 = 350$

$b_w = 400$

$A_v = 400 \times 350 = 140000\,mm^2$

The requirement to provide minimum shear reinforcing was that the shear strength required to resist the applied shear was greater than half the shear strength provided by the concrete.

i.e. $v_i > 0.5 v_c$ cl.7.3.4.1

$v_i = \frac{V_u}{\phi}$ where $\phi = 0.85$ according to
cl.7.3.1.1 NZS3101:1982 cl 4.3.1.2.

These requirements are for members not required to be designed using the extra provisions for seismic design and were less than those requirements

For CTV Building however even these provisions were not complied with.

Minimum shear reinforcing steel according to these base provisions of cl7.3.4.3

$A_v = 0.35 \frac{b_w s}{f_y}$

for 6mm spirals. $f_y = 275$ MPa

$s = \frac{\left(\frac{\pi \times 6^2}{4}\right) \times 275}{0.35 \times 400}$ = 56mm minimum was required.

L4-L5 c.f. 250mm provided for the non-seismic requirements

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PROJECT NAME CTV Building. PROJECT NO PAGE L2-4
SUBJECT Line 2 Column Shear. DATE 2/8/12. BY CWH.

For the recommended case that columns be analysed for Ig uncracked the shear demands are much higher than the cracked case based on moment curvature analysis.

	f_c'	P_u	v_c	V^*	V^*/ϕ	$v_{i\,reqd}$	
L5	25	399	1.38	230.4kN	271.1kN	1.94	$\nless v_c$
L4	25	744	1.75	194.8	229.2	1.64	$\nless 0.5v_c$
L3	25	1076	2.07	182.7	214.9	1.54	"
L2	30	1420	2.39	157.3	185.1	1.32	"
L1	35	1757	2.66	95.3	112.1	0.80	$< 0.5v_c$.

Therefore L2 to L4 columns required the minimum shear re-forcing of R6 @ 55 c/c as $v_{i\,reqd} \nless 0.5\,v_c$ of column based on Ig as recommended. This is less than the requirements for the additional seismic design triggered for these columns.

L5 columns required the minimum requirements also.

$$A_v = \frac{(v_i - v_c)\,b_w s}{f_y} \quad \text{cl 7.3.6.3 NZS3101:1982.}$$

$$= \frac{(1.94-1.38)\,400 \times 55}{275} = 45\,mm^2/55\,mm.$$

R6 @ 55mm = 56mm²/55mm.

⇒ ok.

Column bending is affected by the assumptions made on cracked section properties with greater moments attracted to the stiffer uncracked columns than the cracked ones.

	Column Moments: I_g kNm.	$I_{ecracked}$ kNm.	Axial Action kN.	Bending Capacity (M/c). kNm.	Dependable Bending Capacity kNm.
L5	299.0	158.0	399	152	106 ⇒ NG
L4	265.4	147.2	744	179	125 ⇒ NG
L3	294.8	136.9	1076	190	133 ⇒ NG
L2	219.8	113.1	1420	194	136 ⇒ NG I_g; ok I_{cr}
L1	168.3	140.5	1757	191	134 ⇒ NG

$\phi = 0.7$ for columns without spiral reinforcing complying with 6.4.7.1(a), 6.4.7.2(a) or 6.5.4.3 according to cl 4.3.1.2(c).

From Fig 159 which shows maximum moments achievable for certain axial compression actions for $f_c = 27.5$ MPa (refer L2-6)

Axial Action.	Mont Max.
350	149 kNm
550	163
750	179
950	190
1150	191
1350	194
1750	191

Maximum bending capacities at axial compressions between these values can be derived by interpolation.

These capacities are upper bound capacities based on $\varepsilon_c = 0.004$ c.f. $\varepsilon = 0.003$ for design purposes.

Hyland

FATIGUE + EARTHQUAKE ENGINEERING

PROJECT NAME CTV Building

SUBJECT Line 2 Column Study

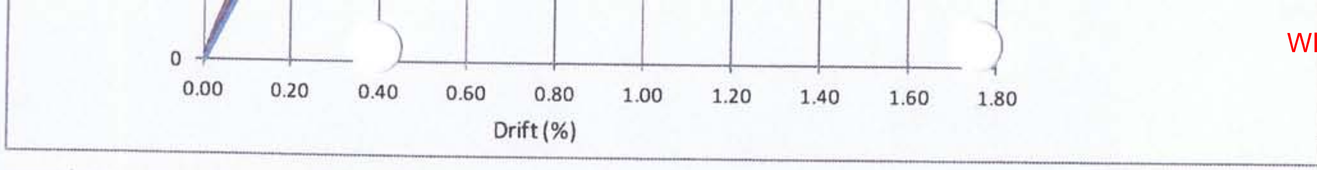
PROJECT NO

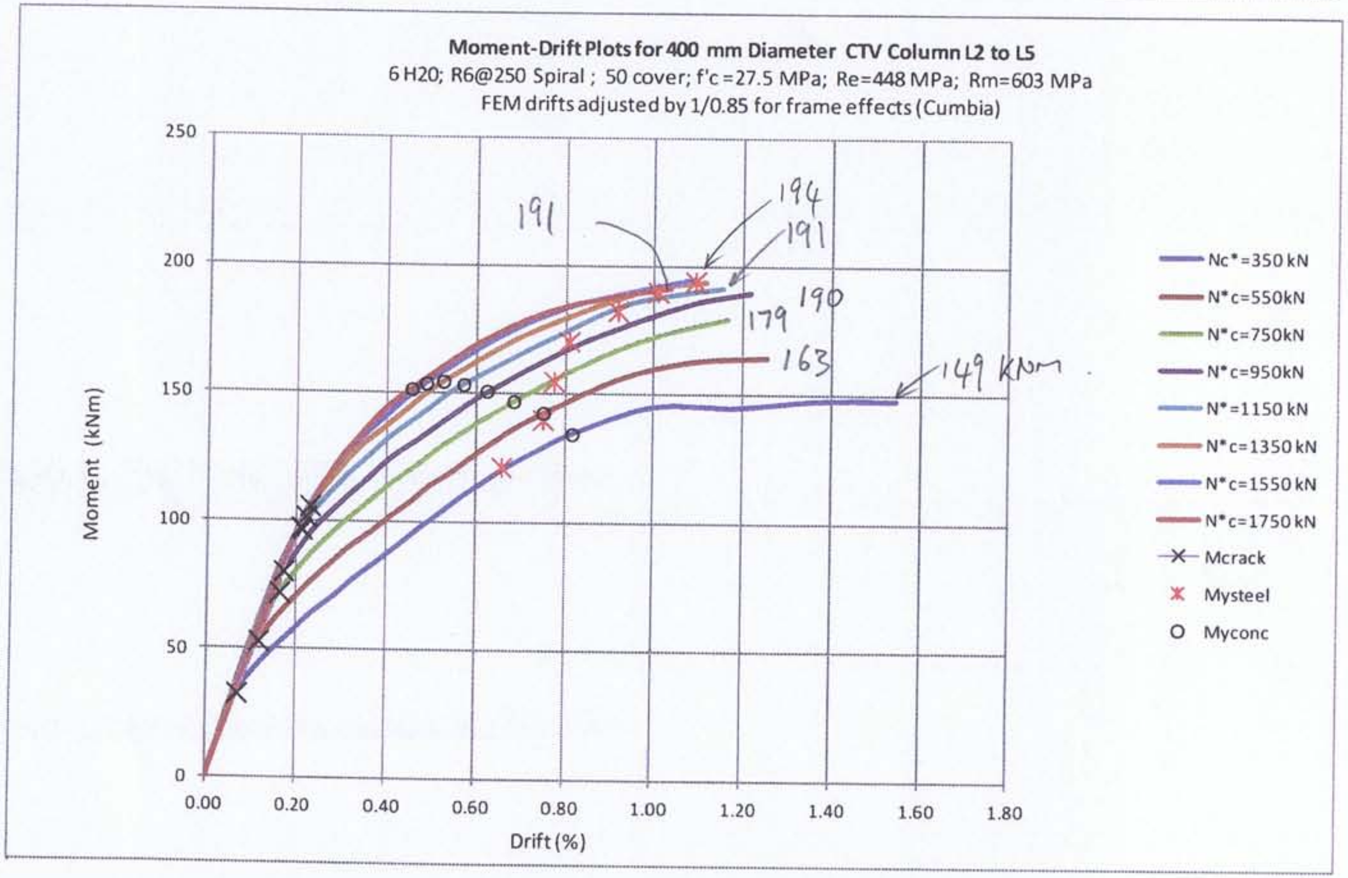
DATE 3/8/12

PAGE L2-6

BY Cwelt

The results show that none of the columns had sufficient dependable bending capacity to satisfy the demands imposed by the VS compatibility analysis even if analysed assuming cracked section properties in the columns.

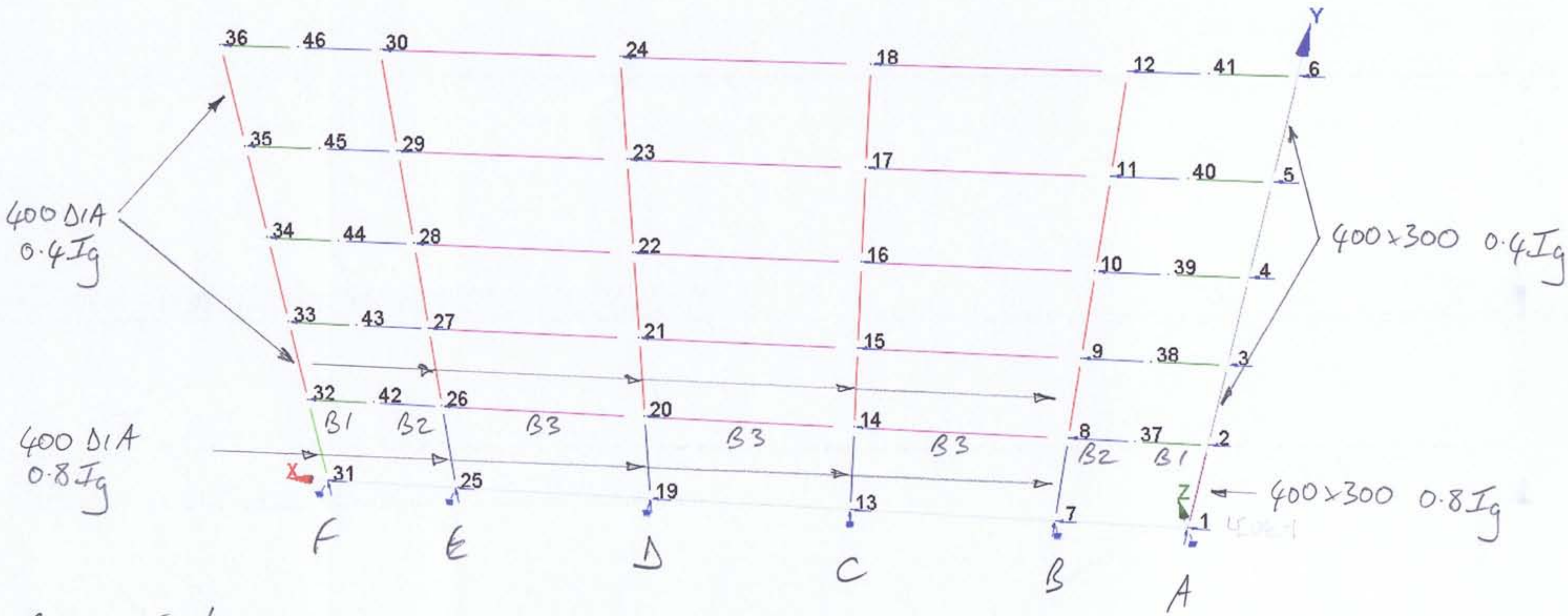




Note: These current capacities are greater than normally used in design as they are for εc = 0.004 rather than εc = 0.003 limiting compression strain.

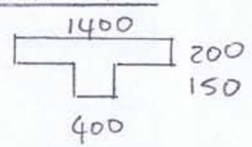
Figure 159 - Moment-Drift plots for 400 mm diameter CTV columns for f'c=14.2 and 27.5 MPa concrete, using Cumbia software for fixed end conditions adjusted for line 1, 4 and F frame effects. Concrete limiting strain was set at 0.004. This shows that yielding of the reinforcing steel starts at higher drifts as the axial compression action increases. Similarly the ability of the columns to drift more after starting to yield reduces as the axial compression action increases. Columns in the upper levels had lower axial compression actions compared to the lower level columns, and so were able to sustain more inelastic demand than those at lower levels. The crosses indicate the point at which yield of the extreme reinforcing steel bar occurs designated as the yield moment of the column. Due to the wide spacing of the spiral reinforcing, loss of concrete cover may have led to buckling of the H20 bars. For loads greater than 1550 kN the column drift capacity appears to reduce back along the upper drift curve back to squash capacity.

L2-7 Cent.

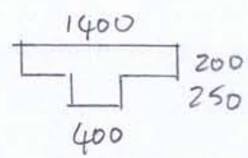


Beam Sections

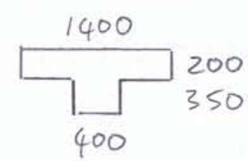
B1: 0.6Ig



B2: 0.6Ig



B3: 0.6Ig

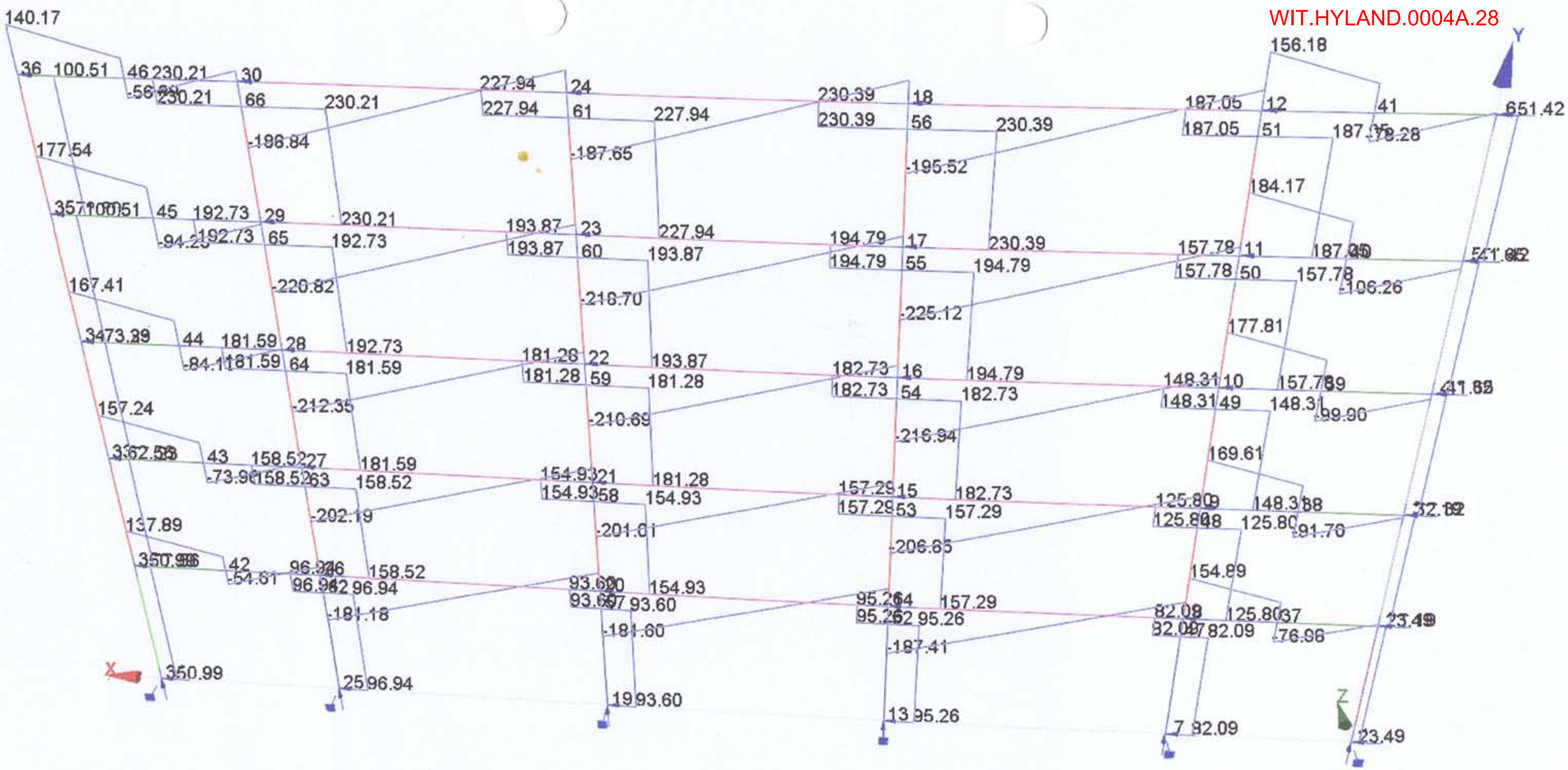


Concrete Properties

Columns : 27.5 MPa }
Beams : 24.6 MPa } Average tested strength.

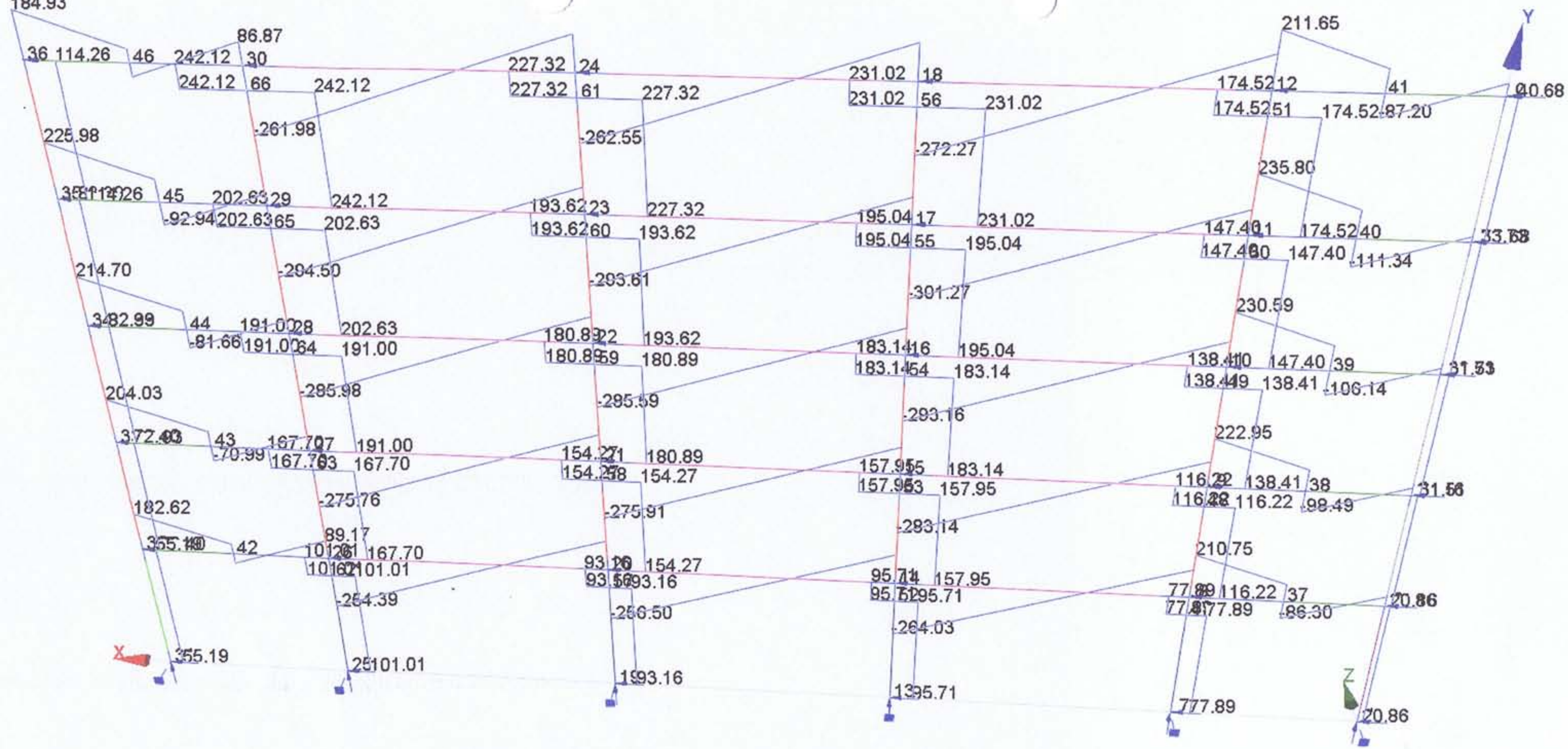
N.B.: For Uncracked Column Case
Columns Ig used.

CZA-1
Cecil.

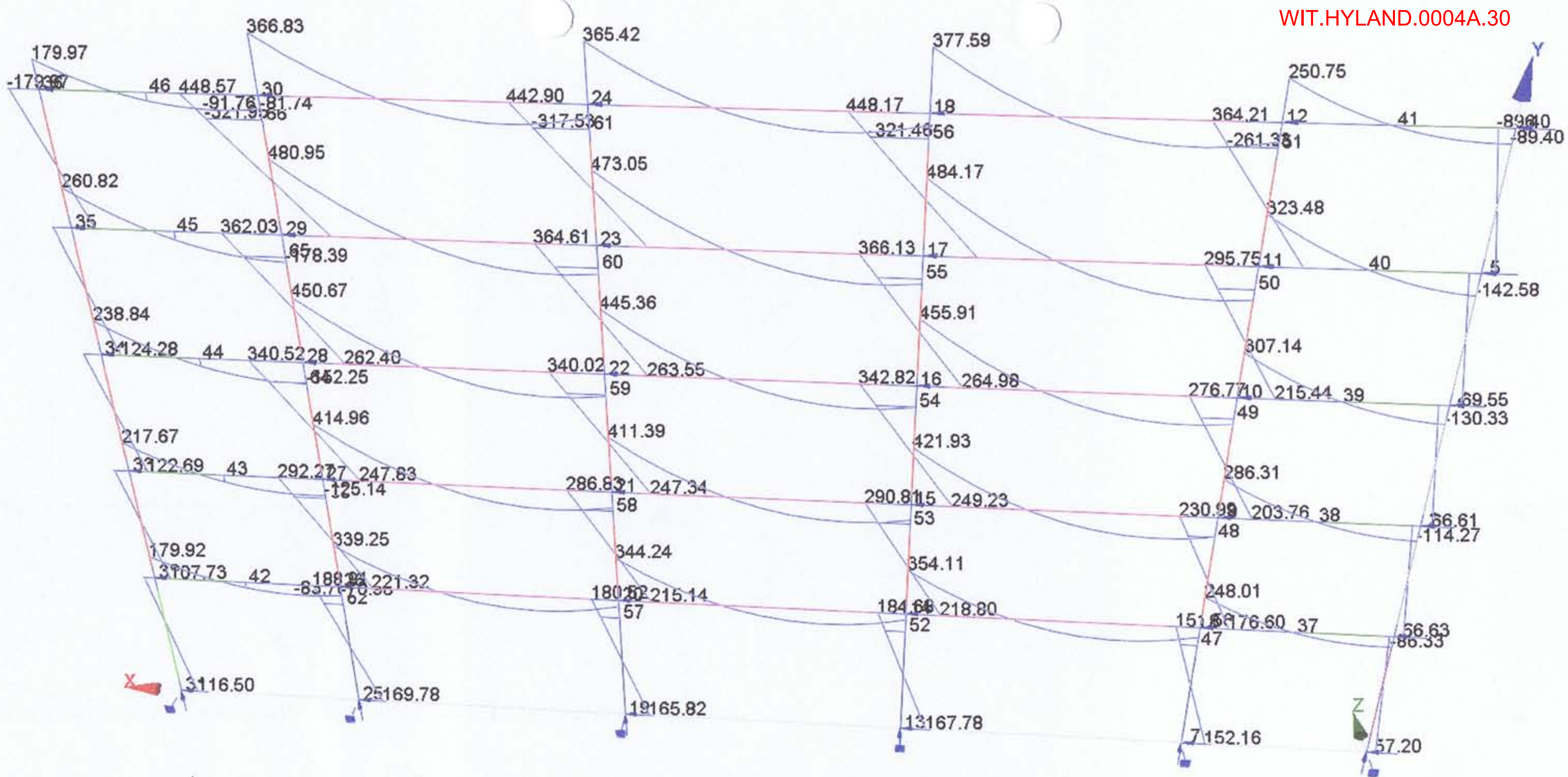


Line 2 shear : 0.9D+E (vδ = 55% ULS Drifts).
(C1)
Ig columns.

L2A – 1A
Cnw



Line 2: Shear : D+1.3Lr+E (Vd=55% ULS Drifts)
(C2)
Ig Columns.



Line 2: Bending Moments : 0.9D + E ($\nu\delta$ = 55% ULS Drifts)
(C1) Ig Columns.

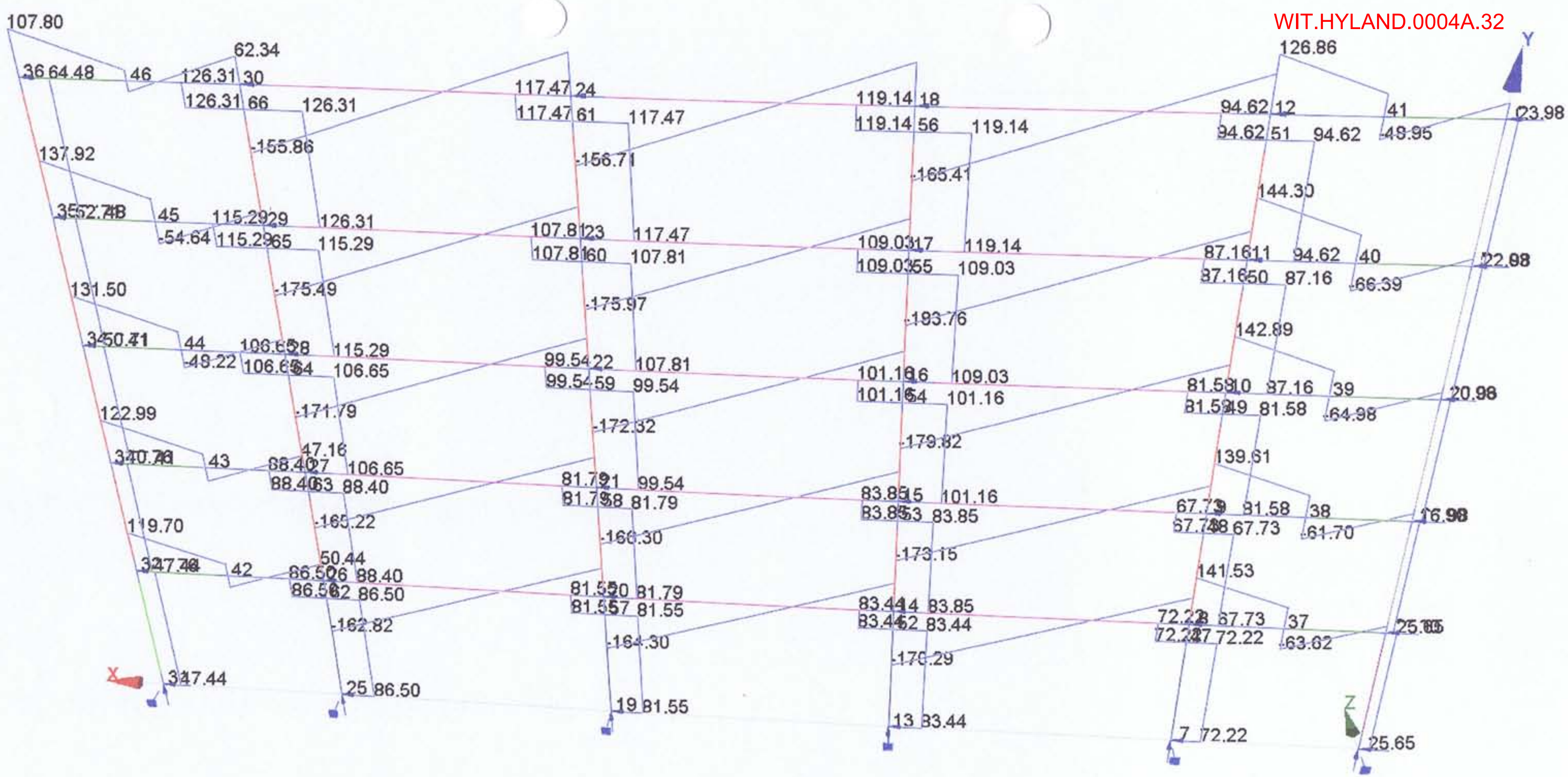
C2A-3 Cont'd

Line 2: Bending Moments : D+1.3Lr+E (vδ = 55% ULS Drifts)

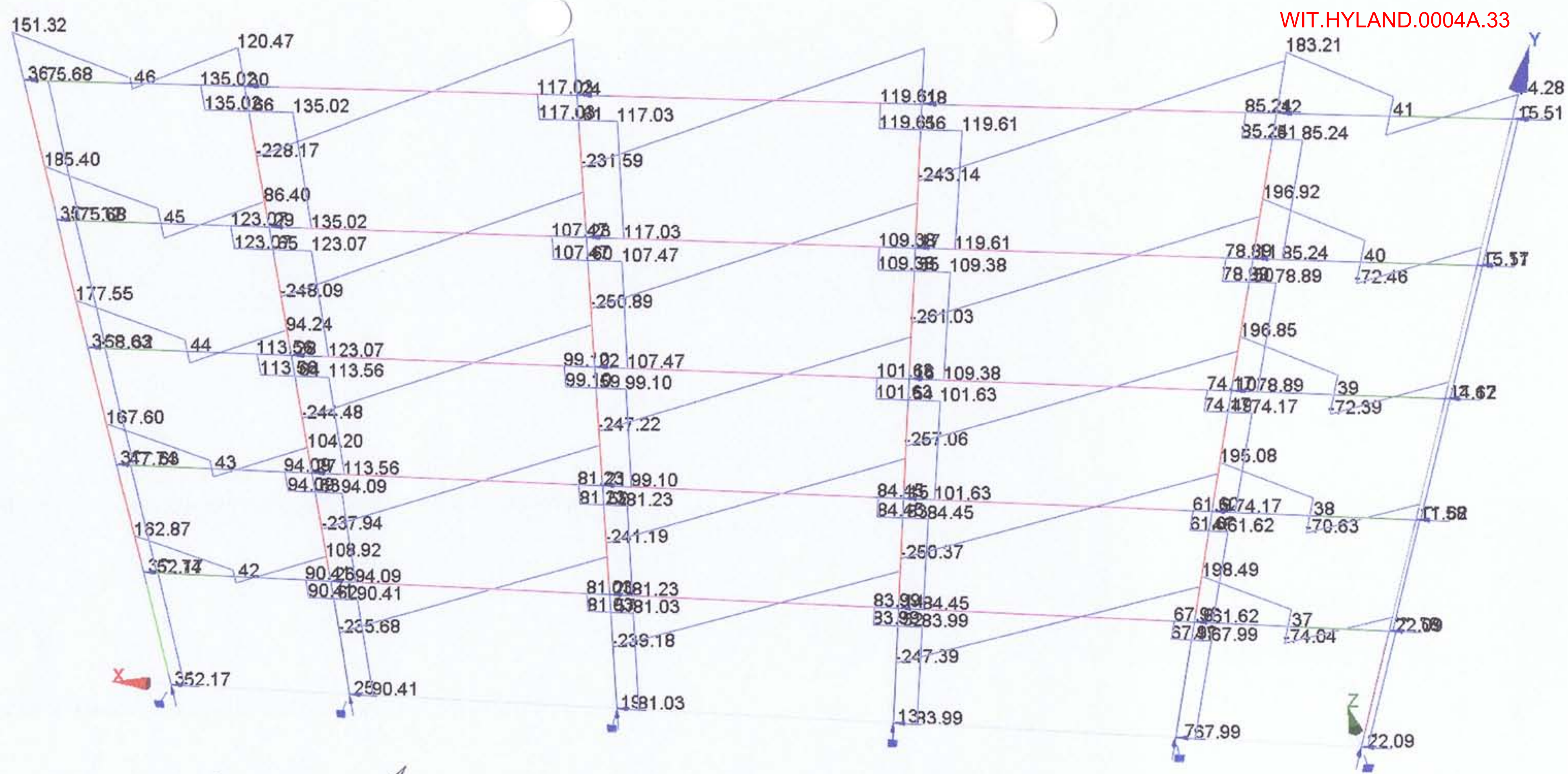
(C2)

Ig Columns.

C2A-4 Cont'd

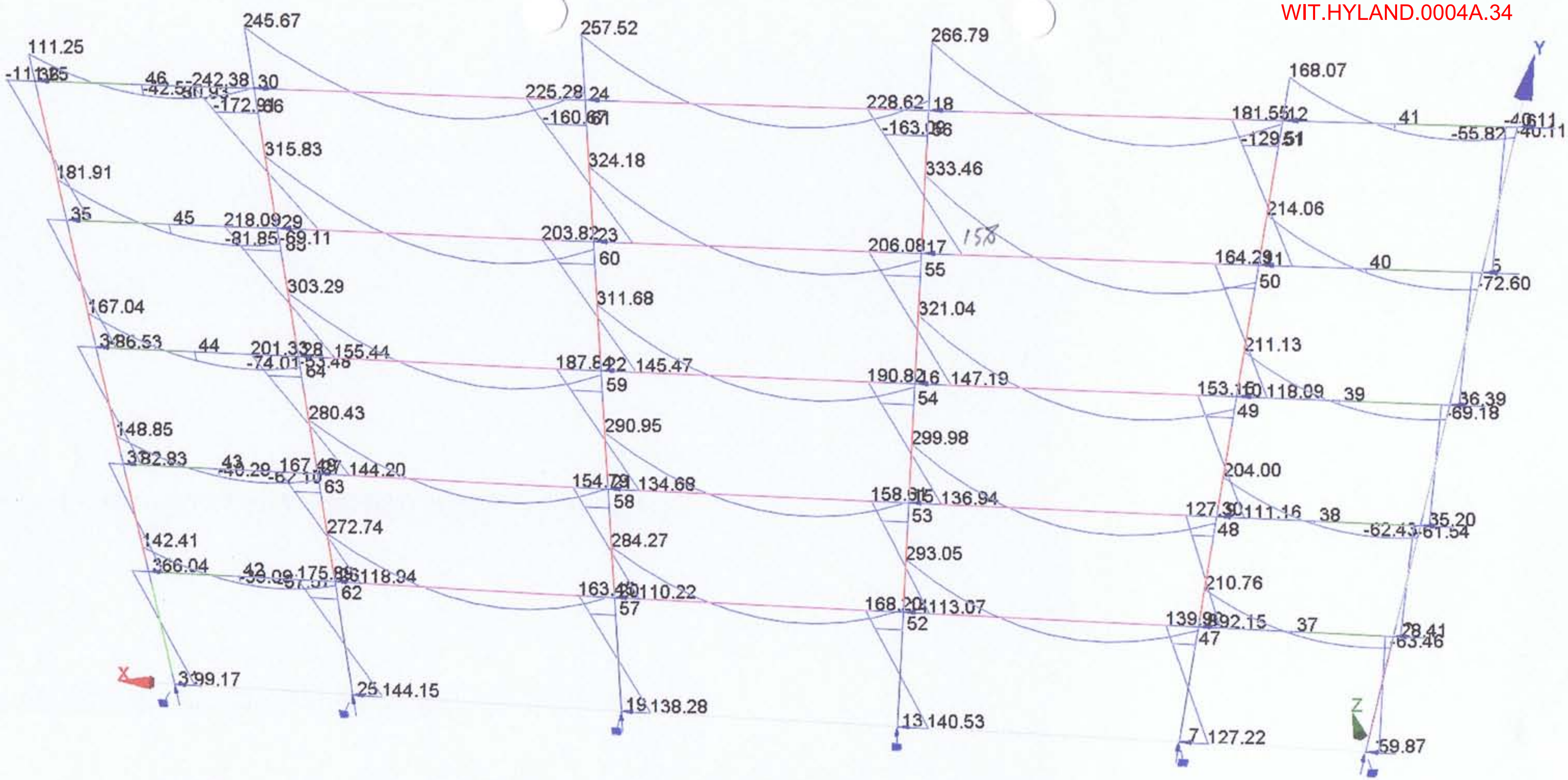


Line 2: Shear 0.9D + E. (VS = 55% ULS Drifts Icracked columns.)



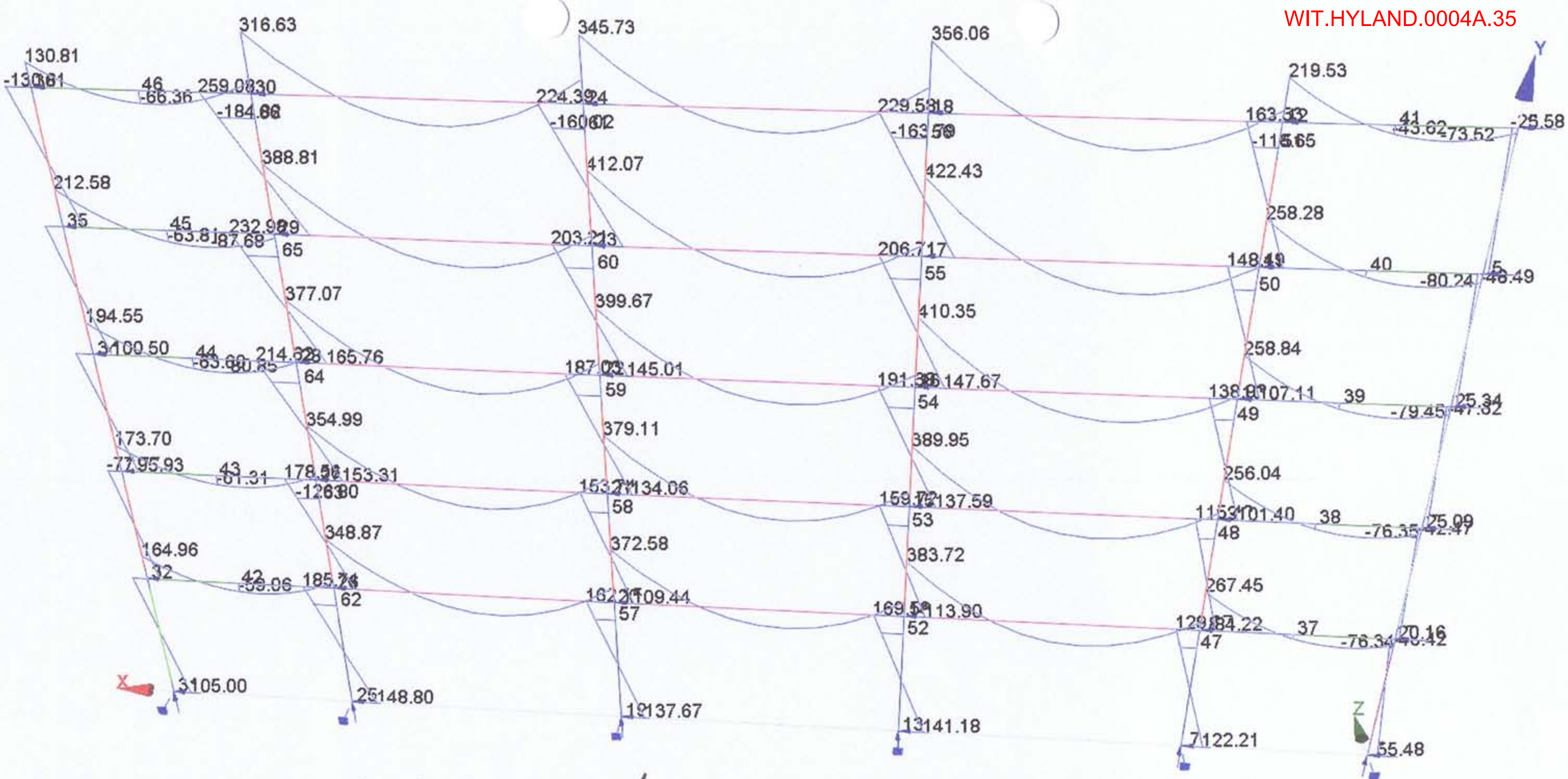
Line 2: Show D+1.3Lr+E (Sf = 55% ULS Drifts. Cracked columns.)

C2C-2



Line 2: Bending Moments 0.9D + E (Vδ = 55% ULS Drifts)
Icracked Columns.

L2C-3



Line 2: Bending Moments: D+1.3Lr+E (C2) (Vf = 55% ULS Drifts)
I cracked Columns.

Hyland
FATIGUE + EARTHQUAKE ENGINEERING

PROJECT NAME CTV Building PROJECT NO PAGE LF-1
SUBJECT Line F check. DATE 3/8/12 BY Cwilf

Line F Dead + Live Loads.

Δ: For 1200mm Trib width of Hibond slab

200 HB : 4.15 kPa × 1.2 = 4.98

960x 550 Beam: 0.96 × 0.55 × 24 = 12.10

Spandrel : (1.4 + 1.12) 0.1 × 24 kN/m^3 = 6.0

Framing : (3.24 − 0.8) 1.5 kPa = 3.7

26.8 kN/m.

LL: No live load reduction as small tributary width.
2.5 kPa office LL : 1.2m Trib width

$\Rightarrow$ ω_Q = 2.5 × 1.2m = 3.0 kN/m.

E:
Drifts are from DA-8 for "soft" soil subgrade stiffness per Tonkin & Taylor.

Hyland
FATIGUE+EARTHQUAKE ENGINEERING

PROJECT NAME CTU Building
SUBJECT Line F Columns Shear Check
PROJECT NO
DATE 3/8/12
PAGE LF-2
BY [illegible]

Column Shear Strength at F/2 + F/3:
for Cracked columns based on Mont Cancelm

Concrete shear stress $v_c = \left(1 + \frac{3P_u}{A_g f'_c}\right)(0.07 + 10 p_w)\sqrt{f'_c}$

NZS3101:1982 cl 7.3.2.1(b)

$\rho_s = \frac{A_s}{b_w d} = \frac{6 \times 314}{400 \times 350} = 0.0134$

$A_g = \frac{\pi 400^2}{4} = 125664 \text{ mm}^2$ & $\phi = 0.85$ cl 4.3.1.2

$A_v = 400 \times 350 = 140000 \text{ mm}^2$ c.

	f'_c MPa	P_u kN	v_c MPa	V^* kN	V^*/ϕ	$v_{i\,reqd}$	
L5	25	269	1.26	137.4	161.6	1.15	$\nleq 0.5 v_c$
L4	25	513	1.52	133.2	156.7	1.12	$\nleq 0.5 v_c$
L3	25	754	1.76	128.5	151.2	1.08	$\nleq 0.5 v_c$
L2	30	995	2.01	113.8	133.9	0.96	$> 0.5 v_c$
L1	35	1245	2.24	184.0	216.5	1.55	$\nleq 0.5 v_c$

⇒ Min shear reinforcing requirements of cl. 7.3.4.3 are required for columns at L1, 3, 4, 5 not including the additional requirements for seismic design triggered

⇒ R6 @ 55 c/c minimum requirements for line F even with cracked columns properties assumed in analysis

The shear demands on the columns are much higher if uncracked column properties are used in the analysis.

	f_c' MPa	P_u kN	V^* kN	v_c MPa	$\frac{V^*}{\phi}$	$v_{c\,reqd}$	
C5	25	269	363.4	1.26	427.5	3.05	$\nleq v_c$
C4	25	513	362.5	1.52	426.5	3.05	$\nleq v_c$
C3	25	754	321.8	1.76	378.6	2.70	$\nleq v_c$
C2	30	995	300.8	2.01	353.9	2.53	$\nleq v_c$
C1	35	1245	223.3	2.24	262.7	1.88	$\nleq 0.5v_c$

For columns C2 to C5 shear reinforcing steel spirals greater than the minimum specified were required

<u>C4 & C5</u> $A_v = \frac{(v_c - v_c)\,b\,w\,s}{f_y}$ cl 7.3.6.3 NZS 3101:1982

$= \frac{(3.05-1.26)\,400 \times 55}{275} = 143.2\ \text{mm}^2$

$\Rightarrow$ <u>R10 @ 55 c/c</u>

<u>C3</u> $A_v = \frac{(2.70 - 1.76)\,400 \times 55}{275} = 75\ \text{mm}^2 / 55\text{mm}$

$\Rightarrow$ <u>R10 @ 55 c/c</u>

$\Rightarrow$ (R6 ± 56 mm²/55 mm)

$\Rightarrow$ <u>If uncracked columns used in analyses then R10 @ 55 spirals reqd C3 to C5 & the min reqd C1 & C2.</u>

Hyland

FATIGUE+EARTHQUAKE ENGINEERING

PROJECT NAME CTV Building PROJECT NO PAGE LF-4

SUBJECT Line F Column Bending Check DATE 3/8/12 BY Carroll

Column bending demands are affected by the assumptions made about cracked section properties, with greater moments & shear actions attracted to stiffer uncracked columns than to cracked columns.

Column	Moments for I_g uncracked kNm	Moments for I cracked kNm	Axial Action kN	Bending Capacity (M/C) kNm	Dependable Bending Capacity kNm
L5	501.8	186.1	269	<149	<104.3 => NG
L4	440.3	179.3	513	163	114.1 => NG
L3	431.3	172.9	754	179	125.3 => NG
L2	399.5	154.1	995	190	133.0 => NG
L1	369.5	299.6	1245	193	135.1 => NG

Refer fig 159 BCR & p. LZ-7 for column bending capacities at specific axial compression actions for $\varepsilon_c = 0.004$. These capacities will be greater than allowed for design based on limiting compression strain of $\varepsilon_c = 0.003$, so are an upper bound for the purposes of this check.

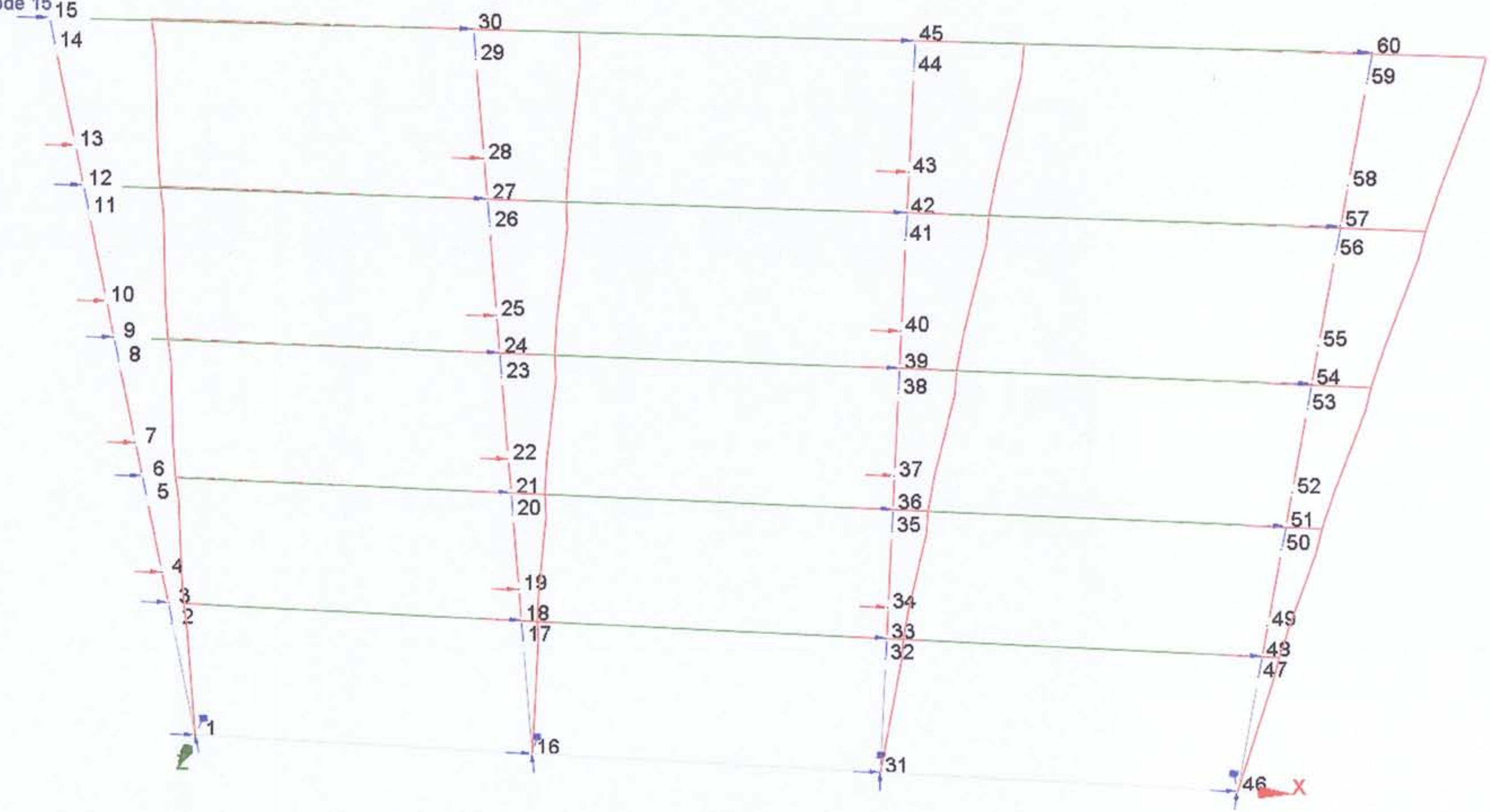
Dependable bending capacity reduces the ideal capacity by $\phi = 0.7$ to cl. 4.3.1.2(c) NZS 3101:1982.

Hyland
FATIGUE+EARTHQUAKE ENGINEERING

PROJECT NAME CTV Building. PROJECT NO PAGE CF-5
SUBJECT Line F Columns. DATE 3/8/12 BY Cwlf.

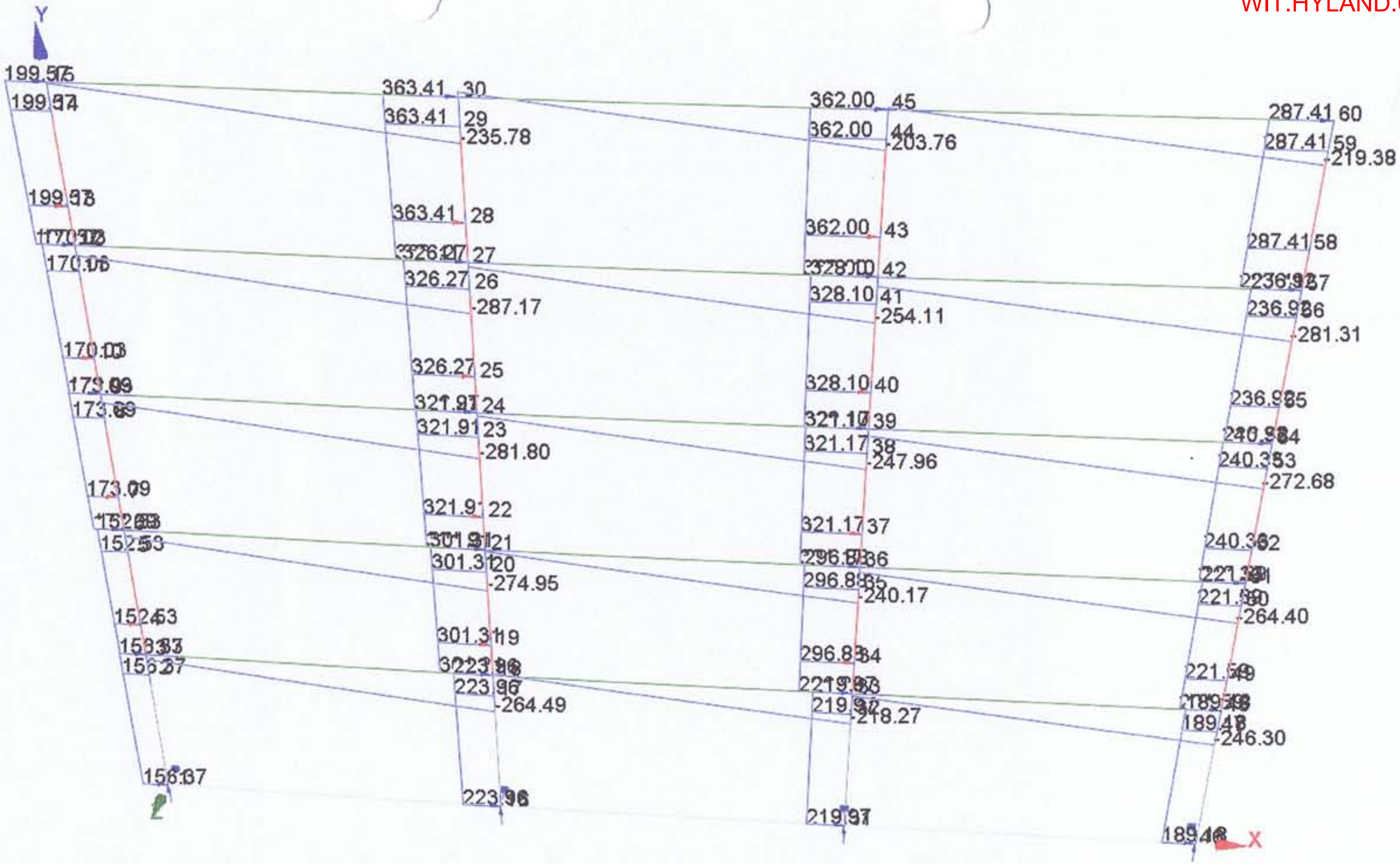
In summary the columns on line F had insufficient dependable bending capacity to cope with the specified Group 2 frame design drifts whether uncracked or cracked section properties were used for the frame analysis.

Maximum Deflections for Load Case E:
X :121.50 mm at node 60
Y :2.18 mm at node 15



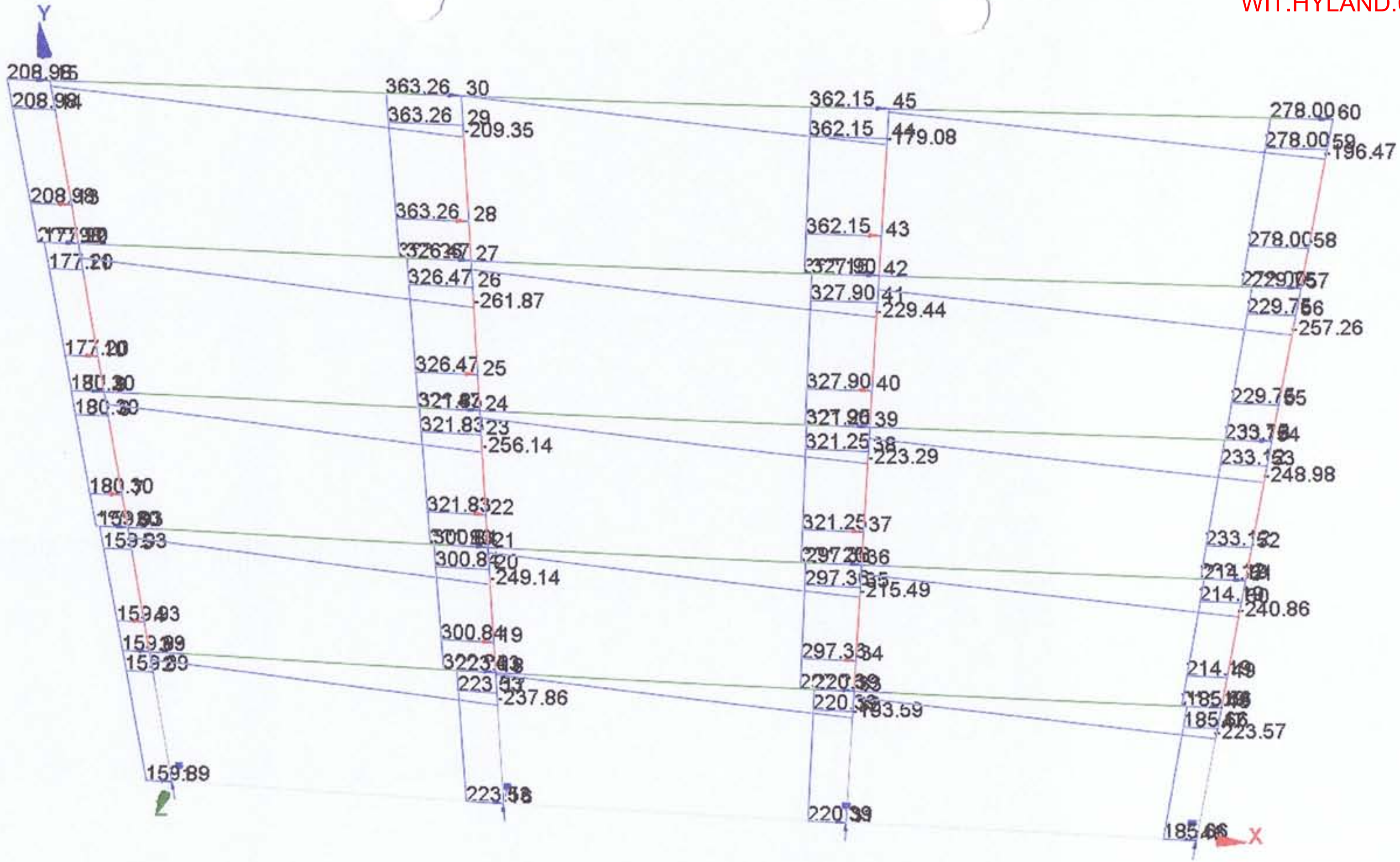
Line F : $\nabla\delta$ = 55% ULS drifts.
No Spandrel Panel Contact.

CFA-1



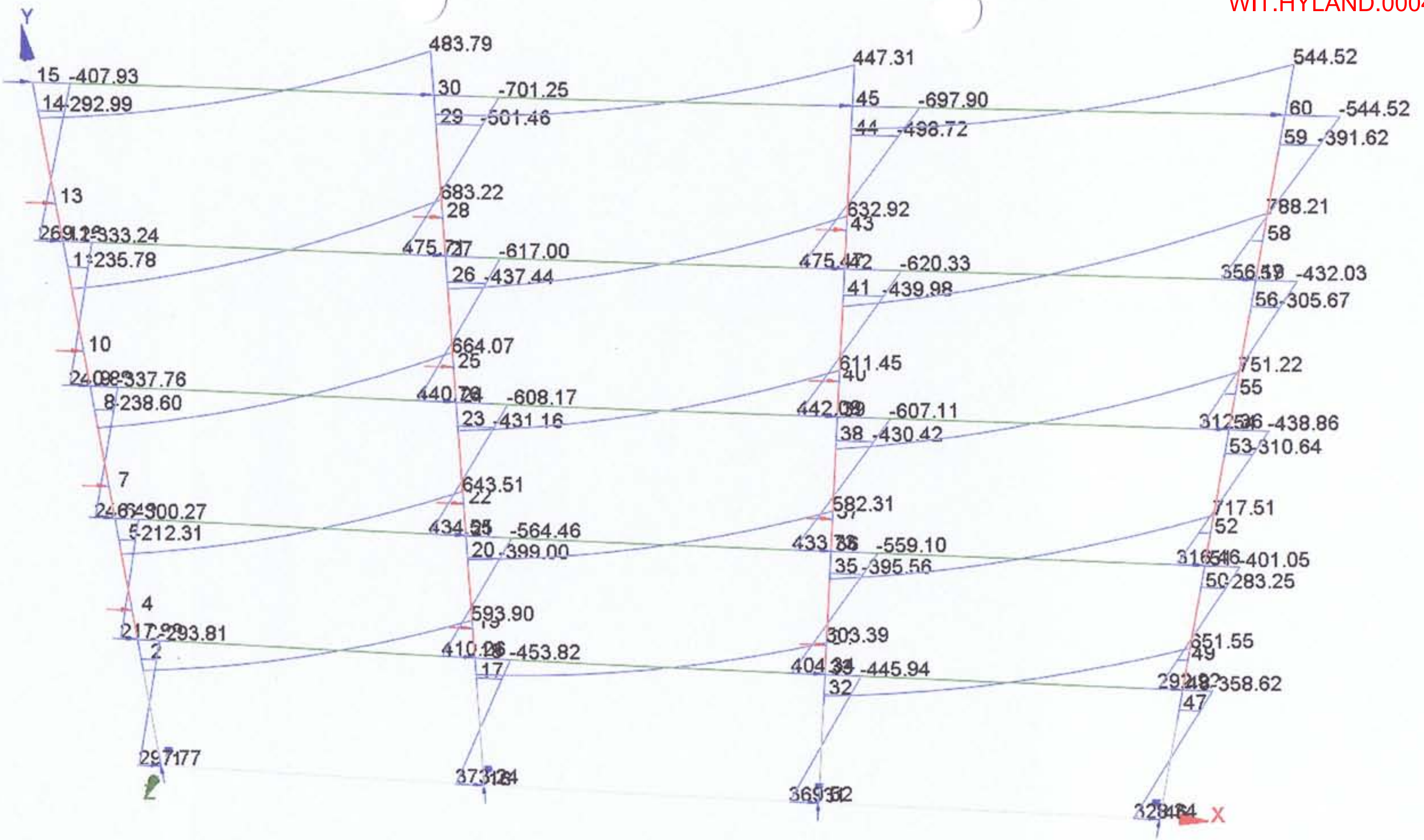
Line F: Ig uncracked columns
Shear D + 1.3LR + E (νδ = 55% ULS drifts imposed).
No spandrel contact

LFA-2

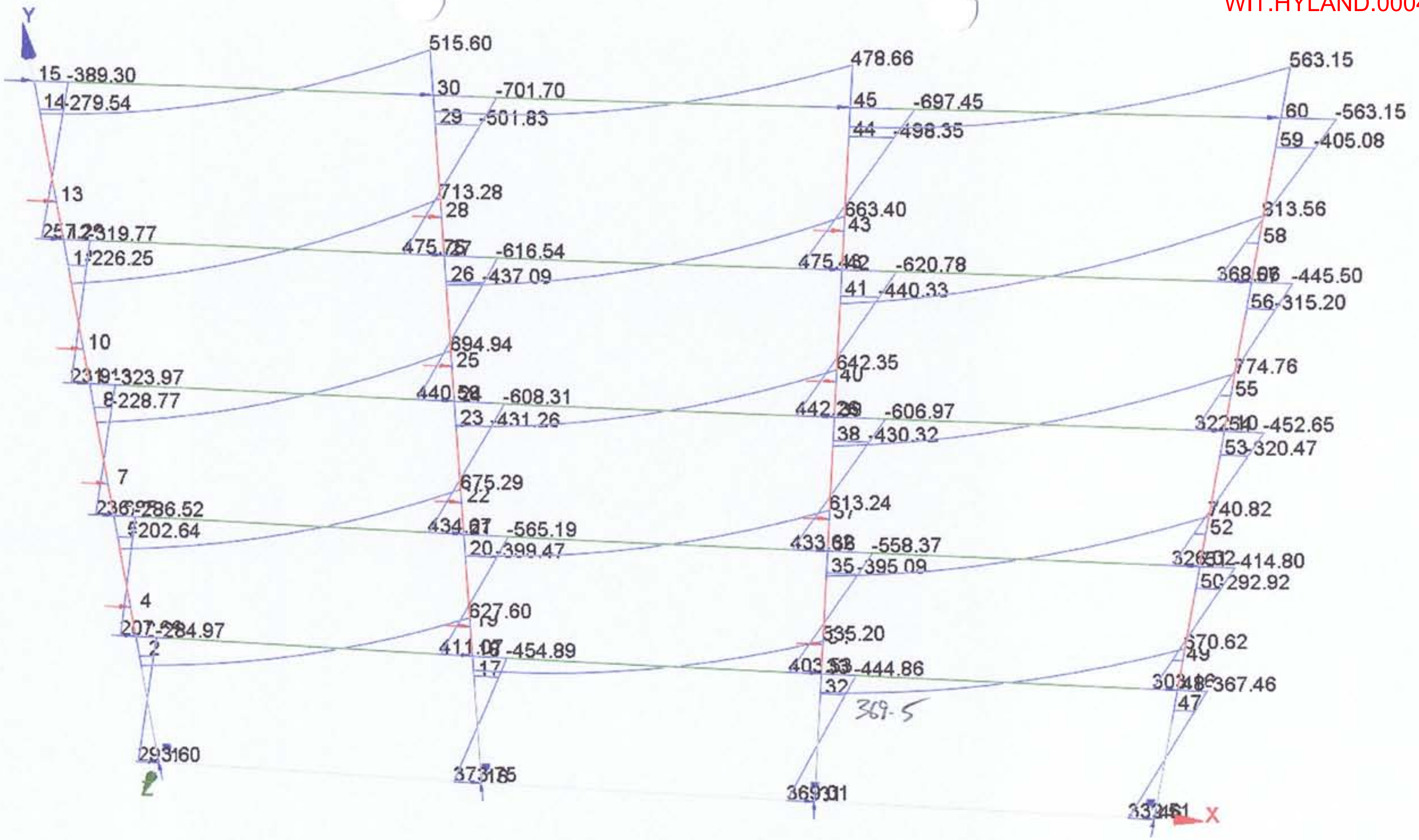


Line F: Ig uncracked columns
Shear 0.9D + E (ν δ = 55% UCS drifts imposed)
No spandrel contact.

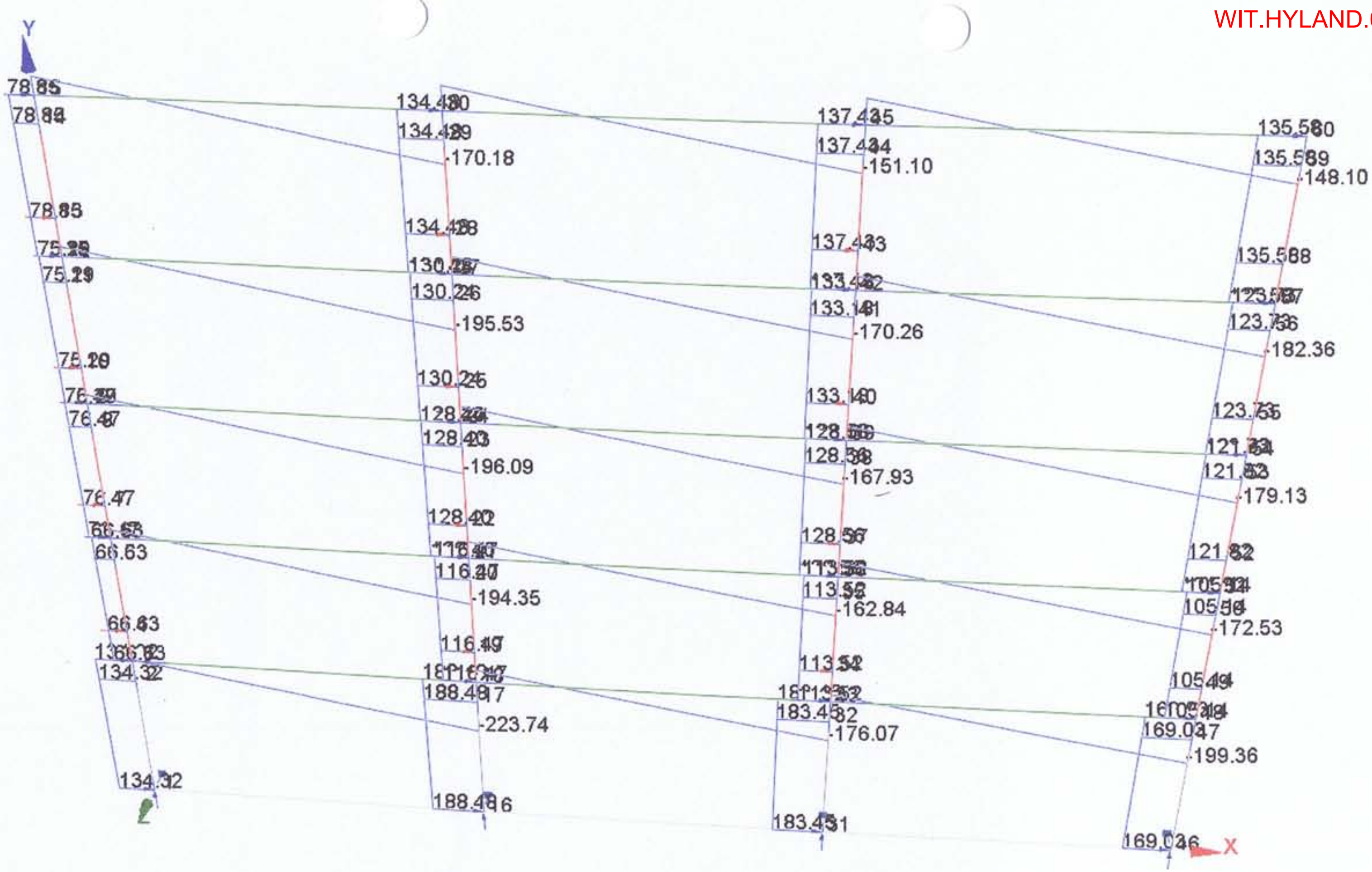
CFA-3
Cuts



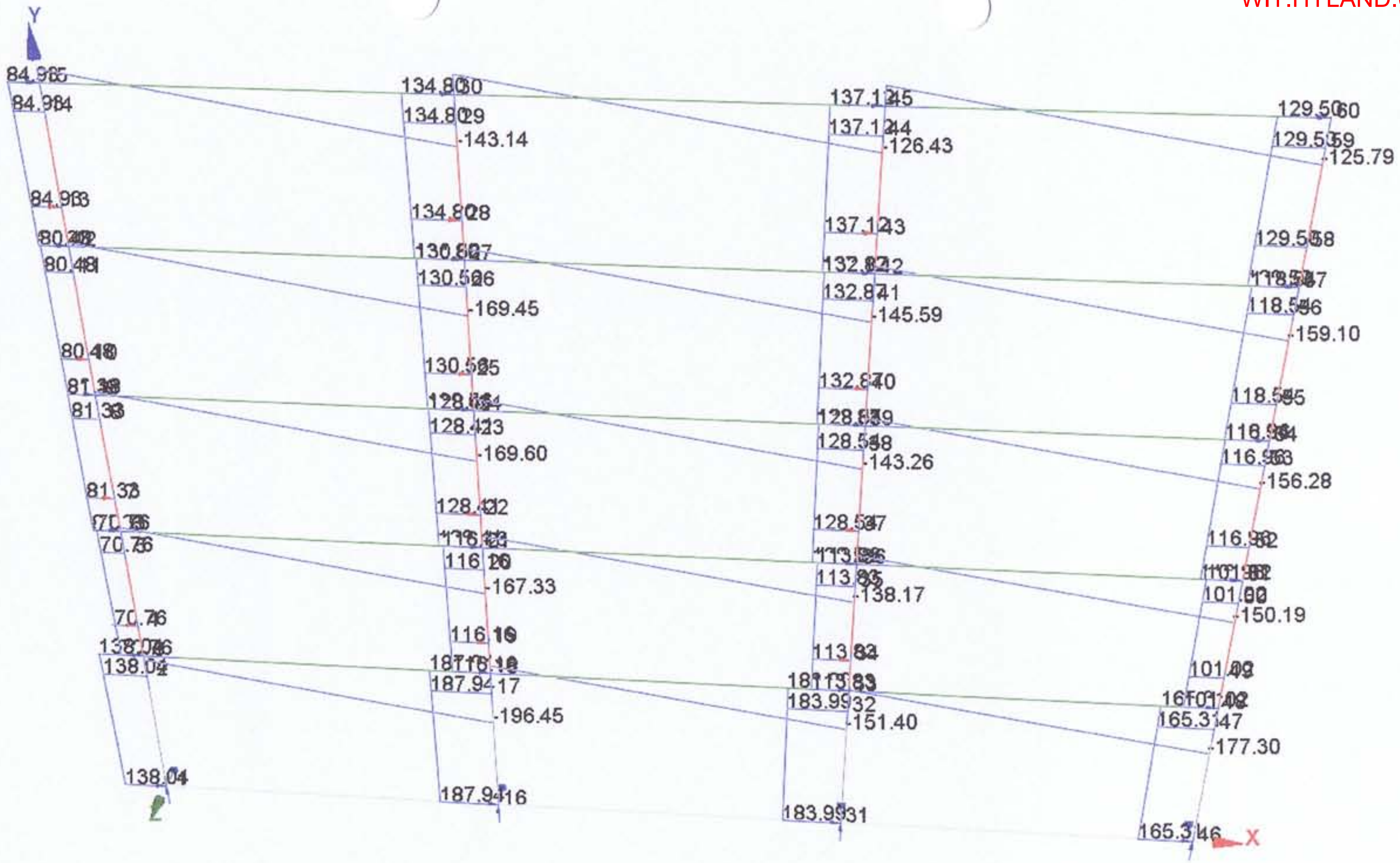
Line F: Ig uncracked columns ($v\delta$ = 55% ULS imposed Drifts)
Bending 0.9D + E.
No spandrel panel contact.



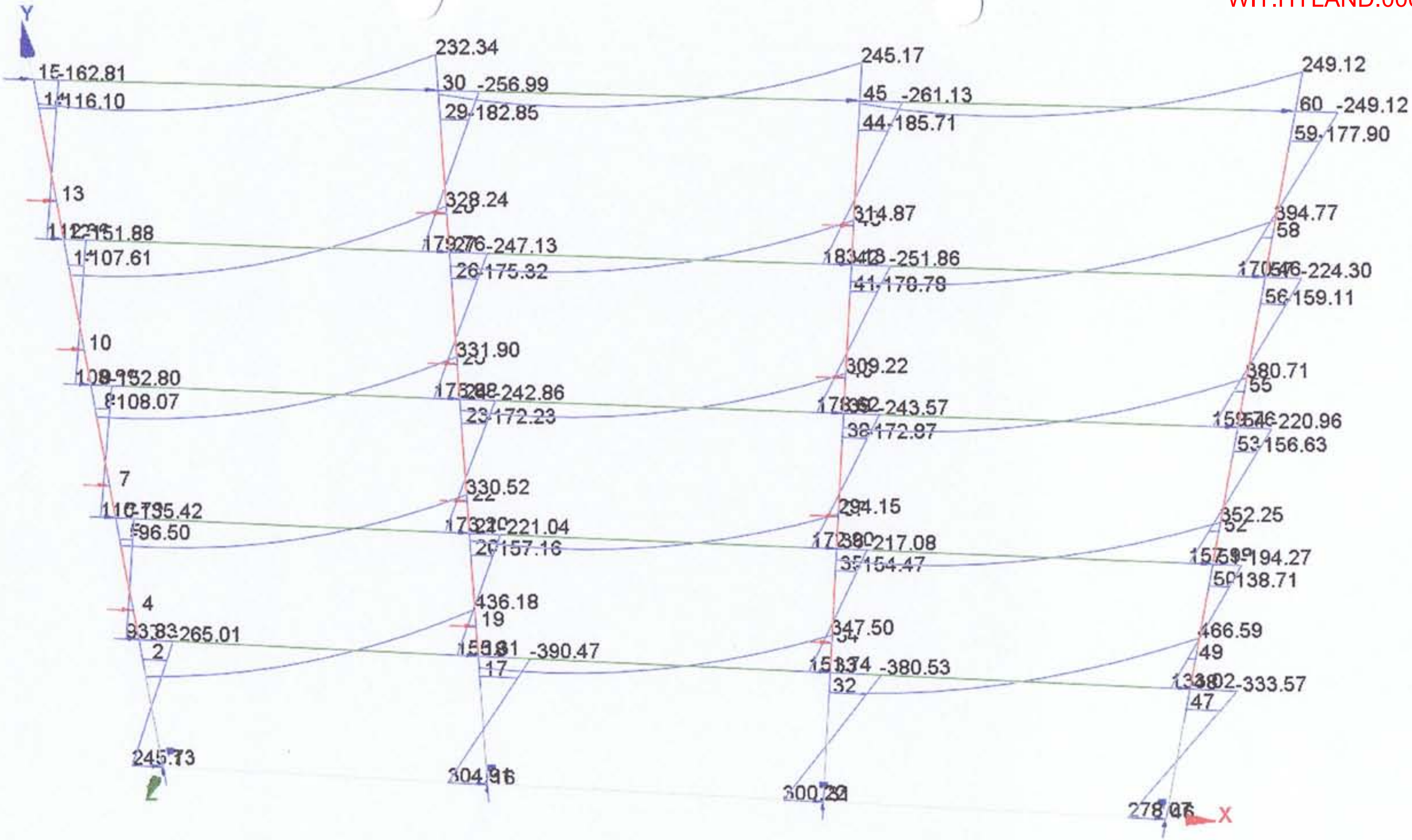
Line F: Ig uncracked columns
Bending D+1.3LR+E (VΔ=55% ULS imposed Drifts).
No spandrel panel contact.



Line F : Icol cracked (VS = 55% Drifts imposed).
Shear 0·9D+E
no spandrel panel contact.

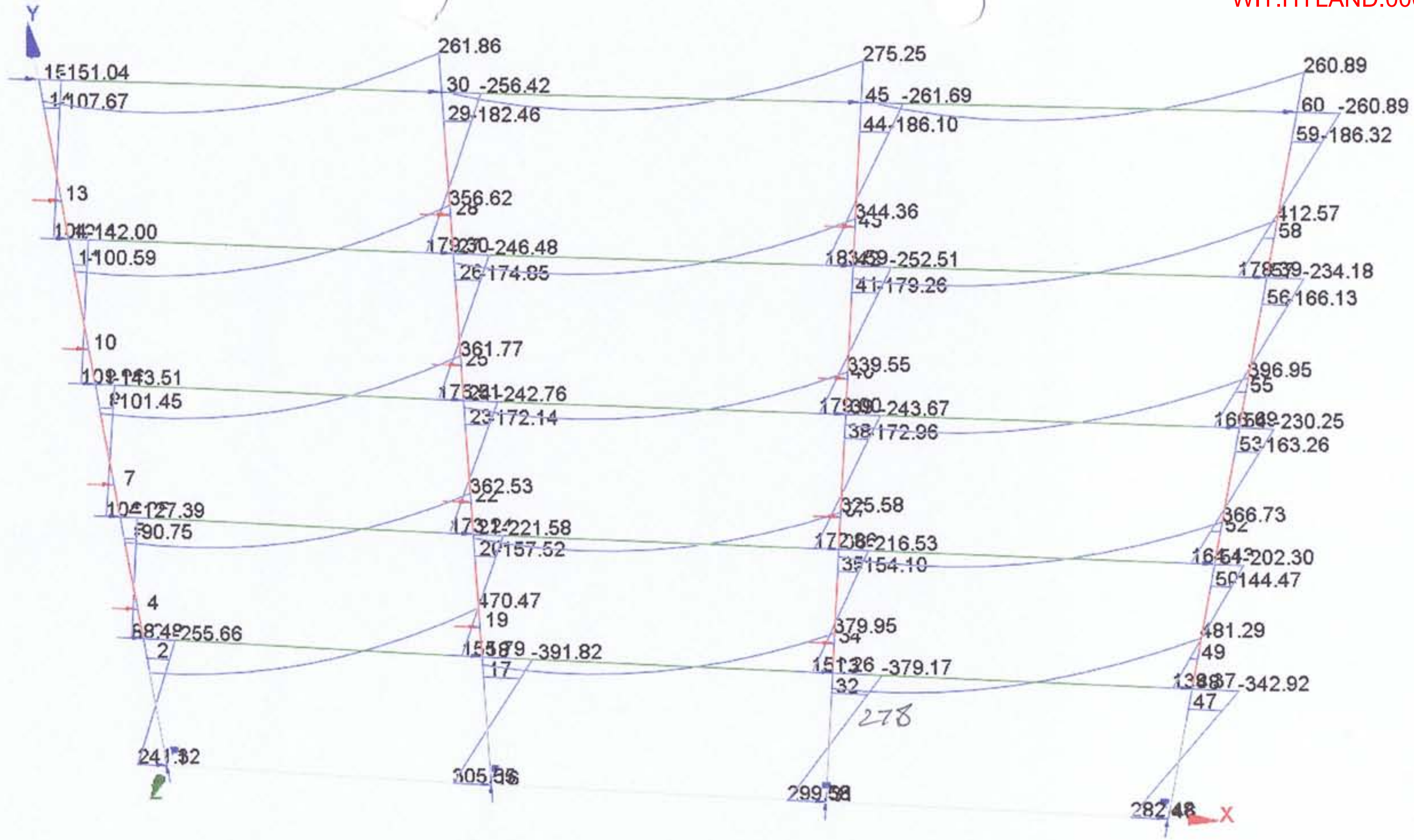


Line F: Icol cracked (ν δ = 55% ULS Drifts imposed).
Shear D+1.3LR+E
No spandrel panel contact.



Line F: Cracked I_{col}
Bending 0.9D + E ($\nu\delta$ = 55% ULS Drifts imposed)
No spandrel panel contact.

UFC-3 Cont.



Line F: Cracked I for Columns
Bending D+1.3LR+E (≈δ=55% ULS Drifts).
No spandrel panel contact

CFC-4 Coverly

PP-1

Reinforced Concrete Structures

R. PARK and T. PAULAY

Department of Civil Engineering,
University of Canterbury, Christchurch,
New Zealand

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PP-2

I

The Design Approach

1.1 DEVELOPMENT OF WORKING STRESS AND ULTIMATE STRENGTH DESIGN PROCEDURES

Several of the early studies of reinforced concrete members were based on ultimate strength theories, for example, Thullie's flexural theory of 1897 and the parabolic stress distribution theory of Ritter in 1899. However at about 1900 the straight-line (elastic) theory of Coignet and Tedesco became generally accepted, mainly because elastic theory was the conventional method of design for other materials and also because it was thought that the straight-line distribution of stress led to mathematical simplification. In addition, tests had shown that the use of elastic theory with carefully chosen values for the allowable working stresses led to a structure displaying satisfactory behavior at the service loads and having an adequate margin of safety against collapse. Thus elastic theory has been the basis of reinforced concrete design for many years.

Recently there has been renewed interest in ultimate strength theory as a basis of design. After more than half a century of practical experience and laboratory tests, the knowledge of the behavior of structural concrete has vastly increased and the deficiencies of the elastic theory (working stress) design method have become evident. This has resulted in periodic adjustment to the working stress design method, but it has become increasingly apparent that a design method should be based on the actual inelastic properties of the concrete and steel. Thus ultimate strength design became accepted as an alternative to working stress design in the building codes for reinforced concrete of the American Concrete Institute (ACI) in 1956 and of the United Kingdom in 1957. These two design approaches may be summarized as follows.

Working Stress Design (Elastic Theory)

The sections of the members of the structure are designed assuming straight-line stress-strain relationships ensuring that at service loads the stresses in

PP-3

perties

= 35 in^2

= 288 + 35 = 323 in^2

ne top edge to find $\bar{y}$:

× 20)   ∴ $\bar{y} = 12.87$ in

× 0.87^2) + (35 × 7.13^2)

ne neutral axis to find $\bar{y}$.

= 0   ∴ $\bar{y} = 8.68$ in

× 8.68 × 4.34^2) + (40 × 11.32^2)

that the transformed area of steel ...
e the moment of inertia about its ...
Also, for the section of this example ...
uce the I value by 51 %.

pture of concrete

320

· in

1. $M = 300{,}000$ lb · in; therefore, section is uncracked.

$$f = \frac{My}{I}$$

$$\text{top } f_c = 300{,}000 \times 12.87/15{,}820 = 244 \text{ psi } (1.68 \text{ N/mm}^2)$$
$$\text{bottom } f_c = 300{,}000 \times 11.13/15{,}820 = 211 \text{ psi } (1.46 \text{ N/mm}^2)$$
$$f_s = nf_c = 8 \times 300{,}000 \times 7.13/15{,}820$$
$$= 1080 \text{ psi } (7.45 \text{ N/mm}^2)$$

2. $M = 900{,}000$ lb · in; therefore, section is cracked.

$$\text{top } f_c = 900{,}000 \times 8.68/7740 = 1010 \text{ psi } (6.96 \text{ N/mm}^2)$$
$$f_s = 8 \times 900{,}000 \times 11.32/7740 = 10{,}530 \text{ psi } (72.6 \text{ N/mm}^2)$$

Note the significant increase in steel stress after cracking. When cracking occurs at a bending moment of 639,600 lb · in (72.2 kN · m) the maximum concrete stress increases from 520 to 720 psi (3.6 to 5.0 N/mm^2) and the steel stress increases from 2300 to 7480 psi (15.9 to 51.6 N/mm^2).

10.2.5 Design of Beams Using the Alternative (Elastic Theory) Method

The 1971 ACI code[10.1] allows the design of flexural members without [illegible]al load by the elastic theory approach (straight line theory). This design method proportions members so that at the service loads the specified allowable stresses are not exceeded. The allowable compressive stress in the concrete is $0.45f'_c$. The allowable tensile stress in the steel is 20,000 psi (138 N/mm^2) for Grade 40 or Grade 50 steel ($f_y = 276$ or 345 N/mm^2), or 24,000 psi (166 N/mm^2) for Grade 60 steel ($f_y = 414$ N/mm^2) or higher grade steel. The modular ratio n stipulated by the code is E_s/E_c, except that in doubly reinforced members, an effective modular ratio of $2E_s/E_c$ is used when considering the compression steel. The value taken for E_s is 29×10^6 psi (200,000 N/mm^2), and for both normal weight and lightweight concrete E_c = $57{,}000\sqrt{f'_c}$ psi ($4730\sqrt{f'_c}$ N/mm^2). The modular ratio n may be taken as the nearest whole number.

The recommended value for the modular ratio ignores the effect of concrete creep except as it effects the compression steel, whereupon the creep coefficient is taken as $C_t = 1$. The reason for this is revealed in Example 10.1. Comparison of stresses after creep with stresses before creep indicates a very insignificant increase in the compressive steel stress, but only a very slight increase in the tensile steel stress and a decrease in the compressive concrete stress. This approach means that when a doubly reinforced section is first loaded, the concrete will be more highly stressed than calculated, but the

(b) The requirements of Section 14, wherever the actions that could be transmitted by the superstructure at the top of the foundations are equal or larger than those which would result from the application of lateral earthquake loading to the superstructure corresponding with $SM = 1.6$.

3.5.12.5 *Rocking foundations.* When special studies are carried out to the satisfaction of the Engineer, structural walls may be assumed to limit the seismic loads induced in the structure by rocking with their foundations, provided that:

(a) The vertical design loads on the foundations are determined from factored gravity loads together with overstrength contributions of adjacent slabs, beams and other elements which may be yielding during the rocking of the wall system, and having regard to all accelerations induced in the superstructure during rocking

(b) The lateral design load acting simultaneously with the vertical forces, in accordance with 3.5.12.4 (a), are determined from special studies.

3.5.12.6 *Lateral forces on retaining walls and piles.* Particular attention shall be given to forces that might develop against retaining walls and piles during earthquakes.

3.5.12.7 *Uplift forces.* Uplift forces that may act on foundation pads during earthquakes, shall be considered to ensure that, when necessary, adequate flexural tension reinforcement is provided in the top of isolated footing pads or in other localities of continuous or combined footings or rafts, where under gravity load compression stresses would prevail. Such reinforcement shall not be less than 0.001 times the gross sectional area of such a pad.

3.5.13 *Structures incorporating mechanical energy dissipating devices.* The design of structures incorporating flexible mountings and mechanical energy dissipating devices is acceptable provided that the following criteria are satisfied:

(a) The performance of the devices used is substantiated by tests

(b) Proper studies are made towards the selection of suitable design earthquakes for the structure

(c) The degree of protection against yielding of the structural members is at least as great as that implied in this Code relating to the conventional seismic design approach without energy dissipating devices

(d) The structure is detailed to deform in a controlled manner in the event of an earthquake greater than the design earthquake.

3.5.14 *Secondary structural elements*

3.5.14.1 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. These are classified as follows:

(a) Elements of Group 1 are those which are subjected to inertia loading but which, by virtue of their detailed separations, are not subjected to loading induced by the deformation of the supporting primary elements or secondary elements of Group 2

(b) Elements of Group 2 are those which are not detailed for separation, and are therefore subjected to both inertia loadings, as for Group 1, and to loadings induced by the deformation of the primary elements.

3.5.14.2 Group 1 elements shall be detailed for separation to accommodate deformations $\nu\Delta$ and Δ_p. Such separation shall allow adequate tolerances in the construction of the element and adjacent elements, and, where appropriate, allow for deformation due to other loading conditions such as gravity loading. For elements of Group 1:

(a) Loading E_p used in the design shall be that specified in NZS 4203

(b) Analysis may be by any rational method

(c) Detailing shall be such as to allow ductile behaviour and in accordance with the assumptions made in the analysis. Fixings for precast units shall be designed and detailed in accordance with 3.5.15.

3.5.14.3 Group 2 elements shall be detailed to allow ductile behaviour and in accordance with the assumptions made in the analysis. For elements of Group 2:

(a) Additional seismic requirements of this Code need not be satisfied when the design loadings are derived from the imposed deformations $\nu\Delta$, specified in NZS 4203, and the assumptions of elastic behaviour

(b) Additional seismic requirements of this Code shall be met when plastic behaviour is assumed at levels of deformation below $\nu\Delta$

(c) Inertia loading E_p shall be that specified by NZS 4203

(d) Loadings induced by the deformation of the primary elements shall be those arising from the level of deformation $\nu\Delta$, specified in NZS 4203 having due regard to the pattern and likely simultaneity of deformation

(e) Analysis may be by any rational method, in accordance with the principles of elastic or plastic theory, or both. Elastic theory shall be used to at least the level of deformation corresponding to and compatible with one-quarter of the amplified deformation, $\nu\Delta$, of the primary elements, as specified in NZS 4203

B2.2 All provisions of this Code for non-prestressed concrete, except 3.3.3.4, shall apply to members designed by the alternative design method.

B2.3 Flexural members shall meet the requirements for deflection control in 4.4 and the requirements of 6.3.2, 6.3.3, 6.4.3 and 6.4.5 of this Code.

B3 GENERAL

B3.1 Design loadings shall be according to NZS 4203 or other appropriate loadings code and capacity reduction factors, ϕ, shall be taken as unity for members designed by the alternative design method.

B4 ALLOWABLE SERVICE LOAD STRESSES

B4.1 Stresses in concrete.

Stresses in concrete shall not exceed the following:

Flexure

Extreme fibre stress in compression . . . . $0.45 f'_c$

*Shear**

Beams and one-way slabs and footings

Shear carried by concrete, v_c . . . . $0.091 \sqrt{f'_c}$

Maximum shear carried by concrete plus shear reinforcement . . . . $v_c + 0.37 \sqrt{f'_c}$

Joists†:

Shear carried by concrete, v_c . . . . $0.10 \sqrt{f'_c}$

Two-way slabs and footings:

Peripheral shear carried by concrete, v_c ‡ . . . . $0.083 (1 + \frac{2}{\beta_c}) \sqrt{f'_c}$

but not greater than . . . . $0.17 \sqrt{f'_c}$

Bearing on loaded area § . . . . $0.30 f'_c$

B4.2 Stresses in reinforcement

Tensile stress in reinforcement f_s shall not exceed ~~the following:~~ 0.55 fy or 200 MPa.

~~Grade 275 reinforcement . . . . 150 MPa~~

~~Grade 380 reinforcement or greater and welded wire fabric, smooth or deformed 200 MPa~~

---

* For more detailed analysis of shear stress carried by concrete, v_c, and shear values for lightweight aggregate concrete, see B8.2

† Designed in accordance with 3.4.2 of this Code.

‡ If shear reinforcement is provided see B8.2.

§ When the supporting surface is wider on all sides than the loaded area, the allowable bearing stress on the loaded area may be increased by A_2/A_1, but not more than 2. When the supporting surface is sloped or stepped, A_2 may be taken as the area ofthe lower base of the largest frustum or a right pyramid or cone contained wholly within the support and having for its upper base the loaded area, and having side slopes of 1 vertical to 2 horizontal.