CTV ELASTIC RESPONSE SPECTRA ANALYSES

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1.0 Introduction

1 I have been engaged by the Royal Commission to undertake a review of the Elastic Response Spectra Analyses (ERSA) modelling and facilitate discussions between the panel of experts.

2 This report summarises the review of the Elastic Response Spectra Analyses used for the CTV building undertaken for the Royal Commission.

3 The reporting covers the discussions between the panel of experts (see Appendix A) regarding the modelling and analysis undertaken by Compusoft Engineering (Compusoft) for StructureSmith Ltd. and the Department of Building and Housing. This is referred to herein as the 'DBH ERSA' analysis.

The panel of experts comprises:

Dr Clarke Hyland (Hyland Fatigue + Earthquake Engineering)

Mr Ashley Smith (Structure Smith Ltd, also representing Compusoft)

Mr Doug Latham (Alan Reay Consulting Ltd.)

Professor Emeritus Athol Carr (University of Canterbury)

Professor John Mander (Texas A & M University)

Professor Robin Shepherd

Dr Brendon Bradley (University of Canterbury)

Dr Graeme McVerry (GNS Science).

2.0 The ERSA Modelling and Analysis

4 Clarke Hyland and Ashley Smith[1] prepared a report (Hyland Report) detailing the analyses carried out on the CTV building as part of the Department of Building and Housing (DBH) investigation of the CTV building collapse.

[1] Hyland/Smith, CTV Building Collapse, Report prepared for the Department of Building and Housing, 25th January 2012,

5 Ashley Smith provided further details regarding the inputs for the DBH code based ERSA in paragraphs 21 to 25 of his first statement of evidence and in Attachment “C” to that evidence (the Modelling Assumptions).

3.0 Expert Panel Review

6 The expert panel was engaged to review the input data used for the ERSA to allow t he Royal Commission to verify code compliance issues.

7 Email messages were sent to all members of the expert panel requesting that concerns with the structural data used for the analyses be submitted and if required what form of conference would be appropriate. The expert panel members indicated that a teleconference would suffice. Drs Bradley and McVerry indicated that the discussion on ERSA was outside their areas of interest.

8 Mr Latham raised the only data issues. These issues are as follows:

3.1. The necessity of the use of ERSA in terms of the NZS 4203 (1984)

9 It was argued that under the 1984 seismic design loadings standard NZS 3203:1984 the terms "regularity" and "eccentricity" are two different things (interpretation of Clause 3.4.7.1).

10 There are three scenarios for considering torsional moments for buildings above four storeys as outlined in this clause:

(a) Reasonably regular structures which are of moderate eccentricity

(b) Reasonably regular structures with a high degree of eccentricity

(c) Irregular structures

11 Latham stated that it is clear from these different scenarios a "regular" building can either be of "moderate eccentricity" or have a "high degree of eccentricity" and therefore shows that there is a difference between regularity and eccentricity.

12 Definitions for "regular" and "eccentric" are suggested in the commentary Clause C3.4.7.1.

13 Clause 3.4.7.2 (b) for reasonably regular structures of more than 4 storeys with a high degree of eccentricity allows the use of a simplified static method of analysis or a two dimensional modal analysis but recommends a three dimensional model analysis. In this case the use of an ERSA analysis to verify code compliance is not required.

3.2. The mass representation used in the analyses

14 Regarding the building mass, Latham noted that the floor weight and SDL (super-imposed dead load) that appears to have assumed by Hyland and Smith is approximately 5% greater than what was allowed for in the original design calculations. Latham requested justification for the increased allowance.

3.3. The foundation stiffness used for the analyses

15 The soil stiffness recommendations provided by Geotech Consulting Ltd (dated 5 June 2012) were more flexible than those provided by Tonkin & Talyor Ltd (T&T). Latham suggested that this would result in the building having a longer period and hence lower design loads. Latham noted that NZS4203:1984 required drifts to be calculated neglecting foundation rotations, therefore the net effect would be lower design drifts. Latham suggested that the analyses should be re-run using Geotech Consulting Ltd's recommendations to quantify the effect. Latham noted that T&T provided a range of soil stiffnesses, and it appears the upper bound (termed "1.36k") was used for the ERSA analysis. Latham questioned the reason why the "most probable" and "lower bound" recommendations made by T&T were not also considered.

3.4 Scaling of results

16 Latham asked for clarification on an aspect of Ashley Smith's brief to Compusoft regarding the modelling assumptions. Latham asked if the base shears presented from the modal analysis were those prior to scaling. Latham noted that at Model 1b E-W, the analysis shows a period of 0.7 seconds and a base shear of 2660 kN. Model 1c E-W shows a period of 0.6 seconds and a base shear of 2996 kN. The design spectrum of NZS4203:1984 is constant over these two periods, so the scaled base shears should be identical.

4.0 The use of ERSA Analyses

17 The panel was required to comment on the data used for the ERSA code compliance analyses for the CTV building. Therefore the arguments about the interpretation of clause 3.7.4. of NZS 4203:1984 are not relevant.

18 It must be noted that the designers of the CTV building used a three dimensional ERSA.

5.0 The Mass used in the DBH ERSA

19 The original design used a dead load of 4.0 kPa for the floor with a SDL (super imposed dead load) of 0.5 kPa.

20 Latham commented that the original design calculations assumed 4.0 kPa for the floor dead weight. The Dimond Hibond literature applicable at the time (see Appendix B) justified this assumption. Latham also noted that in the derivation of the seismic mass used for the DBH models, it appears that a 172.5mm thick slab at 24 kN/m^3 density was assumed giving 4.14 kPa.

21 The original design calculations assumed a 0.5 kPa SDL. A building permit was issued on this basis. In the derivation of the seismic mass used for the DBH models, it appears 0.55 kPa was assumed.

22 Hyland responded with the following points. The use of the base slab weight without allowance for ponding and construction tolerances would results in a low dead load value in his view. For the CTV building the slab was propped to a camber so ponding would have been limited to that occurring between the propping at 1775 mm centres (see drawing S15).

23 Measurements of the actual slab thicknesses at Levels 3, 5 and 6 at the North Core shown in the Hyland Report Figure 166 were 190, 195 and 220 mm respectively.

24 An allowance for a superimposed dead load of 0.5 kPa is quite common but difficult to justify when allowance is made for suspended ceiling, air conditioning, levelling screeds typically required with this sort of floor, floor coverings and partitioning. Hyland typically finds 0.75 to 0.90 kPa to be more realistic for commercial offices.

25 Smith provided the tables on the following pages to compare the mass calculations in the code-based ERSA by StructureSmith/Compusoft with those used by Alan Reay from page S10 of the original calculations. Overall there is very good agreement, within 1% for the total mass. The main differences are at the upper levels where Smith included the main roof mass at Level 6, and the top of the North core as separate eccentric masses at levels 7 & 8. By comparison, Alan Reay had slightly more mass at the top but included it all at roof level

26 Smith included comparisons of the plan coordinates where the masses were applied in the DBH ERSA model versus the plan coordinates shown on page S7 of the original calculations by Alan Reay. For levels 2 to 5 there is good agreement for the centre of mass locations. The greatest differences being in the East-West (Y) direction where the original calculations show the centre of mass between 1% and 3% of the building width closer to the West side. There are some discrepancies between the centre of mass locations at roof level, the reasons for which are explained in the paragraph above.

28 Smith stated that from the earlier mass comparison spreadsheet that he had sent, the DBH seismic weight Wt is 3391.8 x 9.81 = 33274 kN. When Smith divided the ARCL "Implied Seismic Weight" by the DBH seismic weight, he calculated the weight ratios shown in Table 1. Smith stated that he would call that reasonably good agreement.

Table 1. Comparison between Alan Reay and DBH Weights

Alan Reay Implied Weight (kN)	DBH Weight (kN)	Weight Ratio
33407	33274	1.0040
33238	33274	0.9989
33278	33274	1.0001
34213	33274	1.0282
33258	33274	0.9995
33289	33274	1.0005
33619	33274	1.0104
33289	33274	1.0005

29 The seismic live load is not an issue but for interest it must also be noted that the seismic live load is different over the different versions of the NZ seismic design requirements. Both the 1984 and 1992 versions of the loadings standard NZS4203 the office floor live loading is the same at 2.5 kPa. In NZS 1170.5:2004 the office floor live loading is 3.0 kPa.

NZS 4203:1984	Seismic live load factor= 0.333,
NZS 4203:1992	Seismic live load factor= 0.400,
NZS 1170.5:2004	Seismic live load factor= 0.300

30 It should be noted that the original designer used estimates of the masses for the analyses, as the building had not yet been designed. For the analyses carried out for the Royal Commission the engineers had the drawings of the building as designed, and so could get a more reliable estimate of masses.

CTV Building - Comparison of Masses used in Code Response Spectrum Analyses **A H Smith** **26/06/2012 18:08**

Floor Level	from StructureSmith / Compusoft ETABS Model					from Alan Reay original calcs page S10						Floor Level
	ETABS Story	**Mass [A] (tonnes)**	XCM (m)	YCM (m)	**CumMass [C] (tonnes)**	ARCL Level	Weight (kN)	**Mass (B) (tonnes)**	**CumMass [D] (tonnes)**	**Mass Ratio [B/A]**	**CumMass Ratio [D/C]**	
8	STORY7	**71.6**	25.1	12.1	**71.6**)							8
7	STORY6	**64.1**	25.5	13.7	**135.7**)	Roof	1756	**179.0**	**179.0**	**1.32**	**1.32**	7
6	STORY5	**663.6**	12.4	13.9	**799.3**	5	6390	**651.4**	**830.4**	**0.98**	**1.04**	6
5	STORY4	**630.8**	12.8	13.9	**1430.1**	4	6217	**633.7**	**1464.1**	**1.00**	**1.02**	5
4	STORY3	**639.6**	12.8	14.1	**2069.7**	3	6217	**633.7**	**2097.9**	**0.99**	**1.01**	4
3	STORY2	**650.5**	12.8	14.4	**2720.2**	2	6217	**633.7**	**2731.6**	**0.97**	**1.00**	3
2	STORY1	**671.6**	12.9	14.3	**3391.8**	1	6217	**633.7**	**3365.3**	**0.94**	**0.99**	2

CTV Building - Comparison of Mass and Centre of Mass used in Code Response Spectrum Analyses | **A H Smith** | **1/07/2012 14:18**

COMPARE MASS

Floor Level	ETABS Story	from DBH ETABS Model: Mass [A] (tonnes)	XCM (m)	YCM (m)	CumMass [C] (tonnes)	from Alan Reay original calcs page S10: ARCL Level	Weight (kN)	Mass (B) (tonnes)	CumMass [D] (tonnes)	Ratio: Mass Ratio [B/A]	CumMass Ratio [D/C]	Floor Level
8	STORY7	**71.6**	25.1	12.1	**71.6**)							8
7	STORY6	**64.1**	25.5	13.7	**135.7**)	Roof	1756	**179.0**	**179.0**	**1.32**	**1.32**	7
6	STORY5	**663.6**	12.4	13.9	**799.3**	5	6390	**651.4**	**830.4**	**0.98**	**1.04**	6
5	STORY4	**630.8**	12.8	13.9	**1430.1**	4	6217	**633.7**	**1464.1**	**1.00**	**1.02**	5
4	STORY3	**639.6**	12.8	14.1	**2069.7**	3	6217	**633.7**	**2097.9**	**0.99**	**1.01**	4
3	STORY2	**650.5**	12.8	14.4	**2720.2**	2	6217	**633.7**	**2731.6**	**0.97**	**1.00**	3
2	STORY1	**671.6**	12.9	14.3	**3391.8**	1	6217	**633.7**	**3365.3**	**0.94**	**0.99**	2

COMPARE CENTRE OF MASS COORDINATES
CONCENTRIC CASE

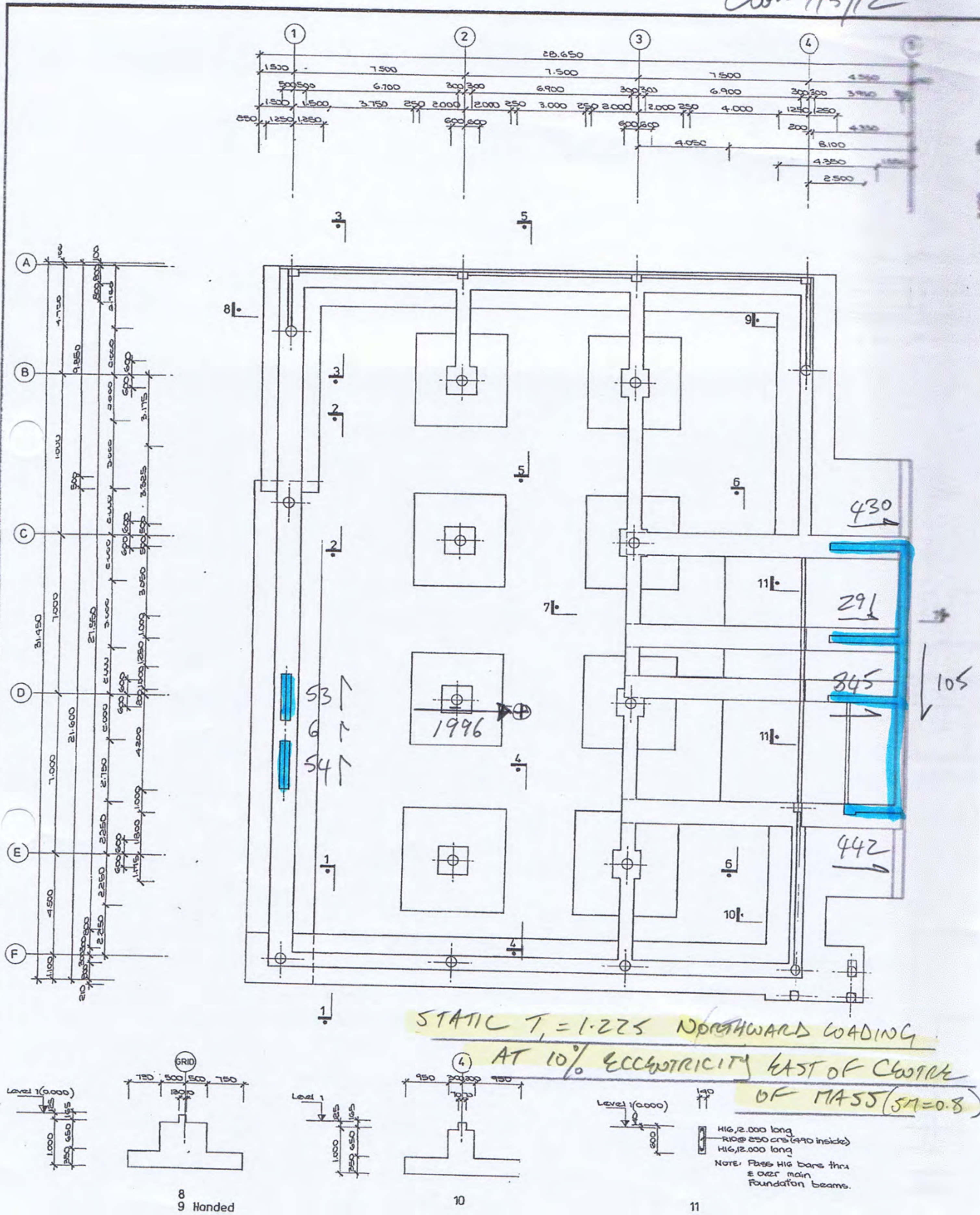
Floor Level	ETABS Story	from DBH ETABS Model: XCM (m)	YCM (m)		from Alan Reay original calcs page S7: ARCL Level	XCM (m)	YCM (m)	Difference: delta XCM (m)	delta YCM (m)	
8	STORY7	**25.1**	**12.1**	)						
7	STORY6	**25.5**	**13.7**	)	Roof	**12.95**	**14.78**			(use weighted average)
6	STORY5	**12.4**	**13.9**		5	**12.95**	**14.78**	**-1.65**	**1.02**	(at Level 6 = ETABS Story 5)
5	STORY4	**12.8**	**13.9**		4	**12.95**	**14.78**	**0.10**	**0.92**	
4	STORY3	**12.8**	**14.1**		3	**12.95**	**14.78**	**0.13**	**0.69**	
3	STORY2	**12.8**	**14.4**		2	**12.95**	**14.78**	**0.15**	**0.42**	
2	STORY1	**12.9**	**14.3**		1	**12.95**	**14.78**	**0.07**	**0.45**	

COMPARE CENTRE OF MASS COORDINATES
XYP (X & Y OFFSET BY +0.1B)

Floor Level	ETABS Story	from DBH ETABS Model: XCM (m)	YCM (m)		from Alan Reay original calcs page S7: ARCL Level	XCM (m)	YCM (m)	Difference: delta XCM (m)	delta YCM (m)	
8	STORY7	**25.5**	**13.2**	)						
7	STORY6	**25.9**	**14.4**	)	Roof	**15.66**	**17.81**			(use weighted average)
6	STORY5	**15.1**	**17.0**		5	**15.66**	**17.81**	**-1.24**	**1.36**	(at Level 6 = ETABS Story 5)
5	STORY4	**15.5**	**16.9**		4	**15.66**	**17.81**	**0.16**	**0.91**	
4	STORY3	**15.5**	**17.1**		3	**15.66**	**17.81**	**0.16**	**0.71**	
3	STORY2	**15.5**	**17.4**		2	**15.66**	**17.81**	**0.16**	**0.41**	
2	STORY1	**15.6**	**17.4**		1	**15.66**	**17.81**	**0.06**	**0.41**	

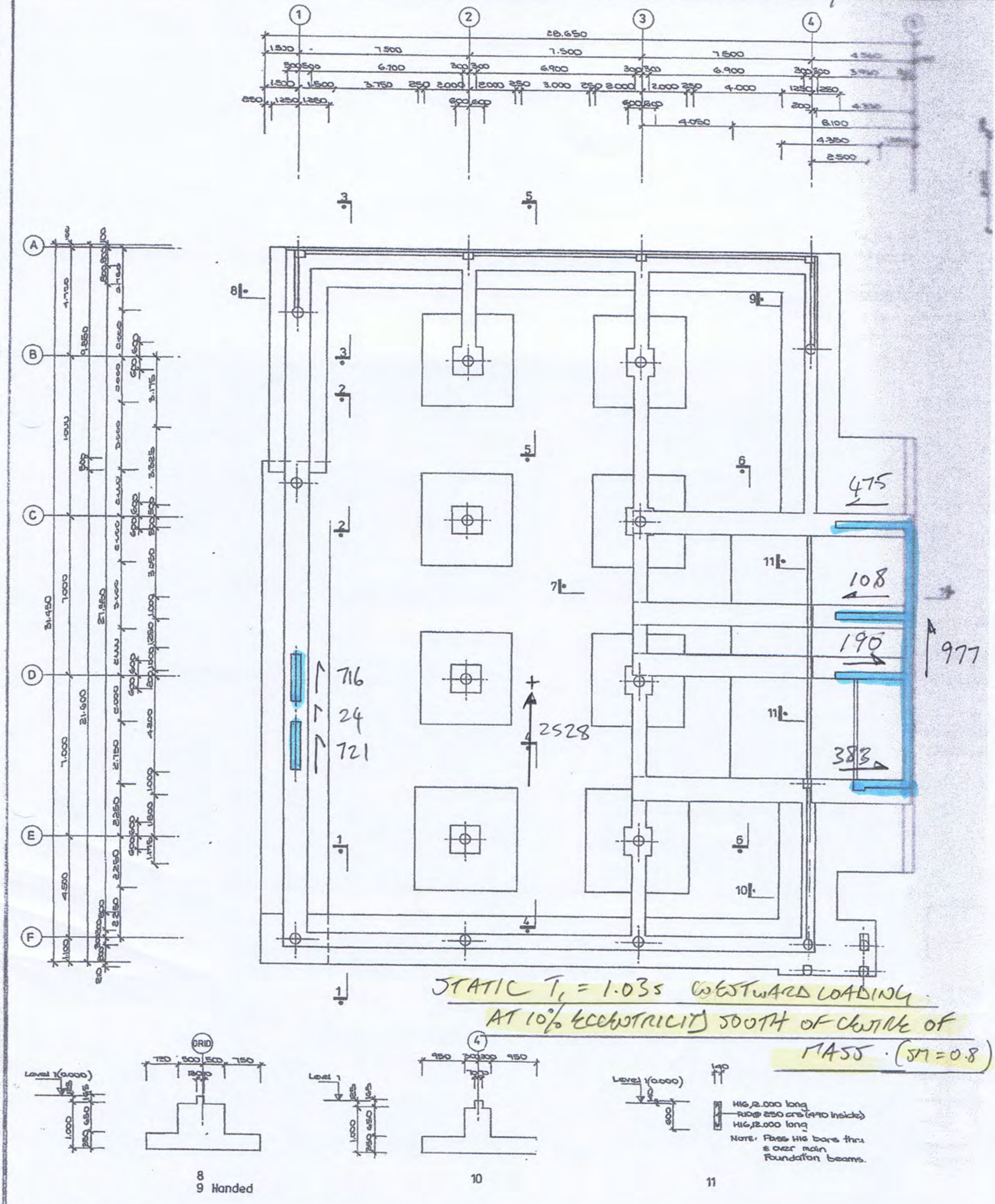
COMPARE CENTRE OF MASS COORDINATES
XYN (X & Y OFFSET BY -0.1B)

Floor Level	ETABS Story	from DBH ETABS Model: XCM (m)	YCM (m)		from Alan Reay original calcs page S7: ARCL Level	XCM (m)	YCM (m)	Difference: delta XCM (m)	delta YCM (m)	
8	STORY7	**24.6**	**10.9**	)						
7	STORY6	**25.0**	**13.0**	)	Roof	**10.24**	**11.75**			(use weighted average)
6	STORY5	**9.7**	**10.9**		5	**10.24**	**11.75**	**-2.02**	**0.68**	(at Level 6 = ETABS Story 5)
5	STORY4	**10.2**	**10.8**		4	**10.24**	**11.75**	**0.04**	**0.95**	
4	STORY3	**10.1**	**11.1**		3	**10.24**	**11.75**	**0.14**	**0.65**	
3	STORY2	**10.1**	**11.3**		2	**10.24**	**11.75**	**0.14**	**0.45**	
2	STORY1	**10.2**	**11.3**		1	**10.24**	**11.75**	**0.04**	**0.45**	

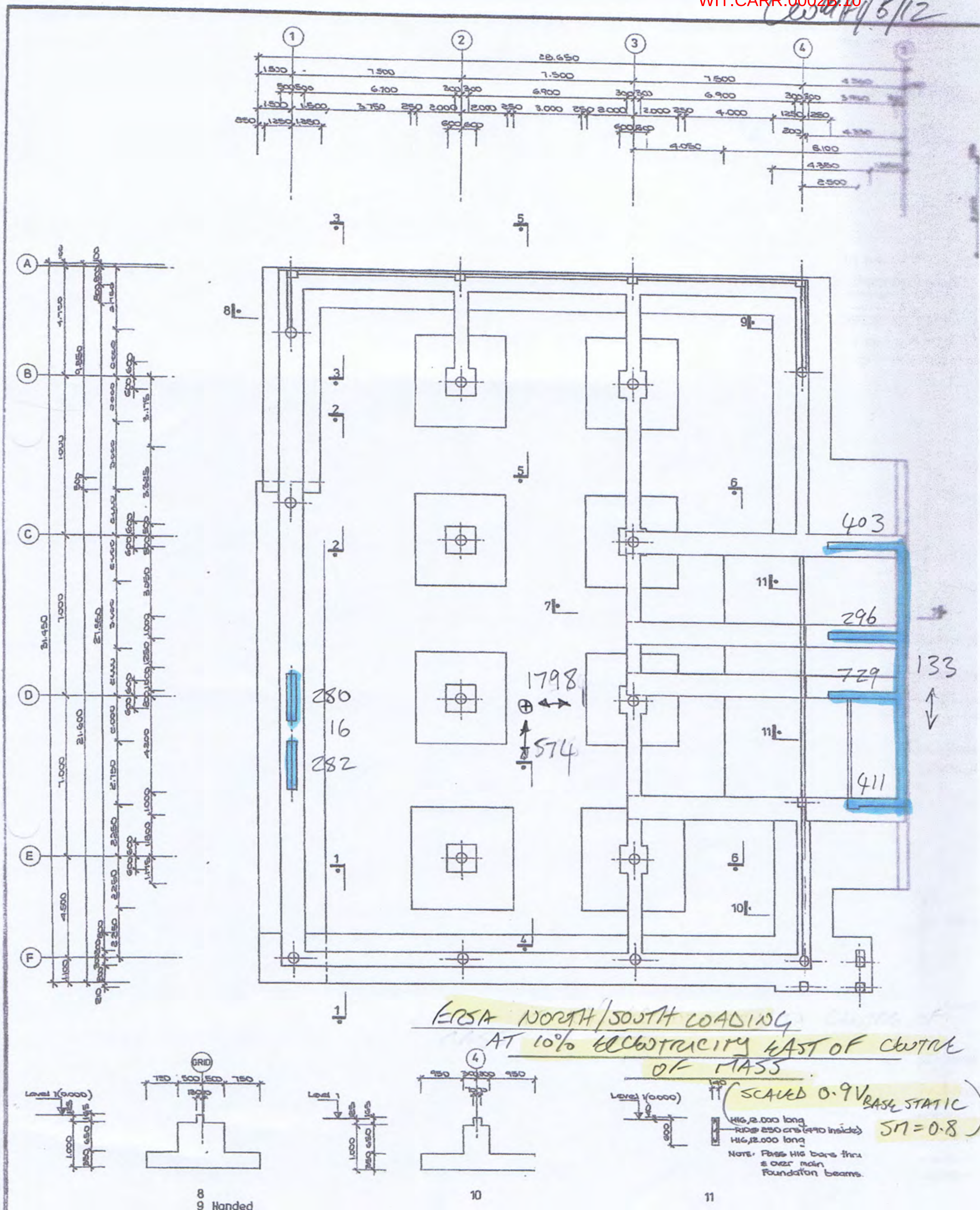


OFFICE BUILDING - 249 MADRAS STREET

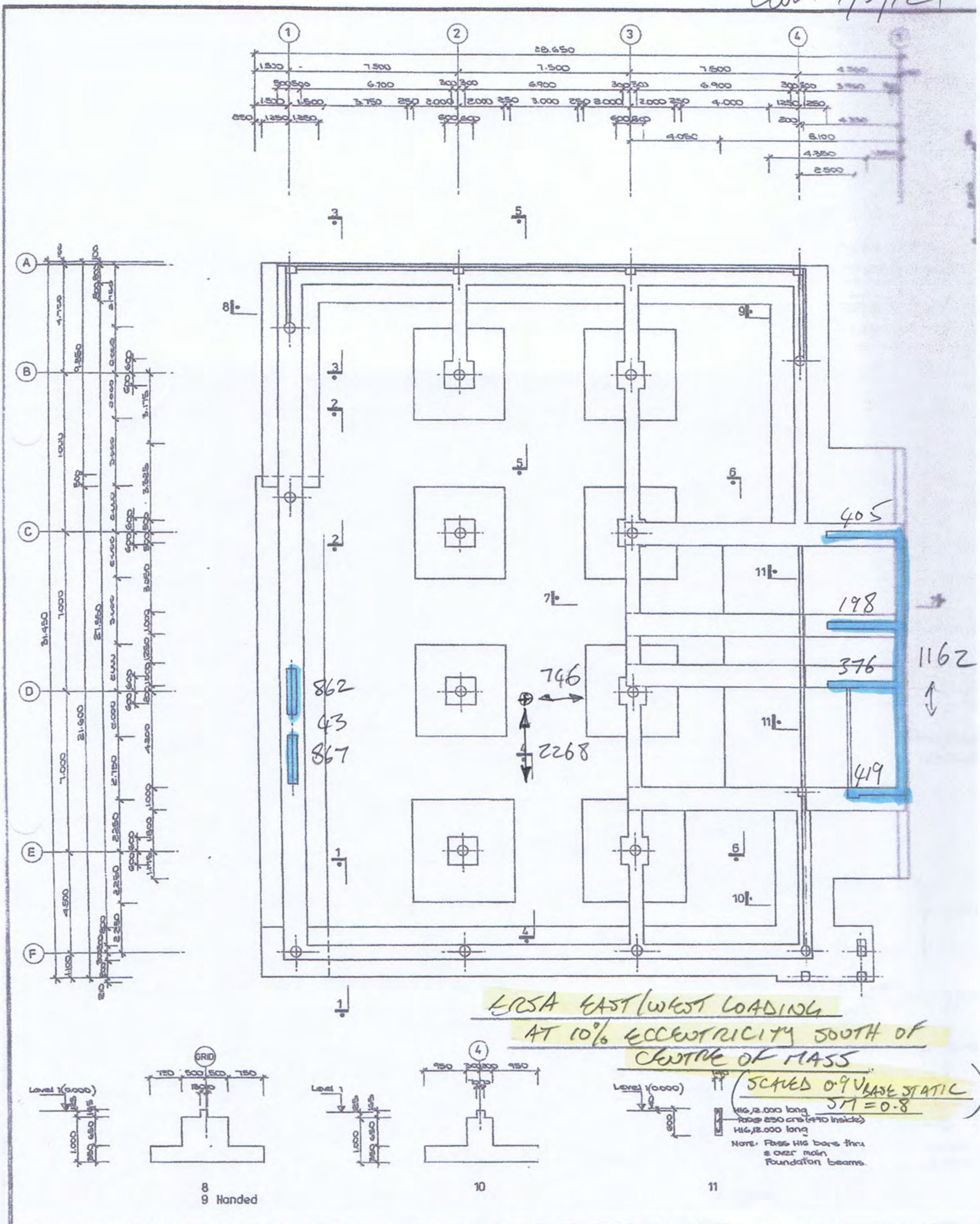
Cook 1/3/12 2



OFFICE BUILDING - 249 MADRAS STREET



OFFICE BUILDING - 249 MADRAS STREET



ERSA EAST/WEST LOADING AT 10% ECCENTRICITY SOUTH OF CENTRE OF MASS

(SCALED 0.9 V BASE STATIC SM = 0.8)

OFFICE BUILDING - 249 MADRAS STREET

CALCULATIONS

ALAN M. REAY
CONSULTING ENGINEER
CHRISTCHURCH

PAGE S3
SECT
FILE
DATE

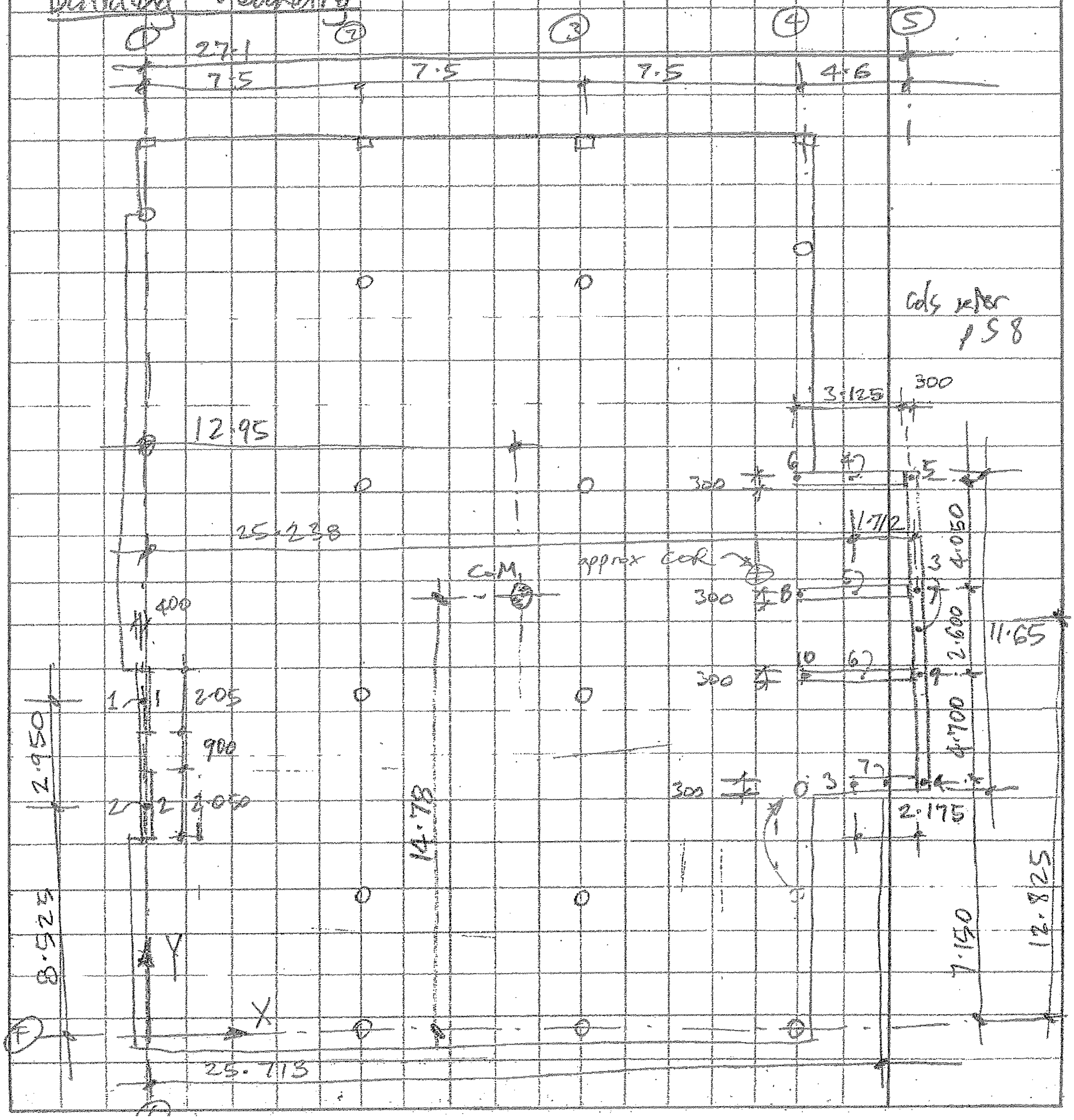
Ht. of equiv. lat. force $= \dfrac{OTM}{V_x}$

$= \dfrac{41576 \text{ kNm}}{3177 \text{ kN}}$

$\bar{h} = 13.09 \text{ m}$

$\dfrac{\bar{h}}{h_e} = \dfrac{13.09}{20.50} = 0.64 \geq$ OK.

Building Geometry



Y co-ordinate.

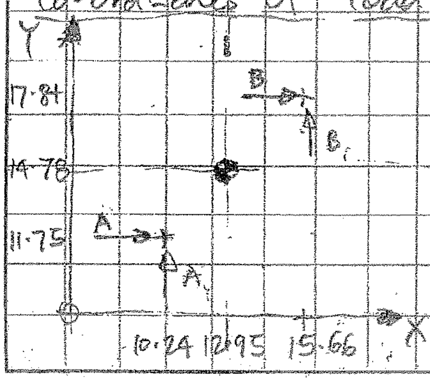
Component	w_i	y_i	$w_i y_i$
floor & Slab	3298	15·125	49,882
	207	12·825	2,654
beams	1146	15·125	17,333
columns	160	15·125	2,420
walls	152	10·000	1,520
	214	14·85	3,178
	50	7·15	357
	266	12·825	3,411
	41	12·825	526
curtain wall	33	30·250	998
	5567		82,279

$$\bar{y} = \frac{82279}{5567} = \underline{14.78}$$

ie X co-ord CoM $= 12.95 \pm (0.1 \times 27.1)$
$= 12.95 \pm 2.71$
$= 10.24,\ 15.66.$

Y co-ord CoM $= 14.78 \pm (0.1 \times 30.3)$
$= 14.78 \pm 3.03$
$= \underline{11.75},\ \underline{17.81}$

Co-ordinates of load cases:



X_{EQ}	Y_{EQ}
$F_{XA} = F_{XB} = H$	$F_{XA} = F_{XB} = 0$
$F_{YA} = F_{YB} = 0$	$F_{YA} = F_{YB} = H$
$X_A = 10.24$	$X_A = 10.24$
$Y_A = 11.75$	$Y_A = 11.75$
$X_B = 15.66$	$X_B = 15.66$
$Y_B = 17.81$	$Y_B = 17.81$

ARCL Check of Base Shear provided by Ashley Smith, Issue date 2/07/2012

	Period from Smith	Coefficient NZS4203	Factors NZS4203	Static Design coefficient	Base Shear from Smith	Implied Seismic Mass
Case	**T**	**C**	**RSM**	**Cd**	**V***	**Wt**
Model 1a NS Rigid	0.82	0.113	0.8	0.090	2718	33407
Model 1a EW Rigid	0.79	0.116	0.8	0.093	2776	33238
Model 1a NS	1.20	0.075	0.8	0.060	1797	33278
Model 1a EW	0.94	0.101	0.8	0.081	2488	34213
Model 1b NS	1.03	0.092	0.8	0.074	2203	33258
Model 1b EW	0.70	0.125	0.8	0.100	2996	33289
Model 1c NS	0.88	0.107	0.8	0.086	2590	33619
Model 1c EW	0.60	0.125	0.8	0.100	2996	33289

Notes:

Coefficient taken from Figure 3: Basic Seismic Coefficient, NZS4203:1984, Zone B, Flexible subsoil
Equation can be represented as C=0.195-0.1*T between the ranges of T=0.7-1.2sec

Factors RSM, where R=1.0, S=1.0, M=0.8

Implied Seismic Mass = Base Shear from Smith / Static Design Coefficient / 0.9
where 0.9 is due to scaling ERSA to 90% of static base shear

CTV Building - ETABS Computer Analyses										Issue Date:	2/07/2012
Modelling Assumptions and First Mode Period, Base Shear Comparisons											
	Units	**Analysis Model 1**								**Analysis Model 2**	
References		**1986 Codes**								**Current Codes**	
		NZS4203:1984								AS/NZS1170.5	
		NZS3101:1982								(pre DBH amndmt)	
										NZS3101:2006	
										NZSEE Guidelines	
Objective		1986 Design Code compliance check								Spectra, Period T1, comparisons with Current Code	
Software program used		ETABS								ETABS	
Analysis Type		Elastic, 3D, Dynamic Spectral Modal Analysis								Elastic, 3D, Dynamic	
Seismic Load Input		Response Spectra								Response Spectra	
Superimposed Dead Load	kPa	0.55								0.55	
Live Load	kPa	2.50								3.00	
Seismic Live Load	kPa	0.83								0.45	
T & L Beams - slab overhang each side	mm	300								300	
Material Properties	Various Units	Specified Material Properties (f'c 25MPa typical - up to 35MPa for level 1 columns)								Specified Properties	
Effective Section Properties, I_e, Av_e											
- I_e, T & L beams	Fraction of I_g	0.50								0.33	
- I_e, Columns	Fraction of I_g	1.00								0.66	
- I_e, Walls	Fraction of I_g	0.60								0.33	
- I_e, Diagonally reinforced coupling beams, Grid 1	Fraction of I_g	0.40								0.60	
- Av_e, Diagonally reinforced coupling beams, Grid 1	Fraction of A_g	0.83								varies 0.048 to 0.092	
Code Subsoil Flexibility / Site Subsoil Class (for seismic load input)		Flexible subsoil								Class D (deep or soft soil)	
Modelled foundation spring stiffness - where k = expected stiffness, 0.77k = lower bound stiffness and 1.36k = upper bound stiffness (refer Tonkin & Taylor report)		Rigid Foundation		1.36k						1.36k	
Accidental Eccentricity		Concentric		**Concentric**		+0.1B		-0.1B		Concentric	
	EQ Direction	N-S (X)	E-W (Y)	**N-S (X)**	**E-W (Y)**	N-S (X)	E-W (Y)	N-S (X)	E-W (Y)	N-S (X)	E-W (Y)
Model 1a. - Concrete Walls only (As-Drawn)											
First mode period of vibration, T1	seconds	**0.82**	**0.79**	**1.20**	**0.94**	1.22	0.81	1.21	1.02	1.39	1.19
Base Shear - (ductile S=1, M=0.8)	kN	**2718**	**2776**	**1797**	**2488**	1796	2728	1796	2220		
				*	*						
Model 1b. - Concrete Walls + Masonry Walls (As-Built)											
First mode period of vibration, T1	seconds	N/A	N/A	**1.03**	**0.70**	N/A	N/A	N/A	N/A	N/A	N/A
Base Shear - (ductile S=1, M=0.8)	kN	N/A	N/A	**~~2342~~**	**~~2660~~**	N/A	N/A	N/A	N/A	N/A	N/A
				2203	2996	**base shears corrected 02-07-2012**					
Model 1c. - Concrete Walls + Masonry Walls + Frame (As-Built)				*	*						
First mode period of vibration, T1	seconds	N/A	N/A	**0.88**	**0.60**	N/A	N/A	N/A	N/A	N/A	N/A
Base Shear - (ductile S=1, M=0.8)	kN	N/A	N/A	**2590**	**2996**	N/A	N/A	N/A	N/A	N/A	N/A
				*	*						
LEGEND		= Analyses to compare first mode period and / or base shear only								Note: Current Code has Ie for beams based on f'c 40MPa	
	*	= Analyses referred to in this report									

6.0 Foundation Stiffness

31 Smith stated that the input data for the ERSA analyses that were carried out, as outlined in his spreadsheet with the various ETABS modelling assumptions (refer to attachment C to his first statement of evidence), came from the standards and also from the NZSEE Journal article of June 1980 by Paulay and Williams[2]. In particular Smith refers to page 112 of Paulay & Williams (see Appendix C) for the effective stiffness of shear walls and modelling of foundation flexibility, neither of which was explicitly covered in the NZS4203:1984 standards.

32 Latham is correct when he states more flexible foundations would increase the natural period, thereby reducing the design forces. This can be seen by comparing the period and base shear values in the spreadsheet for the two cases at the left hand side (both concentric mass models, one with rigid and one with 1.36k soil spring stiffness). Smith stated that one could also check the variation of displacements between those two runs to assess the sensitivity of displacements to variations in foundation stiffness.

33 T&T provided a letter dated 26 June 2012 that stated, after consideration of the points raised by Geotech Consulting, they stood by their original recommendations for dynamic soil stiffness.

34 Regarding Latham's question why the upper bound (termed "1.36k") soil stiffness was used for the DBH ERSA analysis, Smith responded that in his opinion it would be appropriate to use an upper bound soil stiffness for strength design purposes because that would give higher forces, however one could also check the variation of displacements between the two runs with varying foundation stiffness.

35 Latham wishes to see the analyses repeated using the lower foundation stiffness as proposed by Geotech Consulting.

[2] Paulay, T. and Williams, R.L. The Analysis and Design of and the Evaluation of Design Actions for Reinforced Concrete Ductile Shear Wall Structures. Bull. NZSEE Vol. 13, No. 2, June 1980. pp108-143

4402
5 June 2012

GEOTECH consulting ltd

Alan Reay Consultants Ltd
PO Box 3911
Christchurch

Attention: Alan Reay

Dear Sir,

RE: CTV Building – 249 Madras St

You have asked for comment on some aspects of the foundations and subsurface effects at this site. I was the engineer responsible for managing and writing the 1986 geotechnical report from Soils and Foundations, with internal peer review by Mr Don Preston. I do not have the original files to refer to, but you have forwarded a copy of the 1986 report..

1 Modulus of Subgrade Reaction

(a) Values recommended in 1986

You have asked what values I would have used at the time of the CTV building design in 1986.

The modulus of subgrade reaction is defined as ks = q/y where q is applied stress and y = deflection. At the time of the design, I would have been considering the normal gravity loading case, and the simplest way to estimate it is therefore straight off the settlement – bearing pressure curves. Using Figure 299/3 from the 1986 report, I get the ks values for a square pad as shown in columns 6 &7 of Table 1. As a check I have also rerun a settlement estimate independently of the original calculations and model, and obtained the values in the previous four columns, 2 - 5. There is good agreement (in geotechnical terms!!)

1	2	3	4	5	6	7	8
Footing Width (m)	From 2012 estimate				From 1986 report		From plate bearing test Lichfield St 1984
	Gravel Square	Strip	NE Square	Strip	Gravel Square	NE Square	
0.5	20.8	10.8	19.6	8.8			
1	10.6	6.5	9.4	5.0	10.6	10.4	3.4
1.5	7.4	5.0	6.2	3.7			
2	5.9	4.2	4.7	3.0	6.4	4	2.6
2.5	5.0	3.8	3.9	2.6			
3	4.5	3.4	3.3	2.4	5.2	3	2.4
3.5	4.2	3.2	3.0	2.2			
4	3.9	3.0	2.7	2.1	4.6	2.6	2.3
4.5	3.7	2.9	2.5	2.0			
5	3.6	2.8	2.3	1.9			

Table 1 Modulus of Subgrade reaction (MPa/m)

Dr. Mark Yetton E-mail myetton@geotech.co.nz
Nick Traylen E-mail ntraylen@geotech.co.nz
Ian McCahon E-mail mccahon@geotech.co.nz

Tel (03) 9822 538
Fax (03) 3257 555
PO Box 130 122
4 / 6 Raycroft Street
Christchurch 8141 New Zealand

Column 8 of Table 1 shows the ks values derived from a plate bearing test on a similar soil profile in Lichfield St. The plate was 0.3m square, so the applied load increases stresses only in the upper soil layer, and will not reflect what happens with larger footings with deeper stress bulbs, but does provide a measure of reality check.

Another method using the SPT N values to estimate the relative density, and from that the modulus of subgrade reaction gives ks = 4.5 – 12 MPa/m for a 2m wide footing, and 4 – 10 MPa/m for a 4m square footing.

Typical values from text books usually give higher values, but these are likely to be for 0.3m square loaded areas on homogeneous soils. When corrected for width and submergence, values are likely to come down to values somewhere in the above range.

Ks is notoriously hard to determine, and the above values are likely to be +/- 50%. I conclude that the values derived from the 1986 report would have been the values I would have recommended at the time.

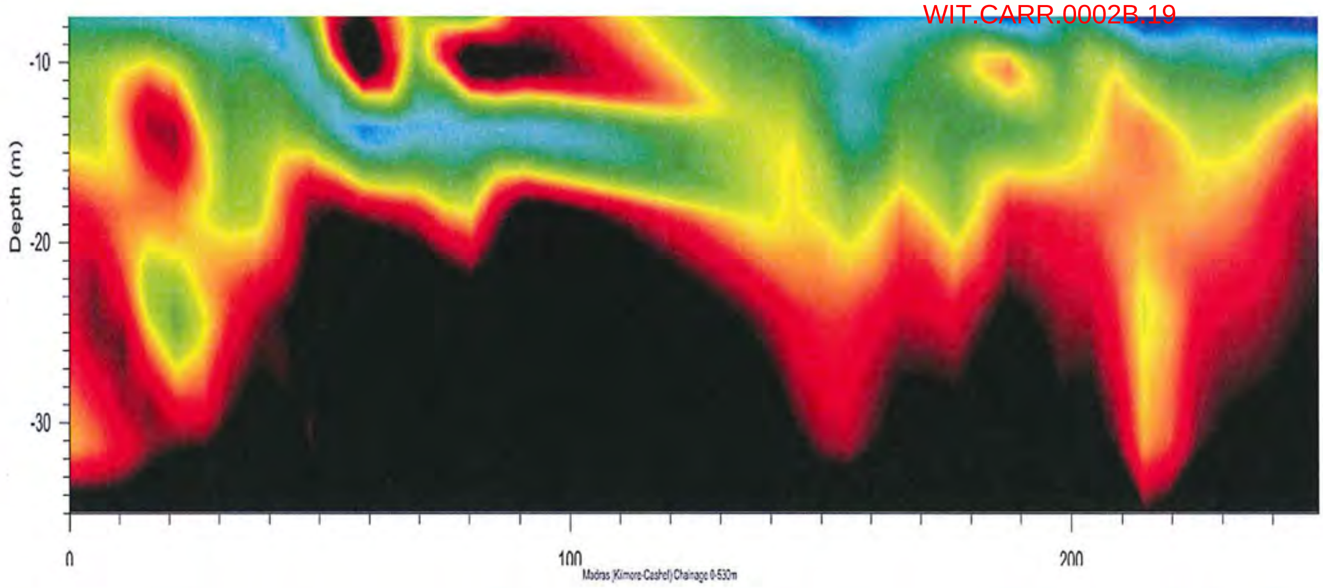
(b) Values used today

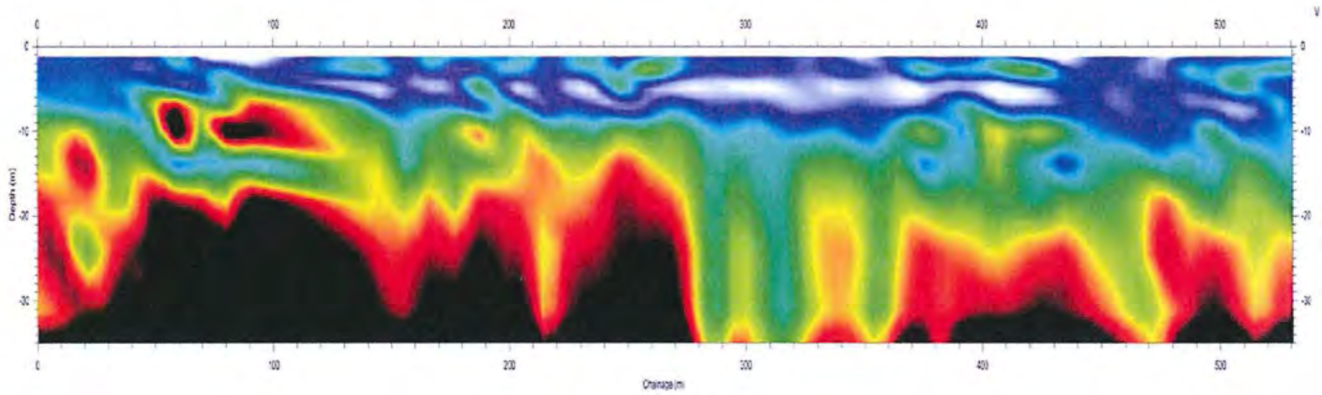
You have asked what values for ks I would suggest for use if the building design was being carried out today. I would still suggest the use of ks values similar to those derived from the 1986 work.

Tonkin and Taylor (T+T) have reported on the site in their letter titled *CTV Building Geotechnical Advice*, dated 11 July 2011, to StructureSmith Ltd. They include a section on subgrade reaction for the dynamic analysis. I am not an expert in this field and do not wish to comment, other than making the comment that with the relatively loose cohesionless soils in Christchurch, seismic shaking appears to have generated high pore water pressures in soils even if there has not been full liquefaction. This must reduce the shear strength of the soil, and the reasoning that subgrade reaction values for dynamic analysis should be expected to be much greater than for static analysis may not be entirely applicable. Dr Kevin McManus has presented an argument to the Royal commission that the use of a strength reduction factor of 0.8 – 0.9 with earthquake overstrength (Table 1 B1/VM4) is unconservative and that a lower value should be used, which is in line with my comments above..

T+T also comment on static subgrade reaction stiffness. They give a range of values of 51 to 116 MN/m^3 for footing type 1 and 10 to 80 MN/m^3 for type 1b (on the area without underlying gravel). These values appear to be derived from published data (Bowles 1988 is referenced). It is unclear whether footing width has been taken into account in their derivation. My understanding, and use of the published data, is that the values are for a standard one foot (0.3m) square plate as traditionally used in plate bearing tests (this was explicit in Bowles 1st edition, 1968, but was not in later editions). Assuming this, then a published value of 80 MN/m^3 becomes about 23 MN/m^3 when corrected for a 4m wide footing. It is also noted that T+T have taken the depth of influence as 3 x footing width. The stress levels are low at this depth and 2x or even 1.5x the footing width are often taken as an effective depth of influence. I conclude that the values suggested by Tonkin and Taylor are high.

The shear wave velocities and derived shear modulus for the site are low in the upper few metres of the site





Based on these shear wave velocities we get the following values for shear modulus (Table2). There is reasonable agreement between the SPT derived values and the MASW derived values in the upper 5 metres of the subsoil on the site (where the SPT tests were made).

Depth	Vs m/s)	P (x10^3 kg/m^3)	G (MPa)
0 – 2	160	1.8	46
2 – 4	180	2	58
4 – 6	300	2	180
6 – 8	400	1.8	320
8 – 10	450	1.8	360
10 – 12	420	1.8	320
12 – 14	300	1.8	160
14 – 16	300	1.8	160
16 – 18	420	1.8	320
18 - 20	500	2	500

Table 2 ***Shear Wave velocities and shear modulus (Madras St)***

Yours faithfully

Geotech Consulting Limited

Ian McCahon

Tonkin & Taylor

T&T Ref: 52118
26 June 2012

Structure Smith Ltd
PO Box 26-502
Epsom
Auckland 1344

Attention: Ashley Smith

Dear Sir

CTV Building Geotechnical Advice: Comment on Report by Geotech Consulting

I have read the report by Ian McCahon dated 5 June 2012 in which he comments on some of the items/issues in my Geotechnical Advice dated 11 July 2011 [WIT.SINCLAIR.0001].

There are some minor points of difference between us, or possibly misunderstanding of my views, but I believe these are not important in the context of this investigation.

I note however, the comment relating to subgrade reaction under dynamic conditions and that increased pore pressures would lead to reduced shear strength, even if there has not been full liquefaction. This may be so, but the increased pore pressure does not reduce the subgrade reaction. Under the short-term, rapidly reversing loading during an earthquake, the soil is essentially "undrained" and the vertical foundation loads are carried partly by the soil skeleton but mostly by the water in the pores. As the water is almost incompressible, the dynamic stiffness is very high.

I stand by my original estimates of subgrade reaction for the reasons stated in my original report.

Yours faithfully

T J E Sinclair

T J E Sinclair
Principal

28-Jun-12
TJES:rmt
p:\52118\communications\tjes260612.let.docx

7.0 Scaling of Results

35 The structure has different natural periods of free-vibration in the East-West and North-South directions. Using NZS 4203:1984 the seismic coefficient may be the identical at those different natural periods. However, the Base Shears for both directions of excitation will be identical *if, and only if*, an equivalent static analysis is being used. This is because the static analyses uses only the fundamental period of free-vibration.

36 For an ERSA, the higher modes in each direction will also have different natural periods of the free vibration in each direction. These higher mode periods will be such that the spectral coefficient will no longer be on the long period plateau of the design spectra. These different higher modes will therefore have different contributions to the Base Shear.

37 Smith confirmed that the base shears in his “Modelling Assumptions Spreadsheet” had been scaled to NZS4203:1984.

38 Hyland commented that NZS 4203:1984 wasn't clear on how the scaling was to be done in terms of 3D analysis if the torsional mode was 1st or 2nd mode. I suspect that was due to most analyses in practice being 2D at the time with approximations to 3D, so perhaps the code writers assumed that 1st mode would always be translational. This It is clearer in the current loadings standard.

39 Hyland stated that his practice was to find the worst case scaling factor and apply that in each direction.

Appendix A. Panel of Experts to Consider ERSA for CTV Building

Canterbury Earthquakes Royal Commission
Te Komihana Rūwhenua o Waitaha

UNDER	**THE COMMISSIONS OF INQUIRY ACT 1908**
IN THE MATTER OF	**THE CANTERBURY EARTHQUAKES ROYAL COMMISSION**
AND IN THE MATTER OF	**THE CTV BUILDING COLLAPSE**

ORDER AS TO DIRECTIONS IN RELATION TO ELASTIC RESPONSE SPECTRA ANALYSIS EVIDENCE

Dated: 18 June 2012

Unit 15, Barry Hogan Place, Addington, Christchurch
P O Box 14053 Christchurch Mail Centre 8544
Freephone (NZ only) 0800 337 468
www.royalcommission.govt.nz

ORDER AS TO DIRECTIONS IN RELATION TO ELASTIC RESPONSE SPECTRA ANALYSIS EVIDENCE

1. The Royal Commission directs that the expert witnesses whose evidence will relate to Elastic Response Spectra Analysis (ERSA) of the response of the CTV Building are to confer.

2. These witnesses are:

   2.1. Dr Clark Hyland

   2.2. Ashley Smith.

   2.3. Professor Athol Carr.

   2.4. Professor John Mander.

   2.5. Professor Robin Shepherd.

   2.6. Douglas Latham.

   2.7. Brendon Bradley.

   2.8. Graeme McVerry.

3. The Royal Commission appoints Professor Athol Carr as a facilitator with authority to take the steps necessary to achieve the purposes of this order.

4. The purposes of the experts conferring are:

   4.1. To endeavour to reach agreement on the input data to be used to conduct an ERSA of the response of the CTV Building to determine whether the design of the building was consistent with the provisions of NZS 3101:1982 and NZS 4203:1984.

   4.2. Where agreement cannot be reached on the inputs, to identify:

      4.2.1. The inputs which cannot be agreed.

      4.2.2. The reasons for the disagreement.

   4.3. To produce ERSA results which provide the most reliable model for the purposes set out in clause 4.1, and which can then be analysed and interpreted. In this respect:

4.3.1. Compusoft has already conducted an ERSA ('the Compusoft ERSA').

4.3.2. The experts are to consider whether the Compusoft ERSA provides the most reliable model for the purposes set out in clause 4.1.

4.3.3. If the experts cannot agree about whether the Compusoft ERSA provides the most reliable model, the experts are to identify the reasons for their disagreement.

4.4. If the experts do not reach agreement that the Compusoft ERSA provides the most reliable model for the purposes set out in clause 4.1, a further ERSA is to be carried out. In this case:

4.4.1. The experts are to agree on the inputs to be used. If agreement is not reached, they are to identify their reasons for disagreement.

4.4.2. If agreement is not reached, or in the opinion of the facilitator is not likely to be reached, the facilitator is to report to the Royal Commission on the areas of disagreement and their significance so that the Commission can consider whether any further orders are required.

5. These directions apply to ERSA input data and ERSA results, but not to any evidence relating to subsequent interpretation of ERSA results, which shall be a matter for individual parties to address.

6. The experts are to take all necessary steps to achieve the purposes described above, including:

6.1. All input data used in the Compusoft ERSA and any other ERSA are to be made available to every other expert.

6.2. The data is to be provided in a form suitable for use in an alternative model.

7. The input data used in the Compusoft ERSA:

7.1. Is confidential to the persons listed in paragraph 2 of this order.

7.2. Must not be used by any person for any purpose other than those described in these directions.

7.3. Must be returned to Compusoft following the conclusion of the CTV hearing.

7.4. Must not be copied or retained.

8. All other information shared between experts:

   8.1. Remains confidential to the parties, their legal advisors and the experts except where it is included in the joint report.

   8.2. Must not be used by any person for any purpose other than those described in these directions.

   8.3. Must be returned to the provider following the conclusion of the CTV hearing.

   8.4. Must not be copied or retained.

9. The experts are to produce a joint report for the Royal Commission which identifies the following:

   9.1. All areas of agreement.

   9.2. All areas of disagreement, including the reasons for the disagreement.

   9.3. The results of any further ERSA/s.

10. The experts are to comply with the Code of Conduct for Expert Witnesses set out in schedule 4 to the High Court Rules. In particular the experts are to:

    10.1. Attempt to reach agreement about the matters set out above.

    10.2. Exercise independent and professional judgement and not to act on the instructions or directions of any person to withhold or avoid agreement.

11. The joint report is to be provided to the Royal Commission by 2 July 2012. If this date cannot be met, the Royal Commission is to be advised immediately this becomes apparent.

12. The Royal Commission reserves the right to alter these directions.

13. Any matters of dispute about the processes to be followed must be raised with the Royal Commission forthwith.

14. The experts may be required to participate in a 'hot tub' in the course of the CTV hearing during which they will be called upon to give evidence and answer questions about the matters set out in these directions.

15. These directions are made in the exercise of powers of the Chair of the Royal Commission as a Judge of the High Court of New Zealand under section 13 of the Commissions of Inquiry Act 1908.

Dated: 18 June 2012

The Honourable Justice Cooper
Chair of the Royal Commission
A Judge of the High Court of New Zealand

Appendix B. Dimond Hibond Literature

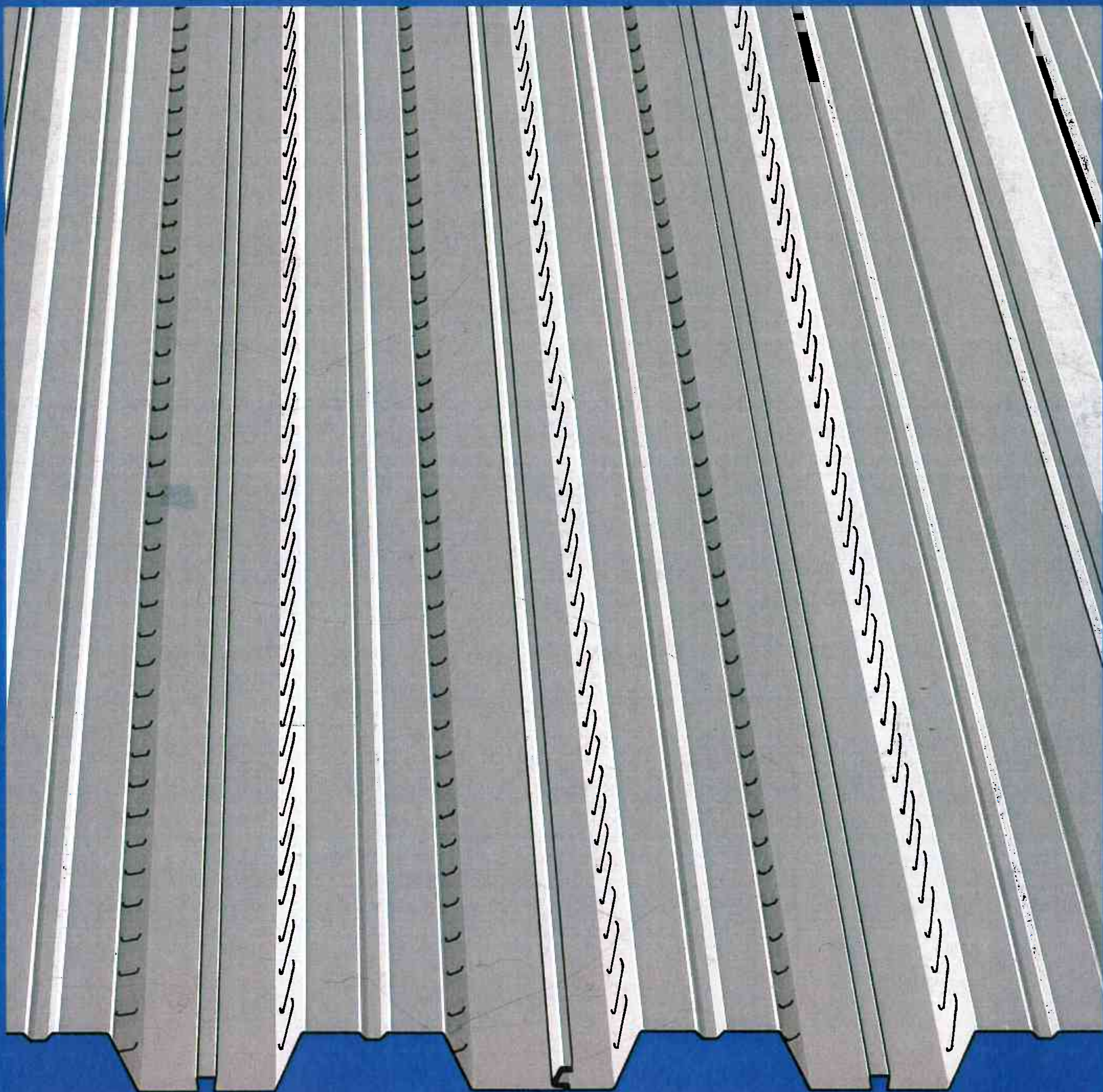
DIMOND

section 04·1
product data

CI/SfB 1976
(23 4)Eq4:Hh2
June 1982

HI-BOND H.S.

composite
steel/concrete
floor system



DIMOND INDUSTRIES LTD

Index

Introduction

The Dimond Hi-Bond High Strength (H.S.) composite steel/concrete floor system brochure is designed as an aid to specifying authorities and builders. This Product data brochure should be read in conjunction with the 'Dimond Steel Floor Laying instructions' and it is not intended as a substitute for consultation with a qualified engineer or local authority.

Description

Dimond Hi-Bond H.S. is a composite steel/concrete floor system. In-situ concrete develops a composite action through the keying action of lugs on the ribs of the Hi-Bond which provide positive lateral and vertical bonding between the concrete and the Hi-Bond steel deck. Dimond Hi-Bond H.S. provides both permanent formwork and positive tensile reinforcement in one way reinforced concrete slab construction.

Dimond Hi-Bond H.S. is manufactured under license to Inland Ryerson, one of the largest floor deck manufacturers in the USA, who hold internationally recognised fire-ratings for their floor decks.

References

The following references including standards, bylaws and codes of practice etc. govern the manufacture of components, use and design of Hi-Bond H.S. composite steel/concrete floor systems.

NZS 1900 New Zealand Standard Model Building Bylaws Ch 8: 1976
General structural design and design loadings

NZS 1900 New Zealand Standard Model Building Bylaws Ch 9 Division 9:3: 1981
Concrete

NZS 3404 : 1977 *Code for design of steel structures*

NZS 4203: 1976 *Code of Practice for General Structural Design and Design Loadings for Buildings*

NZS 3101P: 1970 *Code of Practice for reinforced concrete — design*

NZS 3441: 1978 *Hot-dipped zinc-coated steel coil and cut lengths*

ACI 318-77 *Building Code Required for Reinforced Concrete*

AISI 1980 *Specification for the design of cold formed steel structural members.*

Advantages

Structural

Concrete bond with the steel deck produces a working combination making full use of the structural properties of each material. The result is a system which provides maximum strength that reduces reinforcement requirements, overall material quantities and provides lateral bracing for structural steel.

Fire

1, 1½ and 2 hour fire resistance ratings are obtainable for restrained assembles, based on overseas tests. Refer Underwriters Laboratory Inc. D902 for further information.

Rapid erection

Erection is fast and can be continued in any weather in which men will work. No heavy plant is required and the laid deck is a safe working platform which is capable of being used for light storage.

Easy to run services

Continuous dovetail slot allows for hanging piping, ducts, lighting and suspended ceiling systems.

Low cost

Important economic advantages accrue from:
- the elimination of formwork and minimising propping;
- speedy erection of the lightweight components;
- no special skills required;
- reduced slab thickness;
- reduced dead weight on foundations providing savings in material and excavation;
- low labour content;
- reduced investment or costs in craneage and plant.

High Strength

The use of high strength steel enables one gauge to be used for the complete range of spans.

Installation

New design tongue and groove joint avoids rivetting by crimping (see photograph opposite.) A quick crimping tool is available for hire at the time of ordering.

Structural compatibility

Dimond Hi-Bond H.S. may be used with structures of in-situ or precast concrete, concrete masonry, steel or concrete structural frames, or lightweight galvanised steel Zed or Cee beams.

Composite action

Composite action or partial composite action, with a steel beam, can be obtained with shear studs, pins or plates attached to the beam in such a way as to be firmly embedded in the slab, by using powder activated fasteners, welded cleats or studs.

Applications

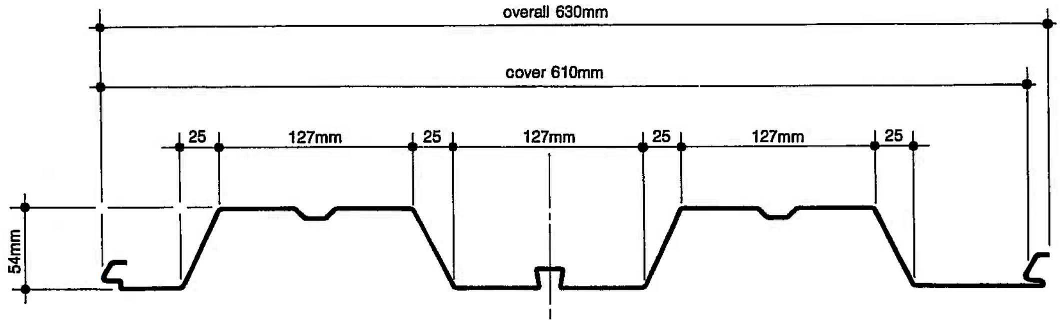
Suitable for use as suspended floors in:
- individual residences
- multi-unit apartments
- motels
- office buildings
- schools
- industrial buildings
- shops

Dimond Hi-Bond H.S. may also be used in built-up roofing construction.

Manufacture

Dimond Hi-Bond H.S. is manufactured from hot dipped galvanised steel coil and is formed to the profile illustrated on page 4. Slanted lugs are formed on the ribs facing in opposite directions on adjacent sections to provide positive lateral and vertical bonding between the concrete slab and the Hi-Bond steel deck. Side laps are specially designed to allow adjacent panels to act integrally. Trapezoidal grooves are formed in each bottom pan of the panel into which key hanger tabs can be inserted for the purpose of supporting ceilings and/or services below the slab.

Dimensions, Shape and Weight



Thickness	:	0.75mm BMT
Width of cover	:	610mm
Width overall	:	630mm
Length	:	available in any lengths to suit spans up to a maximum length of 14 metres
Weight	:	5.31 kg/m
Tolerances	:	-0mm and + 10 mm

Section Properties

Table 1 (a) 0.75mm Dimond Hi-Bond, G500 Material

Fy MPa (min)	Mass kg/m^2	If mm^4/m $x10^6$	Id mm^4/m $x10^6$	Z(+) mm^3/m $x10^3$	Z(-) mm^3/m $x10^3$	Z(b) mm^3/m $x10^3$	As mm^2/m	yb mm
500	8.71	0.522	0.452	12.9	14.0	18.7	1058	26.5

Table 1 (b) Composite Section (deck and concrete combined) for 20 MPa and 30 MPa concrete

Slab Thickness mm	Weight kPa	Ic $10^6mm^4/m$ 20 MPa	Ic $10^6mm^4/m$ 30 MPa	Zct $10^3mm^3/m$ 20 MPa	Zct $10^3mm^3/m$ 30 MPa	Zcb $10^3mm^3/m$ 20 MPa	Zcb $10^3mm^3/m$ 30 MPa
100	1.7	3.48	3.67	1110	1030	50	51
110	2.0	4.49	4.72	1320	1230	58	59
120	2.2	5.66	5.95	1550	1440	66	67
130	2.4	7.00	7.34	1800	1680	75	76
140	2.6	8.50	8.90	2070	1920	84	85
150	2.9	10.2	10.6	2360	2190	93	95
160	3.1	12.0	12.6	2660	2460	102	104
170	3.3	14.0	14.6	2970	2760	112	113
180	3.5	16.2	16.9	3300	3060	121	123
190	3.8	18.6	19.3	3640	3370	131	133
200	4.0	21.1	22.0	4000	3700	140	142

Notation used above

Fy	Yield stress of steel
If	Full moment of inertia of deck, based on gross section properties.
Id	Moment of inertia of deck for deflection.
Z(+)	Minimum section modulus of deck, top in compression.
Z(-)	Minimum section modulus of deck, top in tension.
Z(b)	Section modulus of deck to bottom fibre, top in compression.
As	Area of steel deck
yb	Distance to neutral axis of deck from bottom.
Ic	Composite moment of inertia.
Zct	Composite section modulus to top of concrete.
Zcb	Composite section modulus to bottom of steel.

Design Data

Floor Loadings

The total loading of a floor construction is made up of three load types:

(a) Dead Load
(b) Superimposed Live Load
(c) Superimposed Dead Loads

(a) The dead load comprises the sum of the weights of individual components of the slab, which are the Dimond Hi-Bond H.S. Steel decking, concrete, and reinforcing mesh. Any supplementary reinforcing steel has not been taken into account.

(b) Superimposed Live Load intensities for various building occupancy types are regulated by bylaws and the New Zealand Standard NZS 4203: 1976 *General Structural design and design loads for buildings* (see Table 2).

(c) Superimposed Dead Loads are allowances for any permanent and semi-permanent items suspended from or supported by the floor slab. These include partitions, suspended ceilings, building services (plumbing, electrical, air conditioning, etc).

Table 2 Minimum Basic Live Loads for Floors & Stairs
Extract from Table 2 NZS 4203: 1976.

	(kPa)
1. DOMESTIC (dwellings, flats, etc)	
1.1 Attics	0.5
1.2 Balconies	2.0
1.3 Corridors, hallways, passageways, foyers, lobbies, stairways and landings	as for floor serviced
1.4 Other floors	1.5
6. OFFICES (offices, banks, etc)	
6.11 Offices for general use	2.5

Propping

Temporary propping of steel deck between structural supports will be required as shown on the load-span quick selection graphs. This is to be used as minimum guide only as propping is considered "scaffolding" under the Construction Regulations 1961, Clause 49-51. Job conditions vary including the slenderness ratio and lateral stability of the propping system to be used, but should consist of substantial timber or steel members supported by props which should be adjusted to prevent settlement and strong enough to support both wet concrete and construction live loads.

Where Hi-Bond H.S. is used as permanent formwork with very thick slabs, consideration should be given to the support of the deck due to the wet concrete load. This can usually be satisfied by extra propping or the use of heavier decks which could be available on special request.

When ceilings are finished directly onto the Hi-Bond, bottom surface deflections may become critical for architectural reasons and induced camber or additional propping may be necessary to avoid permanent deflection.

Note: Deflections given in the load/span graphs are top surface deflections of a flat screeded slab after propping has been removed.

Recommended secondary reinforcement

Table 3

Slab thickness mm	Recommended mesh
100 to 115	668
116 to 130	666
131 to 150	665
151 to 200	664

Negative reinforcement

Where the slab is designed to have continuity, negative reinforcement may be required by the engineer, in addition to that noted in Table 3. Table 4 shows typical negative reinforcement required for cantilever spans.

Cantilever spans

Table 4 Cantilever Reinforcement *

Slab Depth mm	Span m	Reinforcement for up to 4.0 kPa Superimposed Load
130	1.8	2-D16
	1.5	2-D12
	1.2	2-D10
140	2.0	2-D16
	1.8	2-D16
	1.5	2-D12
	1.2	2-D10
150	2.0	2-D16
	1.8	2-D16
	1.5	2-D12
	1.2	2-D10

*Note: It is important that the required reinforcement be placed over each trough.

Openings in Hi-Bond H.S. floors

Penetrations may be formed by conventional formwork or by use of polystyrene infills, and the opening gas or saw cut out after the concrete has cured. Where extra loads are applied from items attached to the opening additional structural frame may be necessary. Normally openings in Hi-Bond H.S. floors can be classified into three broad categories — see Table 5.

Table 5 Openings in Hi-Bond H.S. Floors

Opening width measured at right angles to deck span	Treatment
Less than 250 mm	Form prior to concrete pour. Once concrete attains 75% of design strength, the deck can be cut out.
250 mm to 750 mm	Reinforce deck prior to concrete pour with reinforcing bars or small channels welded to the deck around the perimenter of the opening to distribute the loads to the adjacent panels.
Greater than 750 mm	The most practical method is to supply supplementary structural frame.

Dimond Hi-Bond H.S. is a high strength steel and therefore replacing the area of deck cut out with the same area of mild steel reinforcement may not be adequate.

Design Data (contd.)

Load Span Graphs

The load/span quick selection graphs below are generally applicable to simply supported domestic applications (flats, dwellings, etc).

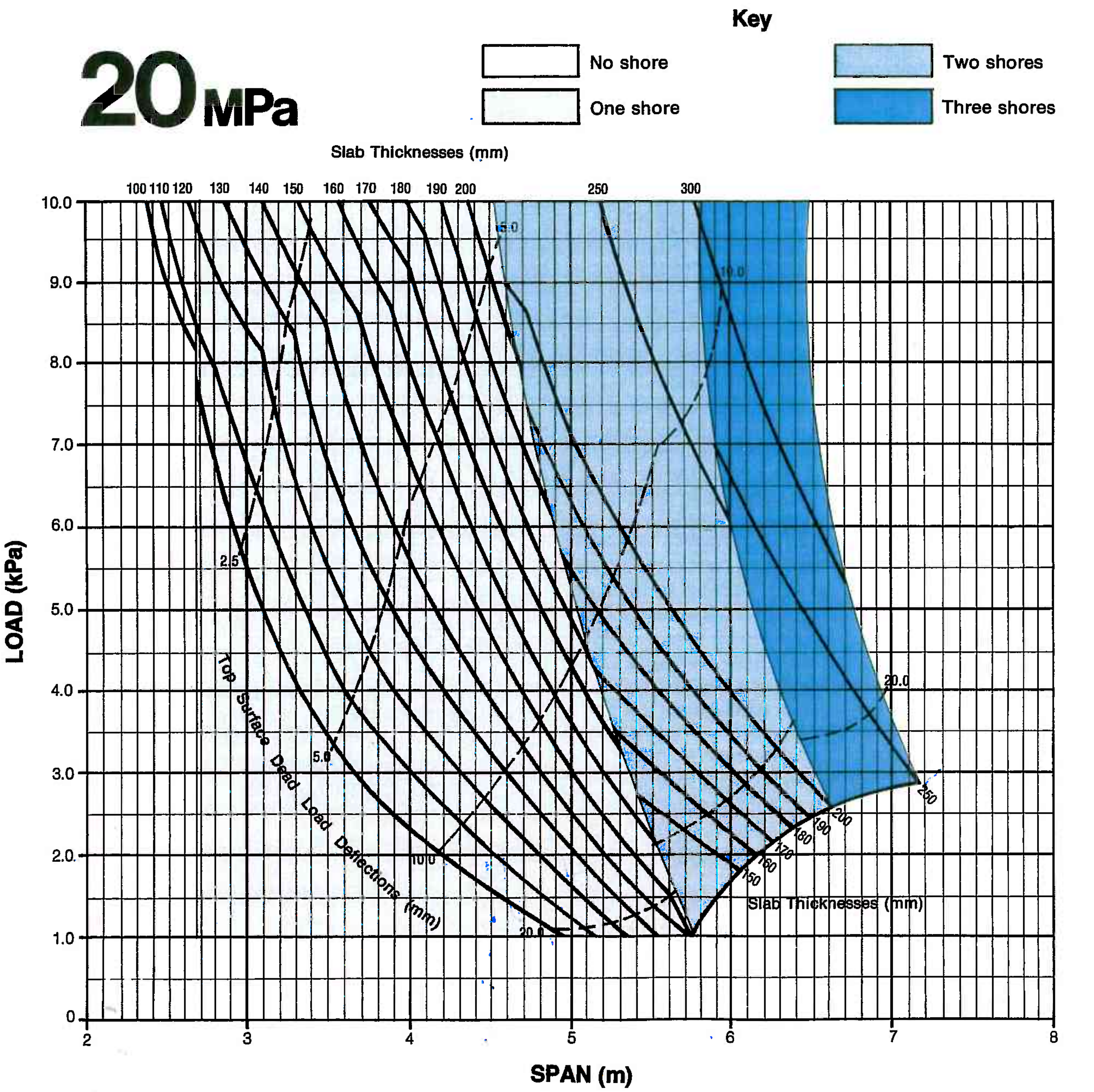
The graphs are based on the following assumptions:

(1) Construction load is assumed to be a uniform 1.0 kPa.
(2) Weight of concrete (including ponding effect) is allowed for as a uniformly distributed load.
(3) Deflection calculations are based on cracked cross-section properties, and are limited to ACI 318-77 criteria for floors not supporting or attached to non-structural elements likely to be damaged by large deflections.
(4) Slab design is to the ACI 318-77 alternate design method, for a simply supported span, with no openings.
(5) Superimposed dead loads are assumed to be small compared with the superimposed live load.

The quick selection graphs should not be used as the sole design criteria when conditions are significantly different from those above.

To use Load/Span Graphs

(1) Check that slab complies with the conditions above.
(2) Calculate the total superimposed load as follows:
  (a) Select applicable live load from Table 2, NZS 4203: 1976
  (b) Add superimposed dead loads as appropriate, for partitions, floor covering, suspended ceilings, building services, etc, supported by the floor slab.
  (c) Total (a) and (b) above to give the design superimposed load.
(3) Enter relevant graph for concrete strength (20 MPa or 30 MPa) with the known simply supported span.
(4) Select a slab thickness, ensuring that it has a safe superimposed load equal to or greater than the design superimposed load calculated in (2) above for the known span.
(5) Check if the top surface dead load deflection is acceptable



Extending range of spans

Note that greater spans than indicated on the load/span graphs are possible by one or more of the following methods:

(1) The use of additional shores and thicker slabs.

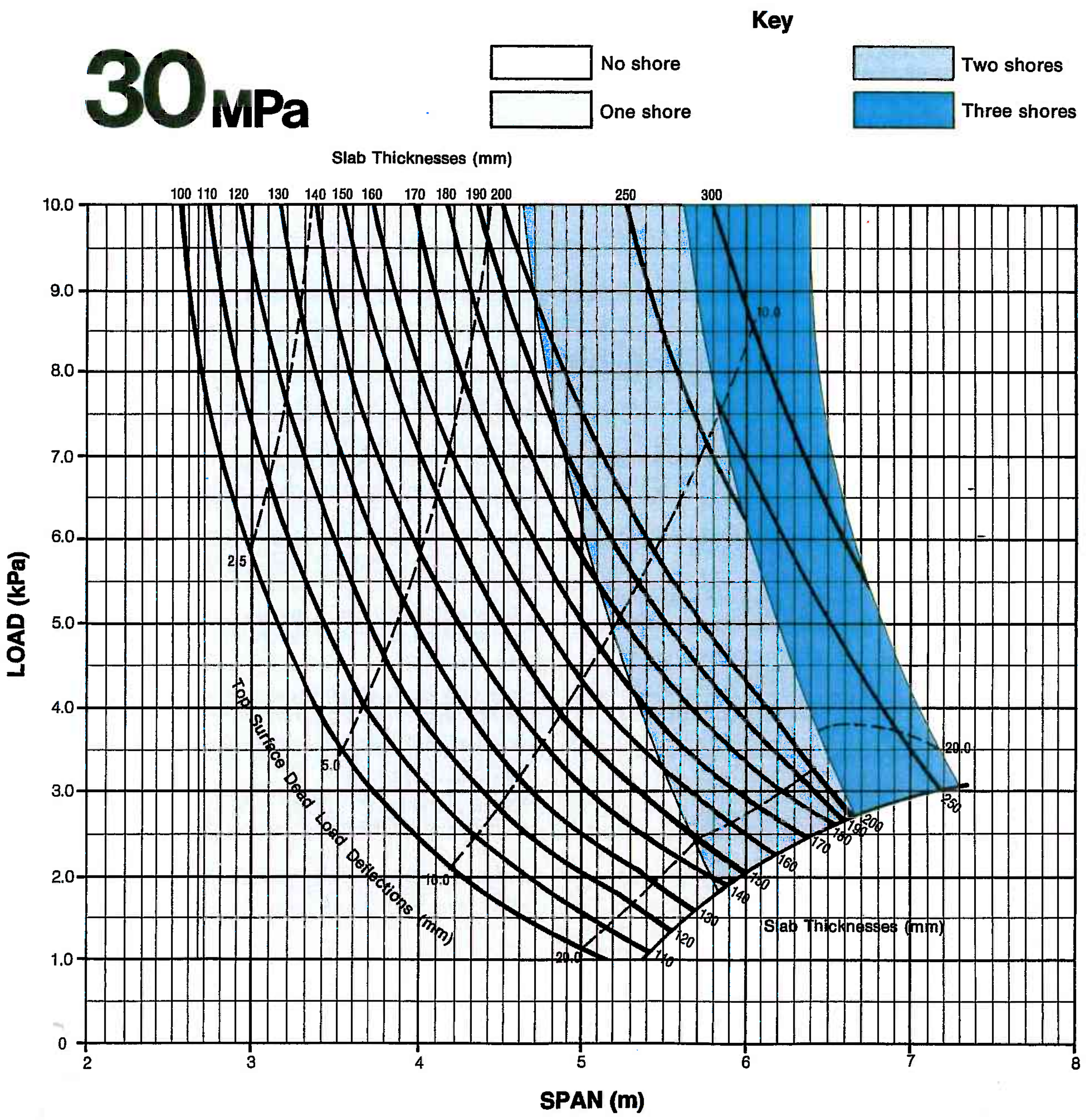
(2) Some camber can be introduced into the decking before pouring the concrete, generally between 10 and 20mm for a 6m span, note that to obtain a truly flat deck, allowance should be made when screeding.

(3) The use of higher strength concrete. Note from comparison of the 20 MPa and the 30 MPa load/span graphs, that this is especially beneficial for long spans and 2 or more shores.

(4) The use of shallow concrete support beams, which means that the Hi-Bond slab is only required to span the clear distance between beams. Refer Construction Details, Fig. 4.

(5) The load/span graphs assume that the slab is simply supported. By making the slab continuous, using negative reinforcement over the supports, significantly greater spans are possible.

Where heavy sustained loads and long spans are used, reference should be made to a Consulting Engineer.



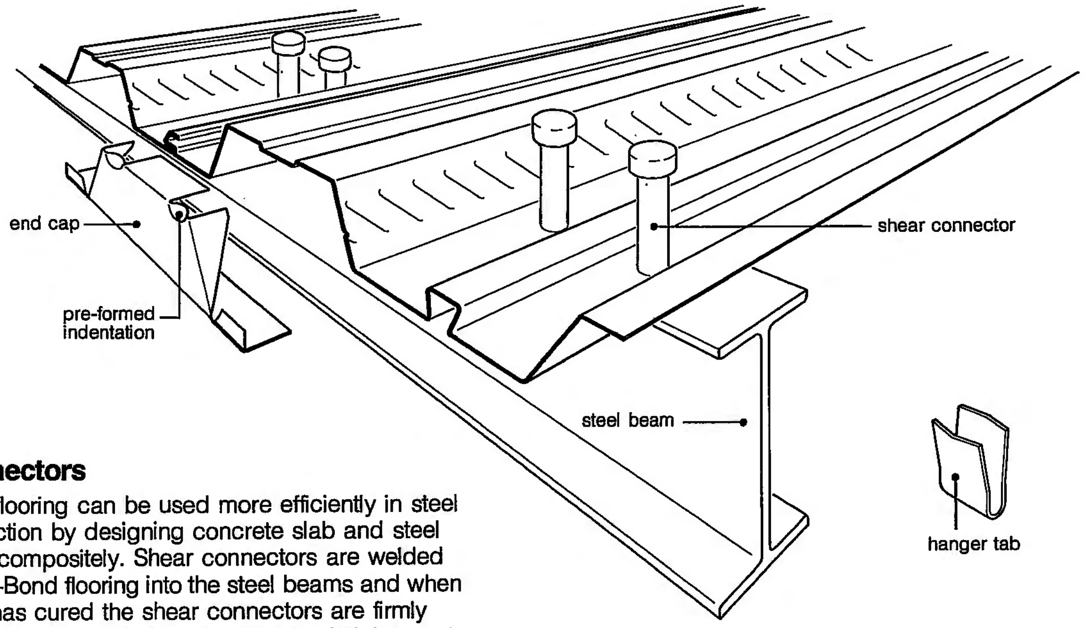
Accessories

End caps

Formed end caps are available to close the inverted channel portions of the deck when used with in-situ concrete beam applications. These can be pop riveted or screwed in place but a simple deformation over the indentation provides adequate holding.

Hanger tabs

The hanger tabs are inserted parallel to the trapezoidal groove and rotated through 90° to provide a suspension point for a suspended ceiling, ductwork or piping etc. Safe load per 1.6mm hanger is 130 kgs.



Shear connectors

Hi-Bond H.S. flooring can be used more efficiently in steel beam construction by designing concrete slab and steel beams to act compositely. Shear connectors are welded through the Hi-Bond flooring into the steel beams and when the concrete has cured the shear connectors are firmly embedded in the slab and allow the steel and slab to act compositely.

A range of shear connectors and a fixing service is available for use with Hi-Bond H.S. flooring.

Performance Properties

Fire

Minimum fire resistance ratings for floors according to occupancy classification are detailed in NZS 1900 Model Building Bylaw Chapter 5 *Fire Resisting Construction and Means of Egress.*

To meet these requirements Hi-Bond H.S. may be left bare or protected as indicated in the Tables below.

UL D902 Fire test gives the following fire ratings, without protection for restrained assemblies. Copies are available on request.

Table 6 (a) Unprotected Deck Fire Ratings

Rating (hours)	Overall depth of slab (mm)
1	143
1½	156
2	169

The following extract from Table 7B of NZMP 9/7 1966 provides fire resistance ratings for protected decks.

Table 6 (b) Protected Deck Fire Ratings

Rating (hours)	Minimum overall slab depth (118 mm)
2	12.7mm gypsum plaster suspended on metal lathing
4	12.7mm vermiculite- or perlite-gypsum plaster suspended on metal lathing

Durability

Corrosion protection

Dimond Hi-Bond H.S. decks are positive tensile reinforcement and must be protected wherever corrosion could lead to a reduction in structural performance. Subfloor areas should be adequately ventilated and should not be less than 450mm above bare ground to minimise the risk of corrosion.

The use of Hi-Bond H.S. decks is not recommended where corrosion hazards exist such as in severe industrial environments.

In exterior applications it is recommended that a high build paint type coating is given to the underside of the Hi-Bond H.S. decking to provide additional protection.

Dynamic behaviour

Although the criteria used for the quick selection graphs complies with ACI 318-77 deflection limits, it is possible that thin slabs over long spans may be susceptible to some vibration when the proportion of superimposed dead load to live load is small.

Design Example

The example below is meant only to clarify the use of the table since other factors must be considered for a complete floor design.

Problem

To determine slab depth, shoring and reinforcement required for the floor over the garage area of the dwelling shown in Fig (i).

The typical superimposed loads are:
(Refer to Table 2)

Balcony superimposed live load	2.0 kPa
Floor superimposed live load	1.5 kPa
Floor superimposed dead load	
from — partitions	1.25 kPa
— ceiling, flooring, etc	0.25 kPa

There are several solutions to the problem depending on the area of garage required.

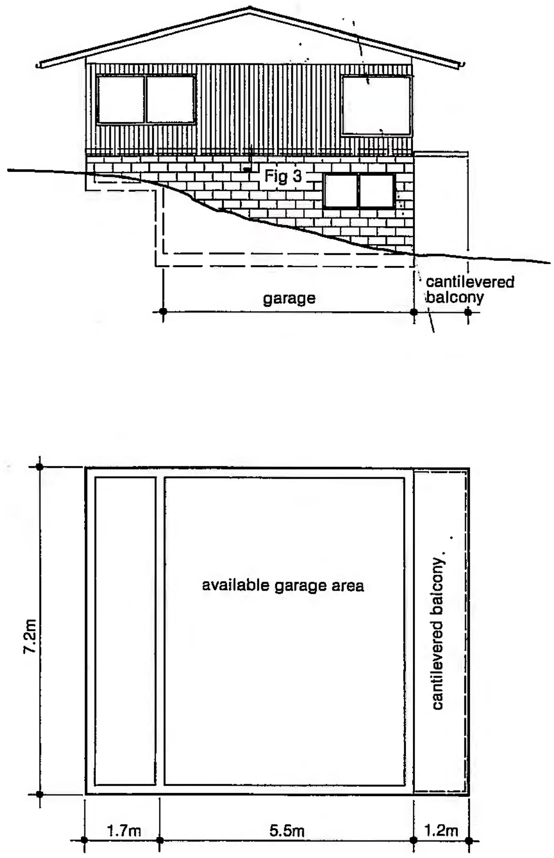


Fig (i) Elevation and plan

Two alternatives are considered:

Alternative A
Floor

As shown in Fig (ii) the garage area is divided in half by a beam and columns, with the decking spanning 3.6m across the width of the dwelling.

Total superimposed load = 1.5 + 1.25 + 0.25 = 3.0 kPa
Enter 20 MPa QS graph with 3.0 kPa and 3.6m span.

0.75mm decking with 100mm slab is suitable
1 row of props required
From Table 3, use 668 mesh in the top of the slab.

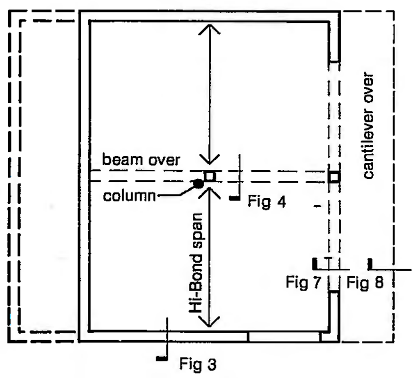


Fig (ii) Basement Plan
Alternative A (using 20 MPa concrete)

Alternative B
Floor

As shown in Fig (iii) the garage area is the full width with the decking spanning 5.5m over the full depth of the garage.

Total superimposed load = 1.5 + 1.25 + 0.25 = 3.0 kPa
Enter 30 MPa QS graph with 3.0 kPa and 5.5m span

0.75mm decking with 160mm slab is suitable
1 row of props is required
From Table 3, use 664 mesh in the top slab.

Balcony

Cantilever span of 1.2m
Total superimposed load = 2.0 + 0.25 = 2.25 kPa

From Table 4, use 0.75mm decking with 130mm slab and 2-D10 bars above each trough of the decking.

Comparative costs can be assembled, once the beams and columns have been chosen, and a choice made be weighing the costs against the benefits of the two alternatives.

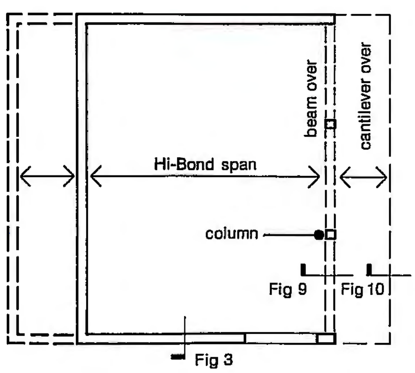


Fig (iii) Basement Plan
Alternative B (Using 30 MPa concrete)

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Construction Details

Scale: 1:10 unless otherwise specified

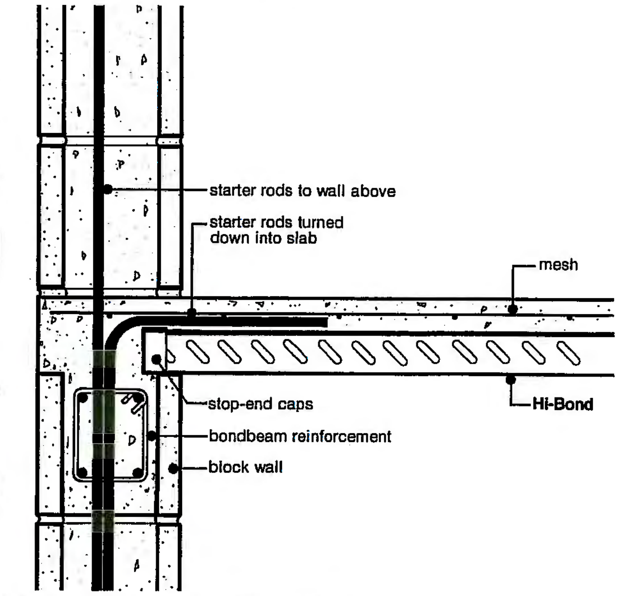

Fig 1 Masonry wall support with masonry wall above

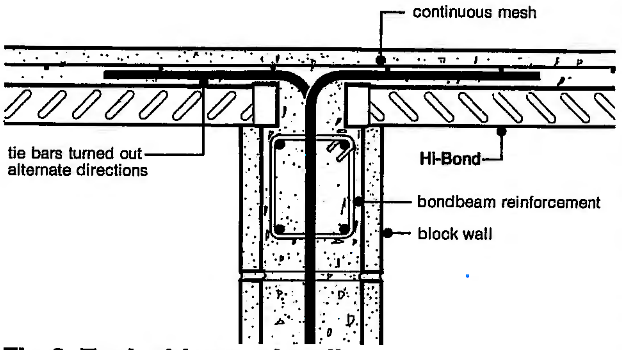

Fig 2 Typical internal wall

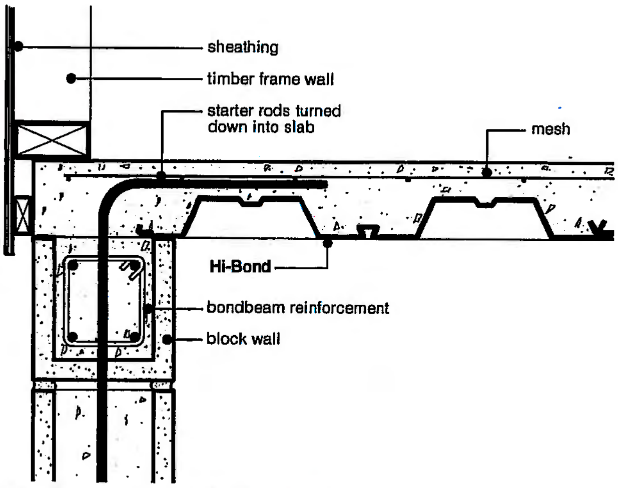

Fig 3 Masonry wall support with timber wall above

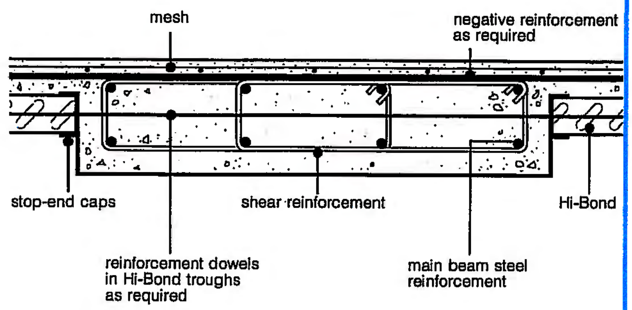

Fig 4 Concrete beam support

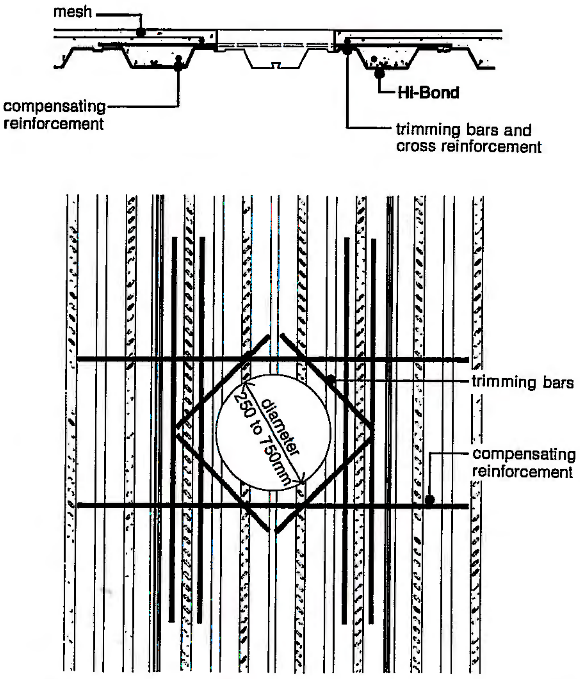

Fig 5 Opening detail Scale: 1:20

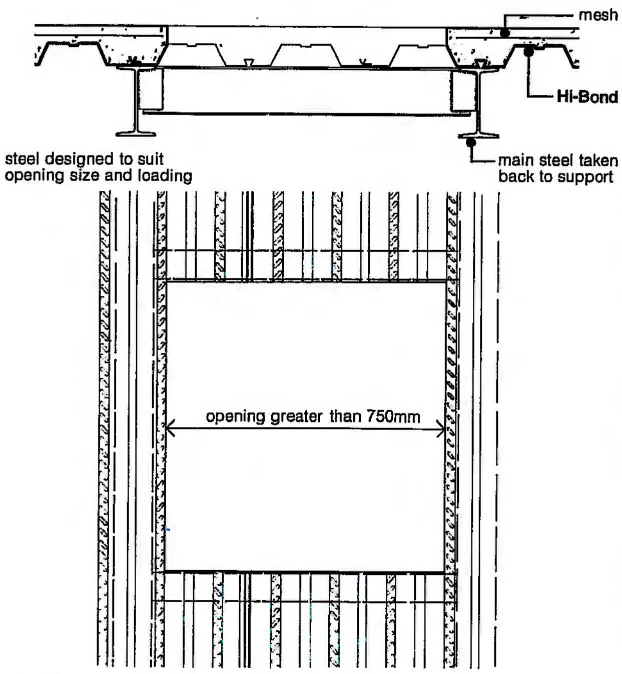

Fig 6 Opening detail Scale: 1:20

Construction Details

Scale: 1:10 unless otherwise specified

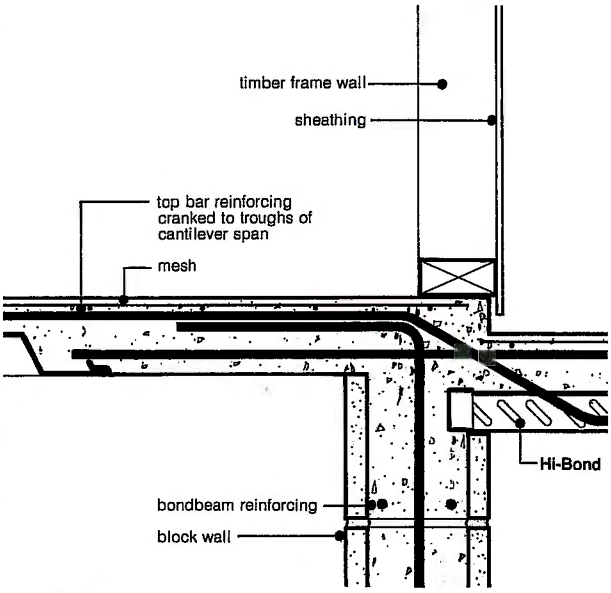

Fig 7 Cantilever — masonry wall
at right angles to general span of Hi-Bond

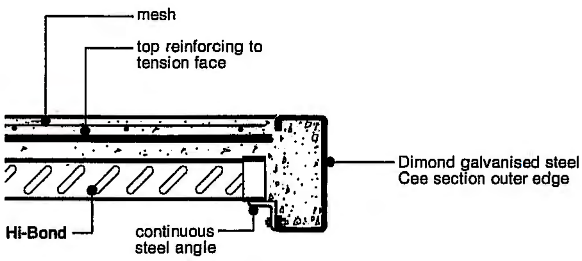

Fig 8 Balcony edge
with Cee section edge

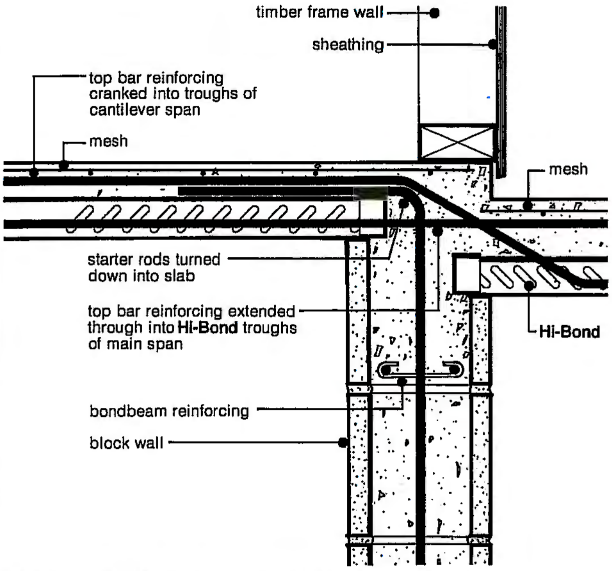

Fig 9 Cantilever — masonry wall
parallel to general span of Hi-Bond

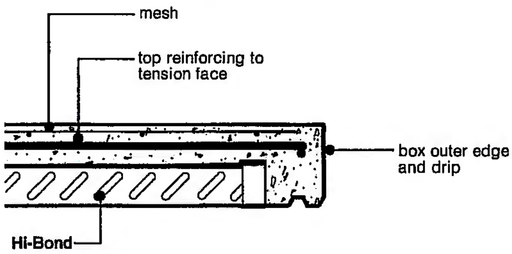

Fig 10 Balcony edge
with formed drip

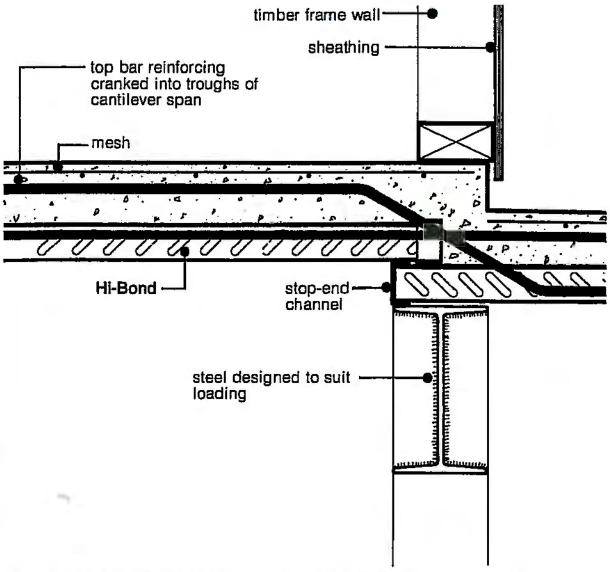

Fig 11 Cantilever — steel beam support
parallel to general span of Hi-Bond

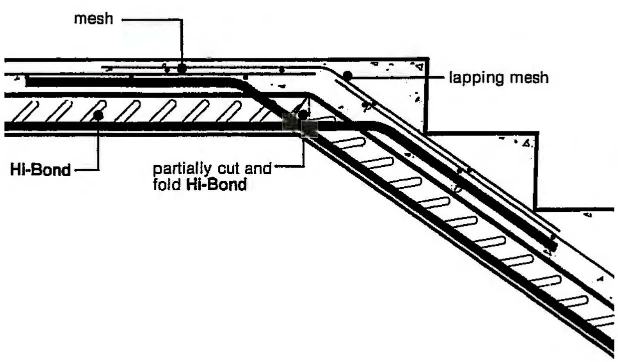

Fig 12 Typical stair detail

Site Work

Handling and storage

Dimond Hi-Bond H.S. decks are normally delivered to the building site by road transport. To obtain maximum advantage delivery should be scheduled so that the decks may be lifted directly from the truck and installed in their final position in the building. If the decks have to be site stored awaiting installation they should be neatly stacked clear of the ground and protected by waterproof covers. If rain or condensation is trapped between stacked decks, wet storage staining may occur which may prove difficult to remove. Care must be taken when lifting long lengths or bundles of decks and several lifting points used to prevent buckling.

Delivery

Delivery is generally three to four weeks from the date of ordering. Check the delivery period when ordering and remember to allow lead time for railage where necessary.

Price

The latest Hi-Bond H.S. price list is available from Dimond Industries Ltd.

Technical Services

Dimond Industries Ltd Technical Representatives are available to designers and contractors to advise on all matters related to the best use of Dimond Hi-Bond H.S. composite steel/concrete floor system.

The information contained in this publication is intended to give a fair description of Dimond Hi-Bond H.S. and its capabilities. It does not constitute an offer by the manufacturers nor do they warrant or guarantee its accuracy or completeness describing the performance or suitability of Dimond Hi-Bond H.S. composite steel/concrete floor system.

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Appendix C. Paper by Williams and Paulay

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SECTION B

THE ANALYSIS AND DESIGN OF AND THE EVALUATION OF DESIGN ACTIONS FOR REINFORCED CONCRETE DUCTILE SHEAR WALL STRUCTURES

T. Paulay* and R.L. Williams**

ABSTRACT:

A comprehensive review of the state of the art in the design of earthquake resisting ductile structural walls is presented. The material has been compiled from the technical literature, the deliberations within the New Zealand National Society for Earthquake Engineering and research efforts at the University of Canterbury. The paper attempts a classification of structural types and elaborates on the hierarchy in energy dissipation. After a review of available analysis procedures, including modelling assumptions, a detailed description of capacity design procedures for both cantilever and coupled shear wall structures is given. The primary purpose of capacity design is to evaluate the critical design actions which can be used in the proportioning and reinforcing of wall actions which can be used in the proportioning and reinforcing of wall sections. An approach to the estimation of structural deformation is suggested. To satisfy the ductility demands imposed by the largest expected earthquake, detailed design and detailing recommendations are given and the application of some of these is presented in an appendix.

INTRODUCTION:

The usefulness of structural walls in the planning of multistorey buildings has long been recognized. When walls are situated in advantageous positions in a building, they can become very efficient in lateral load resistance, while also fulfilling other functional requirements.

Because a large portion of the lateral load on a building, if not the whole amount, and the horizontal shear force resulting from it, are often assigned to such structural elements, they have been called shear walls. The name is unfortunate because shear should not be the critical parameter of behaviour.

The basic criteria that the designer will aim to satisfy when using structural walls in earthquake resistant structures are as follows:

(a) To provide adequate stiffness so that during moderate seismic disturbances complete protection against damage, particularly in non-structural components, is assured.

(b) To provide adequate strength to ensure that an elastic seismic response, generating forces of the order specified by building codes(1), does not result in more than superficial structural damage. Even though during such an event some non-structural damage is expected, it is unlikely that in buildings with well designed shear walls this will be serious.

(c) To provide adequate structural ductility and capability to dissipate energy for the case when the largest disturbance to be expected in the region does occur. Extensive damage, perhaps beyond the possibility of repair, is accepted under these extreme conditions, but collapse must be prevented.

(d) The subsequent sections concentrate on those aspects of the design and response of structural walls that are relevant to this third design criterion. Consequently the inelastic response of structural walls, when subjected to simulated cyclic reversed loading, together with various parameters that must affect this response, will be examined in some detail for various types of structures. It will be assumed that in all cases adequate foundations can be provided so that rocking will not occur and that energy dissipation, when required, will take place in the structural wall above foundation level. A detailed discussion of concepts, relevant to the design of foundations for shear wall structures, is provided in Reference 5. Also it will be assumed that:

(i) Inertia forces at each floor can be introduced to the structural wall by adequate connections, such as collector beams or diaphragms and from the floor system, and that

(ii) The foundation for each wall does not significantly affect its stiffness relative to similar other walls in a building.

TYPES OF DUCTILE STRUCTURAL WALLS:

In this section the principles of the analyses and the design of earthquake resisting structural walls, in which significant amounts of energy can be dissipated by flexural yielding in the superstructure, are examined. The prerequisite in the design of such seismic walls is that flexural yielding in clearly defined plastic hinge zones must control the strength to be utilized during imposed inelastic seismic displacements. As a

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corollary to this requirement, failure due to shear, inadequate anchorage or splicing of the reinforcement, instability of concrete components or compression bars and sliding along construction joints must be avoided, while large inelastic seismic displacements are sustained by the structure. Some of the failure modes mentioned are illustrated in figure 1.

In the evaluation of the equivalent lateral static design load, to be used in establishing the minimum seismic strength of a structure, the New Zealand Design and Loading Code(1) specifies structural type factors, S. These factors are intended to reflect the expected seismic performance of the structure. There are two aspects which are to be considered in the assessment of performance, one is the ability of the type of structure to dissipate energy in a number of inelastic displacement cycles, and the other is the degree of redundancy existing in the chosen structural system. A high degree of structural redundancy, involving a large number of localities where energy dissipation by flexural yielding can occur, is desirable.

Accordingly it is recommended that earthquake resisting ductile structural walls be classified as follows:

(a) Two or more cantilever walls with a height, h_w, to horizontal length, ℓ_w, ratio of not less than two are assigned a structural type factor of S = 1.0 (see figure 2a).

(b) For two or more cantilever walls, each with an aspect ratio h_w/ℓ_w not less than two, which are coupled by a number of appropriately reinforced ductile coupling beams that are capable of dissipating a significant portion of the seismic energy, the value of S is 0.8. This is in recognition of the high degree of redundancy and the fact that damage is likely to be small in the gravity load carrying elements.

The significance of the coupling beams in energy dissipation is conveniently expressed by the contribution of the coupling beams to the total overturning moment that is produced by the code specified lateral loading at the base of the coupled shear wall structure. This is illustrated in figure 17. A suitable parameter which expresses this is the moment ratio

$$A = \frac{T\ell}{M_o} \qquad \text{(B-1)}$$

where T = induced axial load in one of the two coupled shear walls at the base of the structure due to the code specified lateral static loading

ℓ = distance between axes of the two walls

M_o = overturning moment due to the load inducing T, about the base of the structure

These quantities may be seen in figure 17.

Depending on the contribution of the beams to the resistance of overturning moment and hence to total energy dissipation, the structural type factor, S, is made dependent on the moment ratio, A, thus

when $0.67 \geq A \geq 0.33$ (B-2)

then $0.8 \leq S = 0.8 + 0.6 \times (0.67 - A) \leq 1.0$ (B-3)

For intermediate values of A a linear interpolation of S may be made. The application of this is discussed in detail in section B.5.3.4.

Typically for a wall with deep coupling beams, illustrated in figure 17b, the appropriate S factor is likely to be 0.8. When walls are interconnected by slabs only, (figure 17c) as is often the case in apartment buildings, the value of A from Eq. (B-1) will usually be much less than 0.33 and hence S = 1.0. A comparison of the moment contribution of the ℓT component to the total overturning moment M_o is shown in figure 18.

(c) Single cantilever walls, with $h_w/\ell_w > 2$, are to be designed with S = 1.2, to compensate for the lack of redundancy. (See figure 2b).

(d) Squat cantilever walls with an aspect ratio of $h_w/\ell_w < 2$, in which shear effects are likely to be dominant, are not expected to produce as efficient energy dissipation due to flexural ductility as more slender structural walls. Shear deformations, particularly shear sliding, may cause significant pinching in the hysteresis loops exhibited by squat shear walls(2), and thereby loss of energy dissipation will occur.

In order to reduce the displacement ductility demand on squat walls, the strength of the walls with respect to seismic loading should be increased. Hence for walls for which $1 \leq h_w/\ell_w \leq 2$, the structural type factor given above in (a), (b) and (c) should be multiplied by Z where

$1 \leq Z = 2.2 - 0.6\, h_w/\ell_w \leq 1.6$ (B-4)

It is to be noted that the use of higher structural type factors, i.e. S = 1.6 x 1.0 = 1.6 or S = 1.6 x 1.2 = 1.92, is expected only to reduce but not to eliminate the ductility demand on squat shear walls.

Squat walls will have a relatively low fundamental period (T < 0.6 sec). It is known that short period structures, designed to the requirements of the New Zealand loading code(1), are likely to be subjected to higher ductility demands than long period structures.

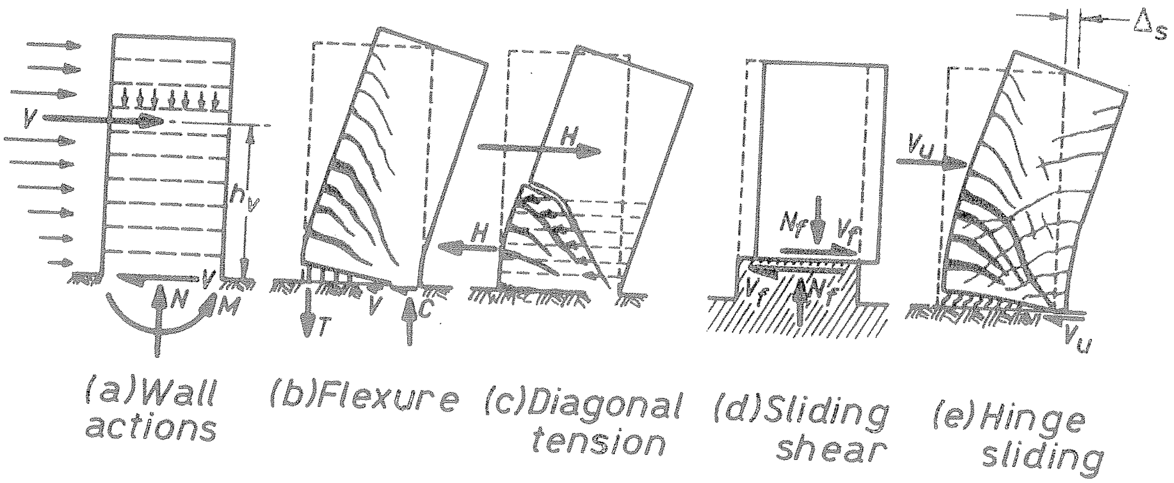

Fig. 1 - Possible Failure Modes in Cantilever Shear Walls

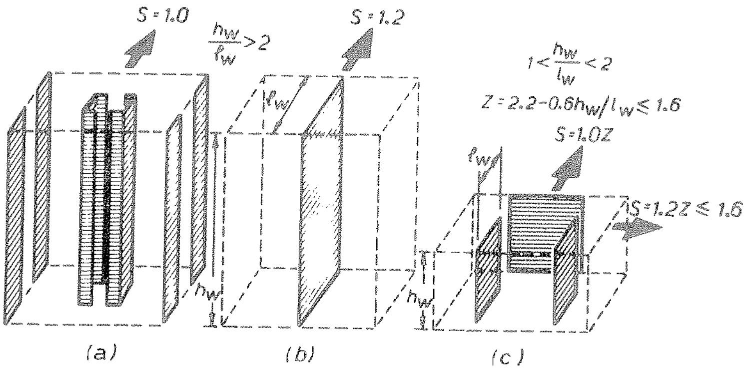

Fig. 2 - Types of Cantilever Shear Walls with Appropriate S Factors

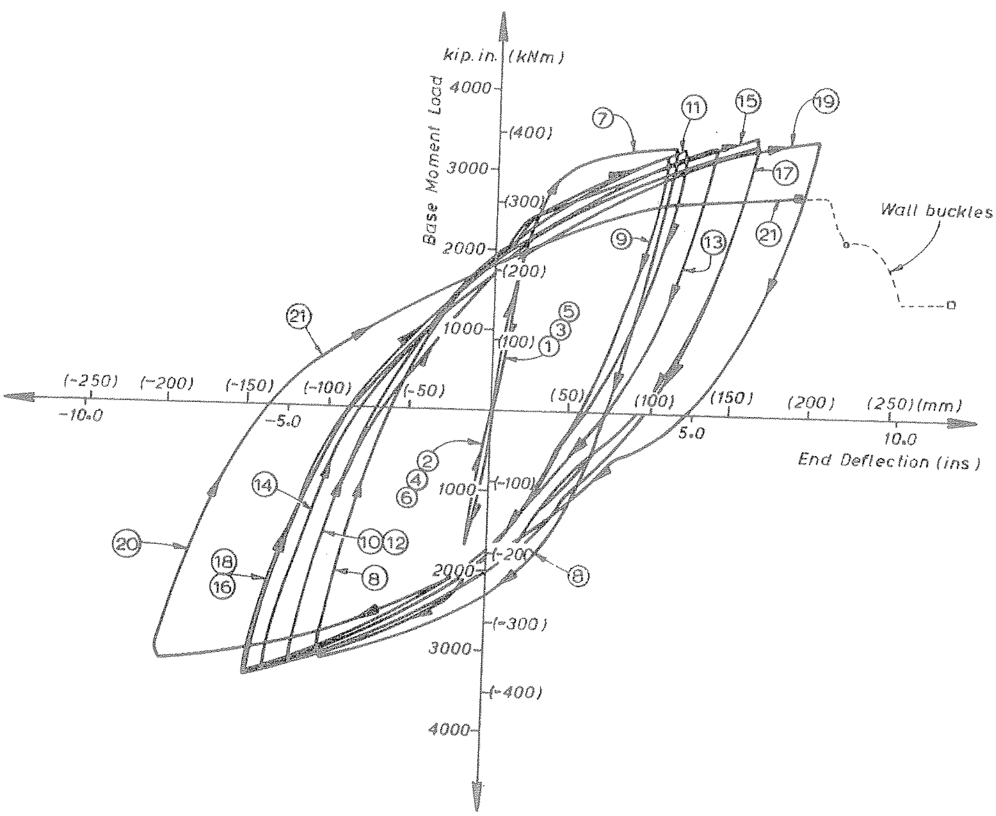

Fig. 3 - Load-Displacement Response to Cyclic Reversed Loading of a Ductile Shear Wall Structure (10)

Moreover, in a given earthquake, a short period squat shear wall is likely to be subjected to a greater number of excursions beyond yield than a long period structure. Therefore the cumulative ductility, which has some relevance to damage, is still high. These observations indicate that squat shear walls, such as shown in figure 2c, designed with a modified structural type factor, S, must also be ductile and hence they must be detailed accordingly.

Structural walls of different types are reviewed in Reference 3 and detailed procedures recommended for walls which cannot be made fully ductile are presented in Reference 4. The requirements for the design of foundations which can sustain inelastic superstructures when their maximum feasible seismic strength is being developed, are examined in Reference 5.

HIERARCHY IN ENERGY DISSIPATION:

It is generally accepted that for most siguations energy dissipation by hysteretic damping is a viable means by which structural survival of large earthquake imposed displacements can be assured. This may involve very large excursions beyond yield. Such structures must therefore be ductile. To ensure the desired energy dissipation, the designer's primary aim will be to minimize the inevitable degradation in both stiffness and strength.

Flexural Yielding of Ductile Walls

An obvious source of hysteretic damping is the yielding of the principal flexural reinforcement. Yielding can be restricted to well defined plastic hinge zones, as shown in figure 1b. Therefore such areas deserve special attention. Concrete, being a relatively brittle material that shows rapid strength degradation, in both compression and shear, when subjected to repeated inelastic strains and multi-directional cracking, should not be considered in structural walls as a significant source of energy dissipation. To ensure the desired ductility, the major part of the internal forces in the potential plastic region of a shear wall should therefore be allocated to reinforcement. The desired response of a ductile shear wall structure manifests itself in well rounded load-displacement hysteresis loops, such as shown in figure 3.

Control of Shear Distortions

While shear resisting mechanisms in reinforced concrete, that rely on the traditional truss mechanism (figure 1c), can be made relatively ductile in shear during monotonic loading, they are generally unsuitable for inelastic cyclic shear loading. Shear resistance after inelastic shear displacements can be attained only when the subsequent imposed displacement is larger than the largest previously encountered displacement. Inelastic tensile strains in stirrup reinforcement can never be recovered and hence in such cases the width of diagonal cracks also increases with progressive cyclic loading. Curves 3 and 4 in figure 4 show typical load displacement responses for one quadrant of a displacement cycle, which have been affected by significant shear displacements. In comparison curves 1 and 2 show the idealized elastic-plastic and the optimal response of a reinforced concrete member. In order to minimize the 'pinching' of hysteresis loops, i.e. the loss of energy dissipating capacity within restricted displacements, designers should endeavour to suppress inelastic shear distortions. In conventionally reinforced walls the detrimental effect of shear increases with the magnitude of the shear stress. For example figure 5 shows the hysteretic response of a cantilever shear wall in which, due to relatively large shear stresses, shear deformations have become increasingly significant with increased cycles of loading and the amplitude of the applied deflection at the top of the wall. It is also seen that in each cycle the stiffness of the wall decreased, even though the full capacity of the wall was attained. The envelope curve follows closely the load-displacement curve that is obtained during monotonic loading with the same displacement ductility. If several cycles with the same magnitude to top displacement are applied, for example to 4 in (10 cm) in each direction, (see figure 5), the load attained would have gradually decreased in each cycle. Such a wall is likely to fulfill the design criteria but its performance is clearly inferior to that demonstrated in figure 4.

The Desired Hierarchy in Strength

From the features considered above it becomes evident that the design procedure must endeavour to minimize the likelihood of a shear failure, even during the largest intensity shaking. This is achieved by evaluating the flexural capacity of a wall from the properties shown on the structural drawings. With proper allowance for various factors, to be examined in "Capacity Design Procedures", the likely maximum of the moment that can be extracted from a shear wall structure during an extreme seismic inelastic displacement can be readily evaluated. The shear force associated with the development of such a moment can then be estimated. This must be done using conservative estimates. Subsequently the wall can be reinforced so as to possess corresponding shear strength.

When the shear strength of a wall is not in excess of the flexural strength, a situation which commonly arises in squat shear walls, not only does stiffness degradation occur but the attainable full capacity of wall will also reduce with cyclic displacements. Such an undesirable response is shown in figure 6.

Similar procedures must be followed to ensure that other undesirable failure modes, such as due to bond and anchorage of the reinforcement or sliding along construction joints, will not occur while the maximum flexural capacity of the wall, usually at its base, is being developed several times in both directions of the loading.

Capacity design procedures will ensure that the desired hierarchy in the energy dissipating mechanism can develop. The procedure is quantified and discussed in detail in "Capacity Design Procedures".

ANALYSIS PROCEDURES:

Modelling Assumptions

Modelling of member properties -

When, for the purpose of either a static or dynamic elastic analysis, stiffness properties of various elements of reinforced concrete shear wall structures need be evaluated, some approximate allowance for the effects of cracking should be made. In this, it is convenient to assume that reinforced concrete components exhibit properties that are similar to those of elements with identical geometric configurations but made of perfectly elastic, homogeneous and isotropic materials. For the sake of simplicity an approximate allowance for shear and anchorage deformations is also made.

These recommendations for modelling may be considered to lead to acceptable results when the primary purpose of the elastic analysis is the determination of internal structural actions that result from the specified lateral static loading or from dynamic modal responses. The estimates given below are considered to be satisfactory also for the purpose of predicting the fundamental period of the structure and for checking deflections in order to satisfy code specified limits for deflections or separations of non-structural components.

In ductile earthquake resisting structures significant inelastic deformations are expected. Consequently the allocation of internal design actions in accordance with an elastic analysis should be considered as one of several acceptable solutions which satisfy the unviolable requirements of internal and external equilibrium. As will be seen subsequently, deliberate departures in the allocation of design actions from the elastic solutions are not only possible, but they may also be desirable.

When it is necessary to make a realistic estimate of the deformations of an elastic wall system which is subjected to a relatively high intensity loading, the absolute value of the stiffness is required. Rather than specify a stiffness, an equivalent second moment of area of the wall section, I_w, will be defined in order to allow deflections to be estimated for various patterns of loading. The first loading of a wall up to and beyond first cracking is of little interest in design. In this recommendation only deformations of the wall, in which cracks have fully developed during previous cycles of elastic loading, will be considered.

In arriving at the equivalent stiffness of a wall section, flexural deformations of the cracked wall, anchorage deformations at the wall base and shear deformations after the onset of diagonal cracking should be considered. Detailed steps of these approximations are set out in Appendix I.

Deformations of the foundation structure and the supporting ground, such as tilting or sliding, are not considered in this study, as these produce only rigid body displacement for the shear wall superstructure. Such deformations should, however, be taken into account when the period of the structure is being evaluated or when the deformation of a shear wall is related to that of adjacent frames or walls which are supported on independent foundations(5).

Accordingly, for cantilever shear walls subjected predominantly to flexural deformations, the equivalent second moment of area may be taken as 60% of the value based on the uncracked gross concrete area of the cross section, with the contribution of reinforcement being ignored i.e.

$$I_e = 0.60\ I_g \qquad \text{(B-5)}$$

When elastic coupled shear walls are considered, where, in addition to flexural deformation, extensional distortions due to axial loads are also being considered, the equivalent moment of inertia and area may be estimated as follows:

(a) For a wall subjected to axial tension

$$I_e = 0.5\ I_g \qquad \text{(B-6)}$$

$$A_e = 0.5\ A_g \qquad \text{(B-7)}$$

(b) For a wall subjected to compression

$$I_e = 0.8\ I_g \qquad \text{(B-8)}$$

$$A_e = A_g \qquad \text{(B-9)}$$

(c) For diagonally reinforced coupling beams

$$I_e = 0.4\ I_g \qquad \text{(B-10)}$$

(d) For conventionally reinforced coupling beams or coupling slabs

$$I_e = 0.2\ I_g \qquad \text{(B-11)}$$

In the above expressions the subscripts "e" and "g" refer to the "equivalent" and "gross" properties respectively.

When only slabs connect adjacent shear walls, the equivalent width of slab to compute I_g may be taken as the width of the opening between the walls or 8 times the thickness of the slab, whichever is less.

For cantilever walls with aspect ratios, h_w/ℓ_w, larger than 4, the effect of shear deformations upon stiffness may normally be neglected. When a combination of "slender" and "squat" shear walls provide the seismic resistance, the latter may be allocated an

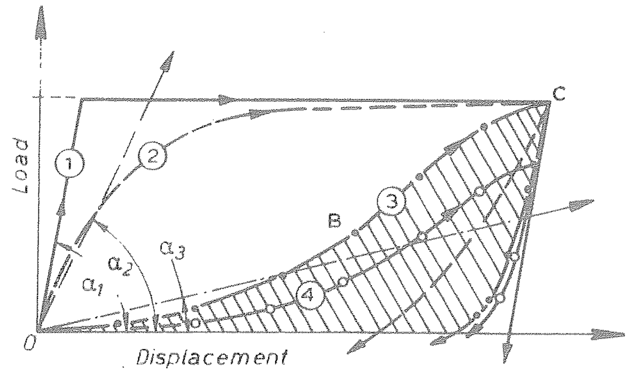

Fig. 4 - Load-Displacement Response for (1) Idealized, (2) Optimal, (3) (4) Repeated Shear Affected Conditions in Reinforced Concrete Members.

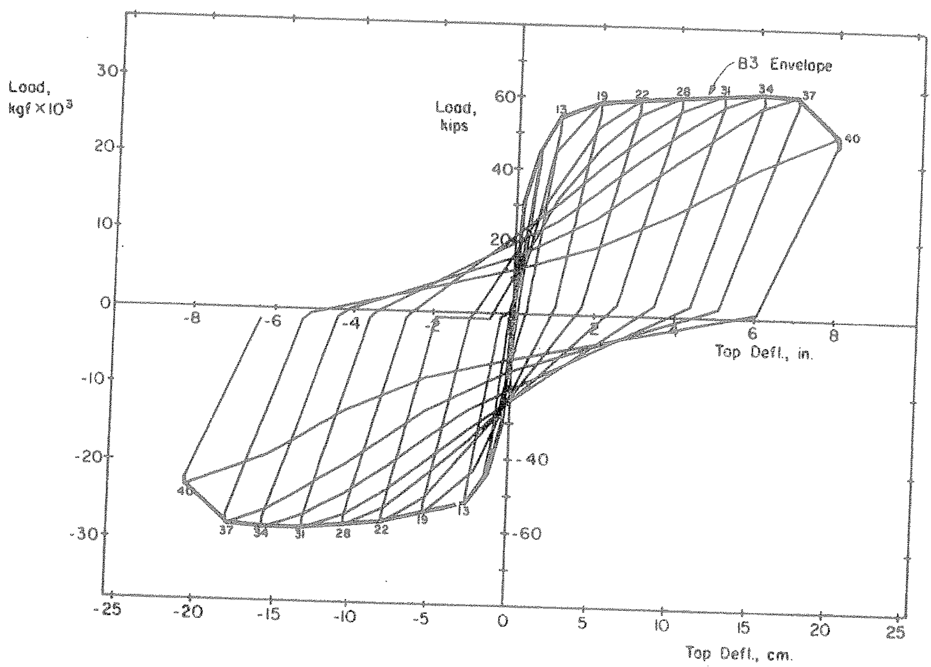

Fig. 5 - The Hysteretic Response of a Cantilever Shear Wall with Significant Shear Deformations (11)

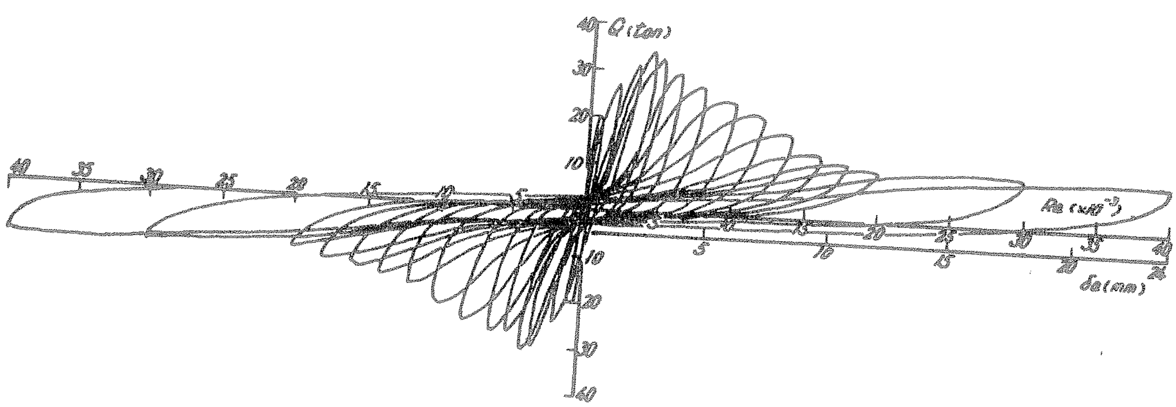

Fig. 6 - The Hysteretic Response of a Shear Wall in which Shear governs the strength.

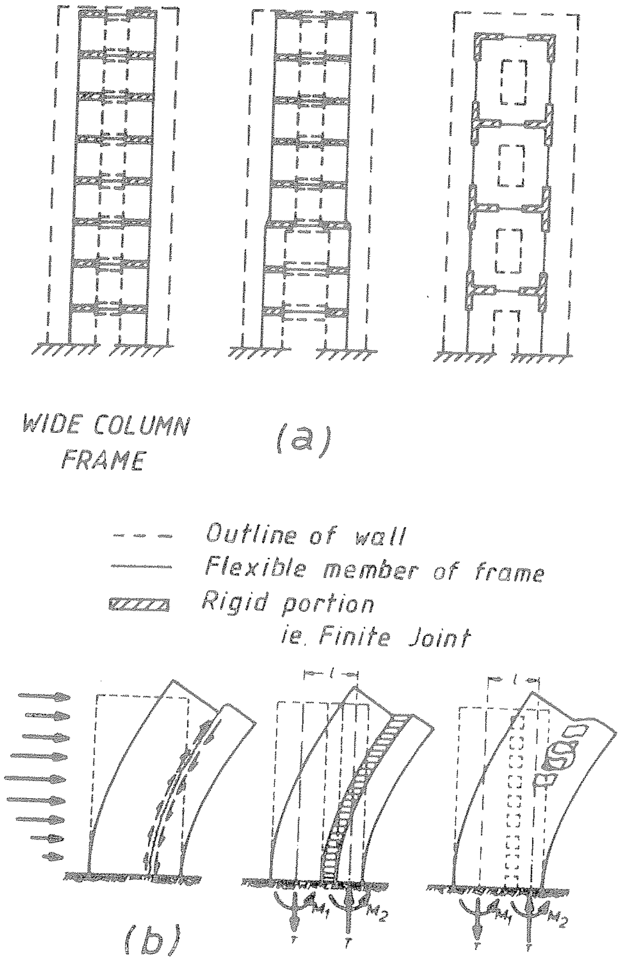

Fig. 7 The Modelling of Coupled Shear Walls for (a) Frame Analysis or (b) Laminar Analysis

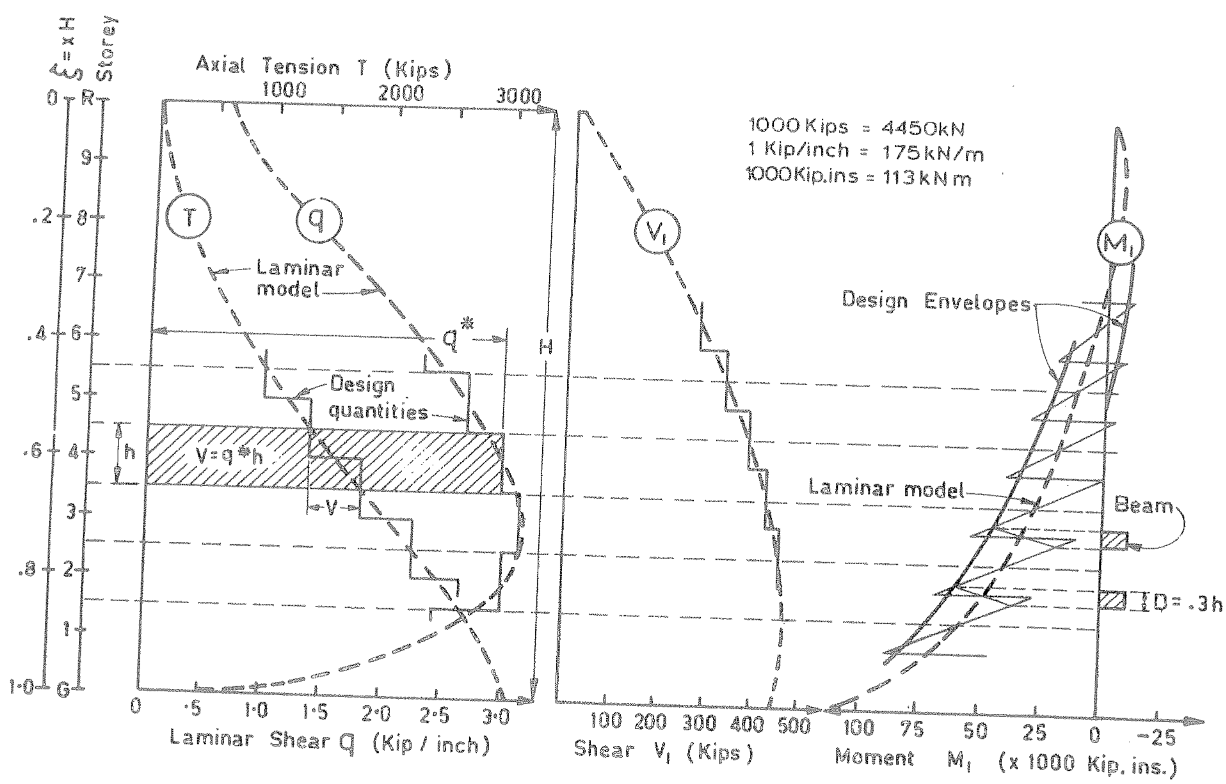

Fig. 8 - The Results of the Laminar Analysis of a Laterally Loaded Coupled Shear Wall Structure.

excessive proportion of the total load if shear distortions are not accounted for. For such cases, i.e. when $h_w/\ell_w < 4$, it may be assumed that

$$I_w = \frac{I_e}{1.2 + F} \qquad \text{(B-12)}$$

where

$$F = \frac{30\ I_e}{h_w^2 b_w \ell_w} \qquad \text{(B-13)}$$

A more accurate estimate of flexural deformations may be made if the ratio of the moment causing cracking to the maximum applied moment is evaluated and an improved value of I_e is used in Eqs. (B-12) and (B-13) thus

$$I_e = \left(\frac{M_{cr}}{M_a}\right)^3 I_g + \left(1 - \left(\frac{M_{cr}}{M_a}\right)^3\right) I_{cr} \qquad \text{(B-14)}$$

where b_w = web thickness of wall section

ℓ_w = horizontal length of wall

h_w = height of wall

M_{cr} = cracking moment according to Eq. (B-15)

M_a = maximum moment at which deflection is computed

I_{cr} = moment of inertia of cracked section transformed to concrete

$$M_{cr} = \frac{f_r\ I_g}{y_t} \qquad \text{(B-15)}$$

where f_r = the modulus of rupture of concrete = $0.62\sqrt{f'_c}$ MPa

y_t = distance from centroidal axis of gross section, neglecting the reinforcement, to extreme fibre in tension

f'_c = specified compressive strength of concrete, MPa

I_g = second moment of area of the gross concrete section

In Eq. (B-12) some allowance has also been made for shear distortions and deflections due to anchorage (pull-out) deformations at the base of a wall, and therefore these deformations do not need to be calculated separately.

Deflections due to code[1] - specified lateral static loading may be determined with the use of the above equivalent sectional properties. However, for consideration of separation of non-structural components and the checking of drift limitations the appropriate amplification factor given in the code[1], must be used.

Geometric modelling -

For cantilever shear walls it will be sufficient to assume that the sectional properties are concentrated in the vertical centre line of the wall. This should be taken to pass through the centroidal axis of the wall section, consisting of the gross concrete area.

When cantilever walls are interconnected at each floor by a slab it is normally sufficient to assume that the floor will act as a rigid diaphragm. Thereby the positions of walls relative to each other will remain the same during the lateral loading of the shear wall assembly. By neglecting wall shear deformations and those due to torsion and restrained warping of an open wall section, the lateral load analysis can be reduced to that of a set of cantilevers in which flexural distortions only will control the compatibility of deformations. Such analysis,based on first principles, can properly allow for the contribution of each wall when it is subjected to deformations due to floor translations or torsion[2]. It is to be remembered that such an elastic analysis, however approximate it might be, will satisfy the requirements of static equilibrium, and hence it will lead to a satisfactory distribution of internal actions among the walls of an inelastic structure.

When two or more walls in the same plane are interconnected by beams, as is the case in coupled shear walls shown in figure 17, it will be necessary to account for more rigid end-zones where beams frame into walls. Such structures should be modelled as shown in figure 7a. Standard programs written for frame analyses[6,7] may then be used. Alternatively coupled shear walls may be modelled by replacing the discrete coupling beams with a continuous set of elastic connecting laminae[2] as shown in figure 7b. The internal actions resulting from such an analysis can be readily converted into discrete moments, shear or axial forces that develop in each floor level. The results of such an analysis are shown in figure 8. The continuous curves for beam shear, moment and axial load on the walls result from the mathematical modelling used in figure 7b. The stepped lines in figure 8 show the conversion of these quantities into usable design actions.

The analysis of wall sections

Because of the variability of wall section shapes, design aids, such as axial load-moment interaction charts for rectangular column sections, cannot often be used. The designer will have to resort to the working out of the required flexural reinforcement from first principles. Programs can readily be developed for minicomputers to carry out the section design analysis. The manual section design usually consists of a number of successive approximation analyses of trial sections.

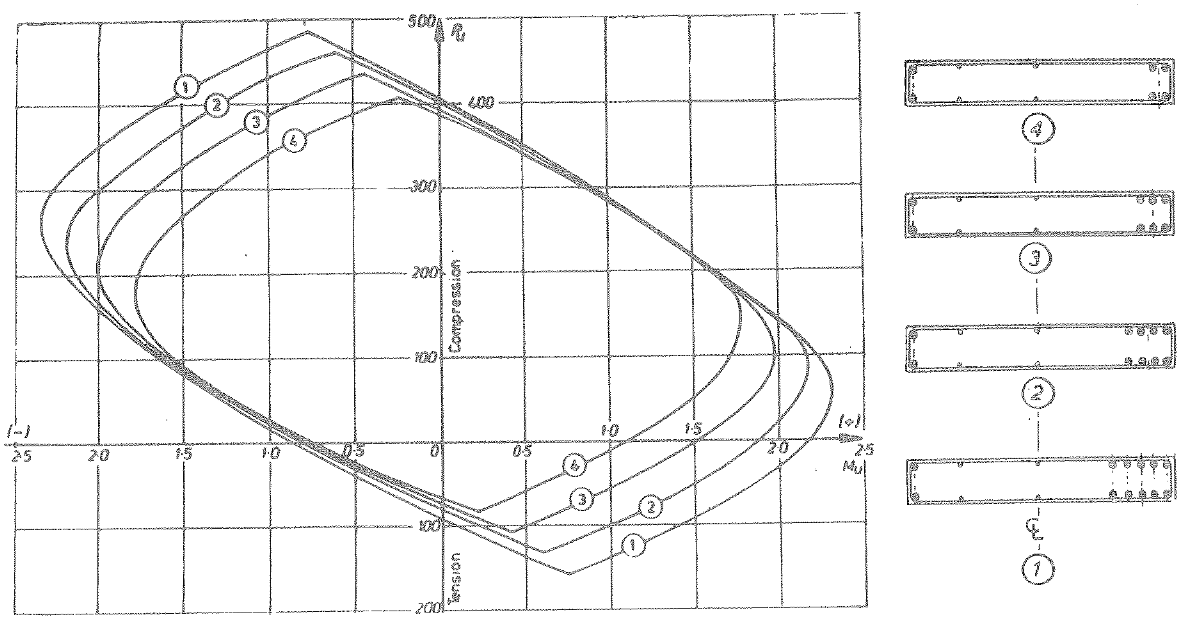

Fig. 9 - Axial Load-Moment Interaction Curves for an Unsymmetrical Shear Wall Section.

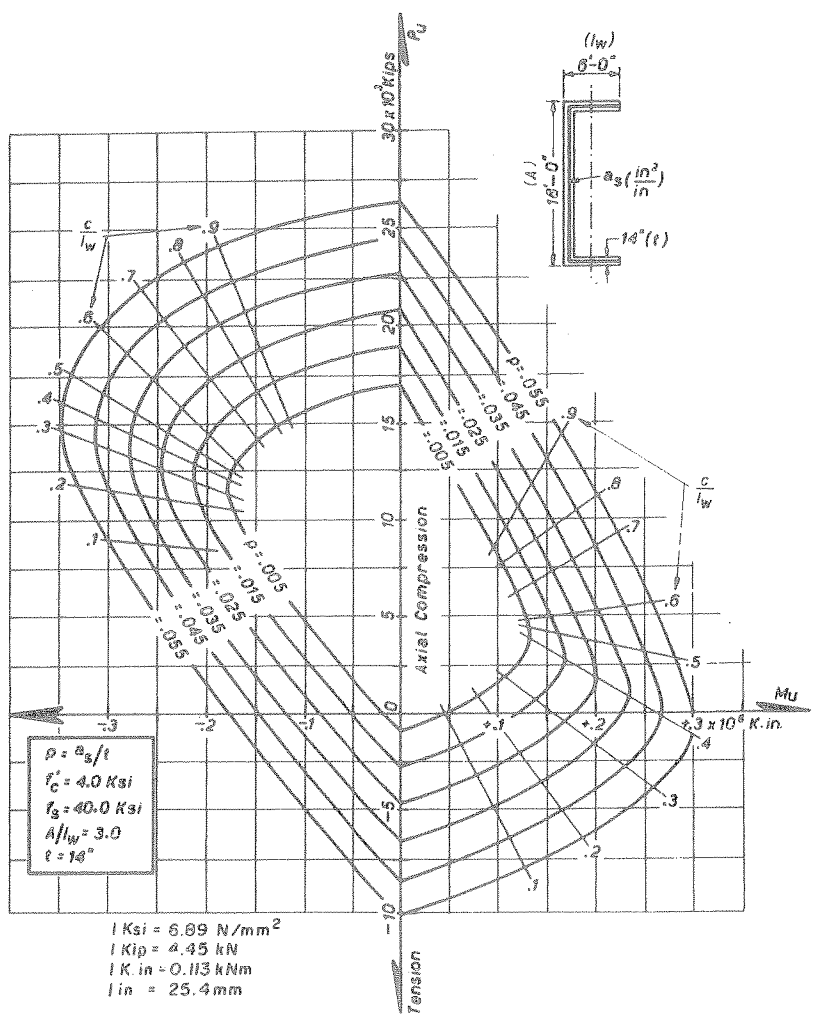

Fig. 10 - Axial Load-Moment Interaction Curves for a Channel Shaped Wall Section (2)

With a little experience convergence can be fast.

One of the difficulties that arises in the section analysis for flexural strength, with or without axial load, is the multi-layered arrangement of reinforcement. A very simple example of such a wall section is shown in figure 9. The four sections are intended to resist the design actions at four different levels of the structure. When the bending moment (assumed to be positive) causes tension at the more heavily reinforced right hand edge of the section, net axial tension is expected on the wall. On the other hand, when flexural tension is induced at the left hand edge of the section by (negative) moments, axial compression is induced in that wall. It is a typical loading situation in one wall of a coupled shear wall structure, such as shown in figure 7.

The moments are expressed with an eccentricity of the axial load, measured from the axis of the section, which, as stated earlier, is taken through the centroid of the gross concrete area rather than through that of the composite section. It is expedient to use the same reference axis also for the analysis of the cross section. It is evident that the plastic centroids in tension or compression do not coincide with the axis of the wall section. Consequently the maximum tension or compression strength of the section, involving uniform strain across the entire wall section, will result in axial forces that act eccentrically with respect to the axis of the wall. These points are shown in figure 9 by the peak values at the top and bottom meeting points of the four sets of curves. This representation enables the direct use of moments and forces, which have been derived from the analysis of the structural system, because in both analyses the same reference axis has been used.

Similar moment-axial load interaction relationships can be constructed for different shapes of wall cross section. An example for a channel shaped section is shown in figure 10. It is convenient to record in the analysis the neutral axis positions for various combinations of moments and axial forces, because these give direct indication of the curvature ductilities involved in developing the appropriate strengths, an aspect examined in "Limitations on Curvature Ductility".

Analyses for Equivalent Lateral Static Loads

The selection of load

The selection of the lateral static load, to determine the appropriate design actions which in turn lead to the desired strength, is in accordance with the earthquake provisions of the loadings code(1). Suitable structural type factors, S, which affect the total design base shear, have been suggested in "Types of Ductile Structural Walls" and elsewhere(3).

To determine the magnitude of the basic seismic coefficient the period of the structure is required. This in turn involves the estimation of the structural stiffness at a state when, due to high intensity elastic dynamic excitation, the reinforced concrete components have extensively cracked. A suggested procedure for estimating stiffnesses for this purpose is outlined in "Modelling of member properties".

With this information the intensity of the lateral design loading and its distribution over the height can be determined because all other parameters (such as importance and risk factors) are specified in the loadings code(1). Using the appropriate model, described in the previous section, the analysis to determine all internal design actions may then be carried out.

Redistribution of actions in the inelastic structure

Because the structure is expected to be fully plastic when it develops its required strength, a departure from the elastic distribution of actions in walls linked together is acceptable as long as the total strength of the system is not reduced. For example the elastic analysis for the prescribed load may have resulted in bending moment patterns in three identically distorted shear walls, as shown in figure 11 by the full line curves. It is seen that these are proportional to the stiffnesses that were defined in "Modelling of member properties". It may be desirable to allocate more load to wall 3 because, for example in the presence of more axial compression, it could resist more moment with less flexural reinforcement (see figure 9). As the dashed curves show the design moments for wall 1 and wall 2 have been reduced and those of wall 3 have been increased by the same amount, so that no change in the total moment of resistance occurs.

In order to ensure that there will be no significant difference in the ductility demands when all three walls are required to develop plastic hinges, it is recommended that moment redistribution between walls should not change the maximum value of the moment in any wall by more than 30%. This is seen to be satisfied in the example shown in figure 11. When such redistribution is used in the design of walls, the floor diaphragms should also be designed to be capable of transferring the corresponding forces to each wall.

Similar consideration suggests that, if necessary, the maximum shear force indicated by the elastic analysis in coupling beams of shear walls could be reduced by up to 20% provided that corresponding increases in the shear capacities of beams at other floors are made. With reference to figure 8, this would mean a reduction of the shear forces at and in the vicinity of the 3rd storey with appropriate increases in the lower and particularly upper storeys, so that the total area within curve "q" does not decrease.

These design quantities may then be used to proportion the wall sections so as to provide the required dependable strength in accordance with the Concrete Design Code[8].

Dynamic Analyses

For most buildings in which reasonable uniformity in layout and stiffness prevails over the height of the structure, the derivation of design quantities from an elastic analysis for the code[1] specified lateral static loading is likely to assure as satisfactory a seismic performance as a more sophisticated dynamic analysis. However, when abrupt changes, such as setbacks or other discontinuities, occur, the dynamic response may expose features which may not be adequately provided for f the static analysis is used. For such situations the spectral modal dynamic analysis is recommended[1,19]. The results need to be scaled and if necessary the static load analysis may be suitably adjusted to provide the desired design quantities.

For unuaual buildings or for special structures a time history dynamic analyses may be necessary. With the development of analysis programs[6,9], in which the cyclic response of plastic hinges can be modelled with a high degree of sophistication, it is now possible to predict the response of a building to a selected ground excitation. In this, moments, shear and axial forces as well as inelastic deformations, deflections, storey drifts etc. are evaluated at every time step during the specified earthquake record. Maxima encountered during the entire duration of the excitations are also recorded. It is an analysis and not a design tool, and for this reason it may be used to check the rformance of the structure as designed. In the definition of properties the probable strengths of the critical regions, discussed in "Probable Strength", should be used. The analysis may warrant certain changes to be made.

In the selection of earthquake records the designers should consider a representative excitation for the locality, which might test the design for its suitability in damage control. Such an analysis will reveal whether adequate stiffness has been provided. A viscous damping of 5% critical is suggested for such analyses.

Another study may be made for an earthquate record representing the largest credible excitation that would be expected in the locality during the probable life of the building. Thereby the inelastic deformations, such as plastic hinge rotations, and maximum actions, such as shear forces across inelastic regions of shear walls, can be predicted and hence compared with values that were envisaged in the design. For such a study a viscous damping of 8-10% of critical may be used.

Torsion

As in all structures in seismic areas, symmetry in structural layout should be aimed at. This will reduce torsional effect due to the noncoincidence of the centre of rigidity, CR, (centre of stiffness) and the centre of gravity, CG, (centre of mass). Typical eccentricities with respect to the two principal actions of design loading, e_x and e_y, are shown for a set of shear walls of an apartment building in figure 12. Deliberate eccentricities should be avoided, if possible, because uneven onset of plastification during large excitations may aggravate eccentricity and this in turn may lead to excessive ductility demand in lateral load resisting elements situation far away from the centre of rotation.

An example of the unintended inelastic response of two ductile shear walls is illustrated in figure 13a. Because the centre of the mass, CG, is approximately at the centre of the plan, approximately one half of the induced earthquake load, E, will have to be resisted by each of the end walls at A and B. It may be difficult to prevent Wall A from having a lateral load carrying capacity considerably in excess of that on Wall B. Hence energy dissipation due to inelastic deformation may well be restricted to Wall B only which, as a result of this, could be subjected to a displacement, Δ, much larger than expected. Irrespective of the relative stiffness or strength of the two shear walls, structures in which only two principal planes of lateral resistance exist parallel to either major axes, are likely to be torsionally unstable during large inelastic seismic excitations.

The structural layout shown in figure 13b is symmetrical with respect to the earthquake loading E. It is seen that any eccentricity introduced during the inelastic response of the two end walls will result in torsion which is readily restricted by three walls acting in the perpendicular direction. These walls are likely to remain elastic and hence they will ensure a uniform inelastic translation of each floor, thereby reducing the ductility demand on each of the end walls at A and B.

The example structure shown in figure 13b also shows that, in spite of considerable eccentricity, it is likely to be much more tolerant with respect to horizontal earthquake loading, H, in the other direction. The very significant torsional resistance of the two end walls, at A and B, can ensure that the other three walls will dissipate seismic energy because of approximately equal inelastic wall diaplacements in the direction of the excitation H. Figure 13b thus shows a desirable, torsionally stable structural layout in which the full utilization of walls in one direction of seismic actions

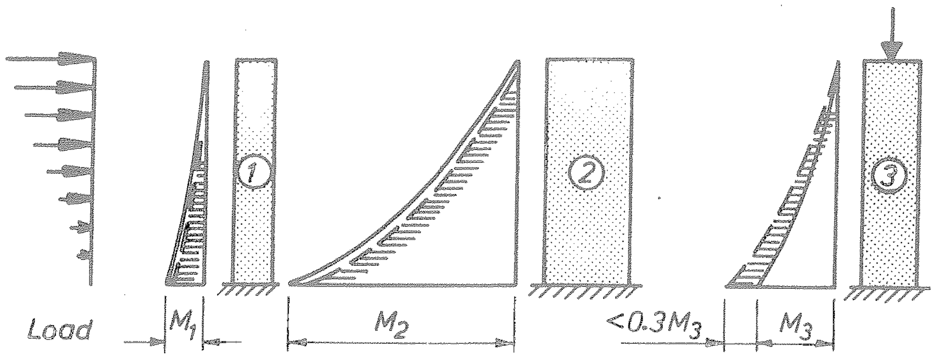

Fig. 11 - Load Redistribution between Three Inelastic Shear Walls.

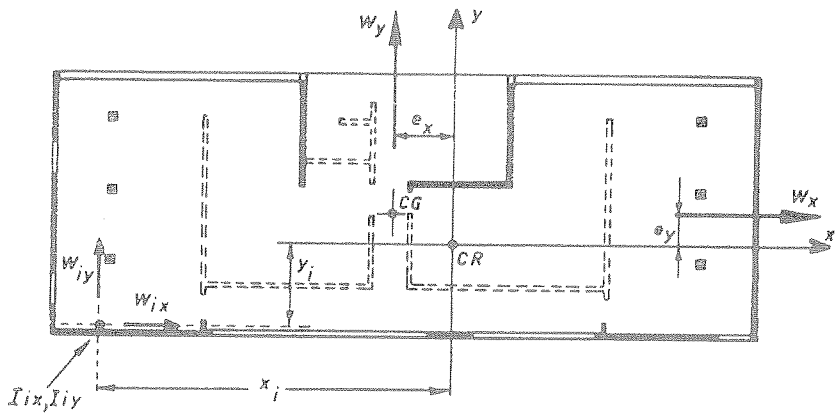

Fig. 12 - Shear Wall Layout for an Apartment Building (2)

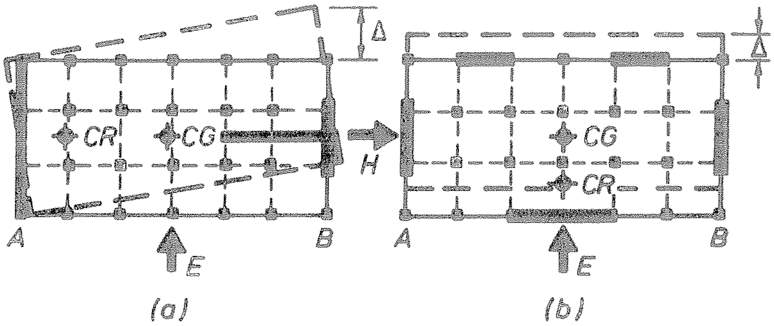

Fig. 13 - The layout of Shear Walls affects the Torsional Stability of the Lateral Load Resisting System.

is enhanced by (elastic) walls acting in the perpendicular direction by preventing inelastic storey twist.

Small single shear cores are particularly vulnerable to torsional instability.

CAPACITY DESIGN PROCEDURES:

The Definition of Strength

Before a hierarchy in the establishment of desirable energy dissipating mechanisms can be established, it is necessary to define the various strengths that might have to be quantified in the design. These have been studied in recent publications(2,8) and for this reason only a brief summary of the definitions and their relative values are given here.

Ideal strength

The ideal or nominal strength of a section is obtained from established theory preducting failure behaviour of the section, based on assumed section geometry, the actual reinforcement provided and specified material strengths, such as f_c' and f_y.

Dependable strength

To allow for the variations in strength properties and the nature and consequence of the failure, only a fraction of the ideal strength is relied upon to meet the load demand specified by the loadings code(1). Therefore strength reduction factors, ϕ, are introduced(8) to arrive at the dependable or reliable strength thus:

$$\text{Dependable Strength} = \phi \text{ Ideal Strength}$$

Probable strength

Routine testing of materials or components indicates the probable strength attainable by prototype components in the structure. The designer will seldom require this information. However, when the likely dynamic response of a shear wall structure during a selected ground excitation is to be studied analytically, as discussed in "Dynamic Analyses", it is more appropriate to consider the probable properties of materials at critical member sections.

Overstrength

The overstrength takes into account all the possible factors that may cause a strength increase above the ideal strength. These include steel strength higher than the specified yield strength and the additional strength due to strain hardening at large deformations, concrete strength higher than specified, section sizes larger than assumed in the initial design, increased axial compression strength in flexural members due to lateral confinement of the concrete, and participation of additional reinforcement such as that placed nearby for construction purposes.

Relationship between strengths

When using Grade 275 flexural reinforcement made in New Zealand the following relationships, based on the actual reinforcement provided, may be used to determine the flexural strengths of members -

(i) Dependable Strength = 0.90 Ideal Strength

(ii) Probable Strength = 1.15 Ideal Strength

(iii) Overstrength = 1.25 Ideal Strength

(iv) Overstrength = 1.39 Dependable Strength

(v) Probable Strength = 0.90 Overstrength

(vi) Probable Strength - 1.28 Dependable Strength

It is preferable, however, to determine these values from measured properties of the steel to be used.

It is recommended that wherever design actions, such as shear forces across shear walls, are derived from the flexural overstrength of the wall, the ideal strength be considered to be sufficient to resist it. Whereas in strength design the actions derived from factored loads, such as moment, M_u, or shear, V_u, need to be equal or smaller than the corresponding dependable strength provided, such as ϕM_i or ϕV_i, where M_i and V_i refer to ideal strengths of a section, in capacity design the criteria should be met:

$$M^o \leqslant M_i \quad \text{or} \quad V^o \leqslant V_i \qquad \text{(B-16)}$$

where M^o and V^o are the design actions at a particular section derived from capacity design procedures.

Cantilever Walls

The determination of the flexural and shear load on cantilever walls, taking into account moment redistribution as outlined in "Redistribution of actions in the inelastic structure", is a simple procedure.

The consideration of flexure and overstrength

When the appropriately factored gravity forces are also considered the required flexural reinforcement can be readily determined from the principles reviewed in 'The analysis of wall sections'. In this the designer should attempt to provide the minimum flexural reinforcement to just satisfy the dependable moment demand at the wall base. Apart from economy it should be the designer's aim to keep the overstrength of the wall to the minimum, otherwise demands for shear resistance and on the foundations might be unnecessarily compounded. In very lightly loaded walls, minimum requirement for wall reinforcement may override this criterion. The flexural overstrength is expressed by the "overstrength factor", ϕ_o, which is defined as follows:

$$\phi_o = \frac{\text{overstrength moment of resistance}}{\text{moment resulting from code loading}} = \frac{M^o}{M_{code}} \qquad \text{(B-17)}$$

where both moments refer to the base section of the wall.

Even though in most walls Grade 380 reinforcement will be used, the flexural overstrength at the base may be assumed to be only 1.25 times the ideal flexural strength of that section. The reason for this is that cantilever walls will seldom be required to develop plastic hinge rotations involving excessive strain hardening of the tensile reinforcement. However, if wall configuration, slenderness or load demand indicate that tensile strains in excess of 10 times yield strain may be involved with Grade 380 reinforcement, it should be assumed that $\phi_o = 1.6$. It should also be appreciated that in compression dominated wall sections the flexural resistance will be significantly larger if the concrete strength at the time of the earthquake is much in excess of the specified value f'_c.

Moment design envelopes

Once the flexural overstrength of a cantilever wall is determined at its base, it is necessary to define the reduction of moment demand at upper floors.

This used to be done by utilising the bending moment diagram. It is to be recognized, however, that the moment envelope that would be obtained from a dynamic analysis is quite different from the bending moment diagram drawn for the specified lateral static load. This has been identified from modal spectral analyses(12) as well as from time history dynamic studies(13). Typical bending moment envelopes for 20 storey cantilever shear walls with different base yield moment capacities, subjected to a particular ground excitation, are shown in figure 14. It is seen that there is an approximate linear variation of moment demand during dynamic excitations.

If the flexural reinforcement in a cantilever wall were to be curtailed according to the bending moment diagram, then flexural yielding (plastic hinges) could occur anywhere along the height of the building. This would be undesirable because potential plastic hinges do require special detailing, and hence more transverse reinforcement. Moreover, flexural yielding reduces the potential shear resisting mechanisms, and this again would require additional (horizontal) shear reinforcement at all levels where hinging might occur. This is discussed in "Control of Diagonal tension and compression".

For the reasons enumerated above it is recommended that the flexural reinforcement in a cantilever wall be curtailed so as to give a linear variation of moment of resistance. The recommendation is illustrated in figure 15. The linear envelope, shown by the dashed line, should be displaced by a distance equal to the horizontal length of the wall, ℓ_w. This allows for the fact that due to shear the internal flexural tension in a beam section at a section is larger than the bending moment at that section would indicate(2,8). Accordingly the design envelope, indicating the minimum ideal moment of resistance to be provided, is obtained. Vertical flexural bars in the cantilever wall, to be curtailed must extend beyond the section indicated by the design envelope of figure 15, by at least the development length for such bar(8).

Flexural ductility of cantilever walls

To ensure that a cantilever wall can sustain a substantial portion of the intended lateral load at a given displacement ductility ratio, μ_Δ, it is necessary that it can develop in its plastic hinge at the base a certain curvature ductility ratio, μ_ϕ. These ductility ratios are traditionally defined as follows:

Displacement ductility ratio:

$$\mu_\Delta = \frac{\Delta_u}{\Delta_y} \qquad \text{(B-18)}$$

Curvature ductility ratio:

$$\mu_\phi = \frac{\phi_u}{\phi_y} \qquad \text{(B-19)}$$

where Δ_u and Δ_y are the deflections at the top of the cantilever at the ultimate state and at the onset of yielding and ϕ_u and ϕ_y are the corresponding curvatures i.e. rotations of the section, at the base of the cantilever.

The relationship between the curvature ductility of the base section and the displacement ductility of the wall will depend on the length of the plastic hinge at the base(2) and the wall height to horizontal length ratio, h_w/ℓ_w. The variation of curvature ductility demand with h_w/ℓ_w for various displacement demands is shown in figure 16. The dark bands represent the limits for the length of the plastic hinge, as obtained from two different proposed equations(14). It is seen that for slender cantilever walls which are expected to be subjected to a displacement ductility demand of four, very considerable curvature ductility will need to be developed at the base. This will need to be taken into consideration when the detailing of the potential plastic hinge zone is being undertaken. (See "Satisfying Ductility Demands").

Shear strength of cantilever walls

It was emphasized in the previous sections that if a shear failure is to be avoided, the shear strength of a wall must be in excess of the maximum likely shear demand. Therefore the shear strength must be at lease equal to the shear associated with the flexural overstrength of the wall i.e. $V_{min} \geqslant \phi_o V_{code}$.

It has been demonstrated that during the inelastic dynamic response of a shear wall, with a given base hinge moment capacity,

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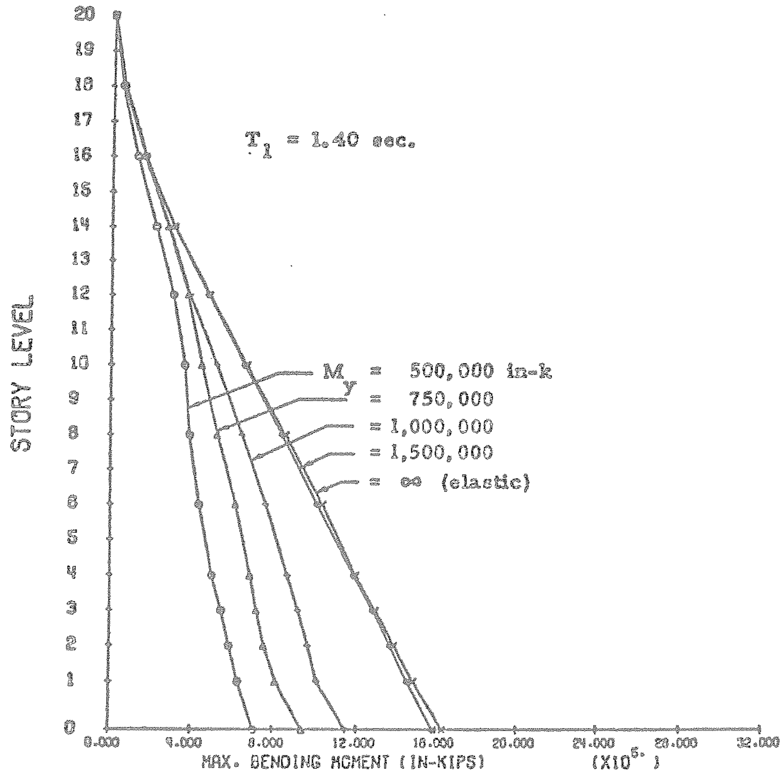

Fig. 14 - Dynamic Bending Moment Envelopes for a 20 Storey Shear Wall with different Base Yield Moment Capacities (13).

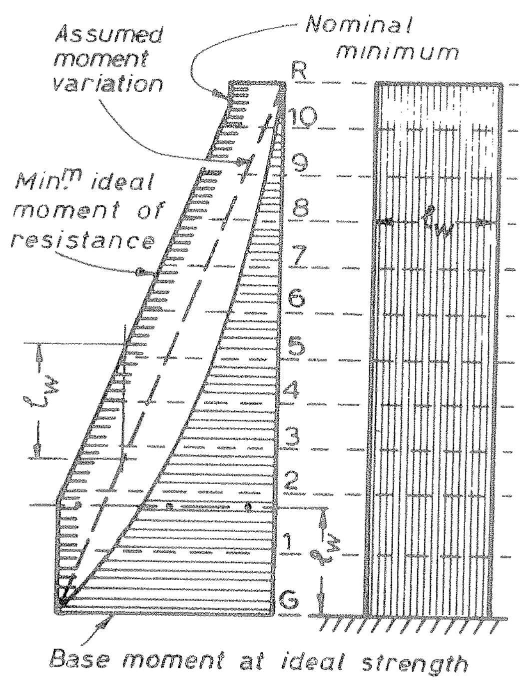

Fig. 15 - Recommended Design Bending Moment Envelope for Cantilever Shear Walls.

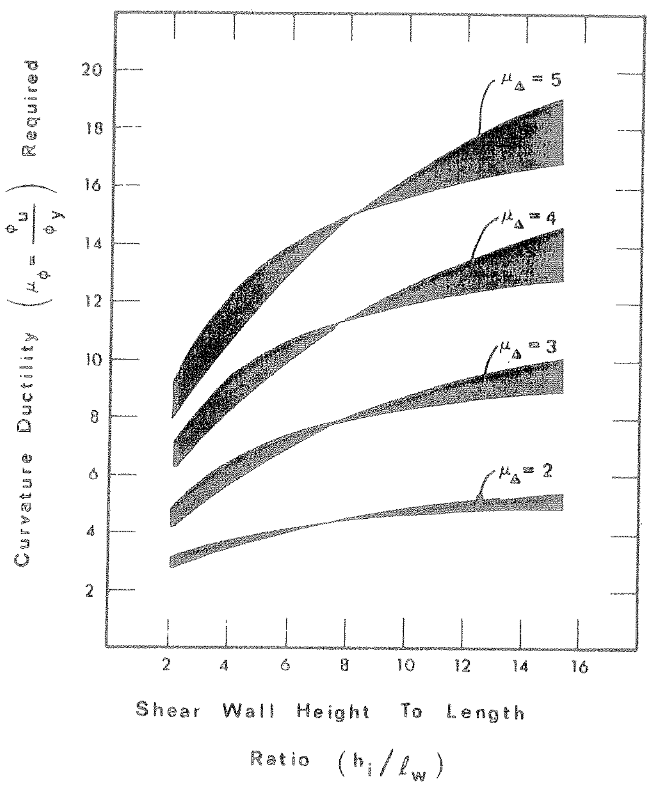

Fig. 16 - The Variation of Curvature Ductility at the base of Cantilever Shear Walls with the Aspect Ratio of the Walls and the Imposed Displacement Ductility Demand (14).

considerably larger shear forces can be generated than those predicted by static analysis[12]. For this reason the design shear forces must be magnified further. Therefore cantilever shear walls at all levels should possess an ideal shear capacity, V_i, of not less than

$$V_{wall} = \omega_v \phi_o V_{code} \qquad (B\text{-}20)$$

where V_{code} is the shear demand derived from code[1] loading, ϕ_o was defined by Eq. (B-17) and the dynamic shear magnification factor is given by Eq. (B-21) for buildings up to 5 storeys high

$$\omega_v = 0.1N + 0.9 \qquad (B\text{-}21)$$

where N is the number of storeys. For walls taller than 5 storeys the value of ω_v is given in Table B-I[12]. However, the ideal shear strength need not exceed

$$V_{wall} < (4/S)\ V_{code} \qquad (B\text{-}22)$$

TABLE B-1

DYNAMIC SHEAR MAGNIFICATION FACTOR ω_v

Number of Storeys	ω_v
1 to 5	Eq.(B-21)
6 to 9	1.5
10 to 14	1.7
15 and over	1.8

It may be that the flexural capacity provided at the base of the structure is so large that inelastic response of the shear wall will become unlikely. For such situations Eq. (B-22) sets an upper limit whereby the product $\omega_v\phi_o$ need not be larger than 4/S. For example a single 8 storey cantilever shear wall need not possess an ideal shear strength in excess of 4/1.2 = 3.33 times the code specified shear load, V_{code}.

The provisions to meet the design shear load V_{wall} from Eq. (B-20) are given in "Control of shear failure".

Coupled Shear Walls

In the following sections a recommended step by step capacity design procedure for coupled shear walls is outlined. When necessary reference should be made to figure 7 or figure 17.

Geometric review

Before the static analysis procedure commences the geometry of the structure should be reviewed to ensure that in the critical zones compact sections, suitable for energy dissipation, will result. Section configurations should satisfy criteria outlined in "Stability".

Lateral static load

The appropriate lateral static load, in accordance with the loadings code[1] is to be determined. To do this it might be necessary to estimate the probable value, S_p, of the structural type factor S, recommended in "Types of Ductile Structural Walls"(b).

Elastic analysis

With the evaluation of the lateral static load the complete analysis for the resulting internal structural actions, such as moments, forces etc. can be carried out. In this the modelling assumption of "Modelling Assumptions" should be observed. Typical results are shown in figure 8.

Confirmation of the structural type factor

Having obtained the moments and axial forces at the base of the structure the moment parameter

$$A = \frac{T\ell}{M_o} \qquad (B\text{-}1)$$

as discussed in "Types of Ductile Structural Walls"(b), can be determined. The significance of the parameter may also be seen in figure 18. With the use Eq. (B-3) the exact required value of the structural type factor, S can be found. If this differs from that assumed earlier i.e. S_p, all quantities of the elastic analysis are simply adjusted by the multiplier S/S_p.

Checking of foundation loads

To avoid unnecessary design computations, at this stage it should be checked whether the foundation structure for the coupled shear walls would be capable of transmitting at least 1.5 times the overturning moment, M_o, received from the superstructure (see figure 17), to the foundation material (soil). It is to be remembered that in a carefully designed superstructure, in which no excess strength of any kind has been allowed to develop, 1.4 times the overturning moment resulting from code loading M_o will be mobilized during large inelastic displacements. (See "Relationship between strengths"). Hence the foundation system must have a potential strength in excess of 1.4 M_o, otherwise the intended energy dissipation in the superstructure may not develop.[5]

Design of coupling beams

Taking flexure and shear into account the coupling beams at each floor can be designed. Normally diagonal bars in cages[2] should be used, preferably with Grade 275 reinforcement. A strength reduction factor of $\phi = 0.9$ is appropriate. Particular attention should be given to the anchorage of caged groups of bars and to ties which should prevent inelastic buckling of individual diagonal bars. (See "Detailing of Coupling Beams"). The beam reinforcement should match as closely as possible the load demand. Excessive coupling beam strength may lead to subsequent difficulties in the design of walls and foundations.

Determination of actions on the walls

In order to find the necessary vertical reinforcement in each of the coupled walls (figure 17) at the critical base section, the following loading cases should be considered:

i) $P_e = P_{eq} - 0.9P_D$ axial tension (or small compression) and M_1

ii) $P_e = P_{eq} + P_D + P_{L_R}$ axial compression and M_2

where P_e = axial design load including earthquake effects

P_{eq} axial tension or compression induced in the wall by the lateral static loading

P_D = axial compression due to dead load

P_{L_R} = axial compression due to reduced live load L_R

M_1 = moment at the base developed concurrently with earthquake induced axial tension load (figure 17c)

M_2 = moment at the base developed concurrently with earthquake induced axial compression load (figure 17c)

iii) If case (i) above is found to result in large demand for tension reinforcement or for other reasons, a redistribution of the design moments from the tension wall to the compression wall may be carried out in accordance with 'Redistribution of actions in the inelastic structure', within the following limits:

(a) $M_1' > 0.7\ M_1$

(b) $M_2' = M_2 + M_1 - M_1' < 1.3\ M_2$

where M_1' and M_2' are the design moments for the tension and compression walls respectively, after the moment redistribution has been carried out.

In the above three steps, which would complete the strength design of the structure, a capacity reduction factor of $\phi = 0.9$ may be used for all cases. The justification for this is considered to result from a subsequent requirement, according to which compression dominated wall sections specifically need to be confined to ensure sufficient curvature ductility.

Using these quantities the vertical flexural reinforcement for each wall, with Grade 275 or Grade 380 steel, can now be determined in accordance with "The analysis of wall sections".

Overcapacity of coupling beams

In order to ensure that the shear strength of the coupled shear wall structure will not be exceeded and that the maximum load demand on the foundation is properly assessed, i.e. to fulfill the intent of "Hierarchy in Energy Dissipation", the overstrength of the potential plastic regions must be estimated. Accordingly the shear overcapacity, Q_i^o, of each coupling beam, as detailed, based on a yield strength of the diagonal reinforcement of $1.25\ f_y \geqslant 345$ MPa is determined. Where slabs, framing into coupling beams, contain reinforcement parallel to the coupling beams which is significant when compared with the reinforcement provided within the beam only, the possible contribution of some of this the reinforcement to the shear capacity of coupling beams should also be considered in computing overstrength.

Earthquake induced axial loads

The maximum feasible axial load induced in one of the coupled walls would be obtained from the summation of all the coupling beam shear forces at overcapacity, Q_i^o, applied to the wall above the section that is considered. For structures with several storeys this may be an unnecessarily conservative estimate, and accordingly it is recommended that the wall axial load at overstrength be estimated with

$$P_{eq}^{\ o} = (1 - \frac{n}{80}) \sum_i^n Q_i^o \qquad \text{(B-23)}$$

where n = number of floors above level i. The value of n in Eq. (B-23) should not be taken larger than 20.

The flexural overcapacity of the entire structure

In order to estimate the maximum likely overturning moment that could be developed in the fully plastic mechanism of the coupled shear wall structure, it is necessary to assume gravity loads that are realistic and consistent with such a seismic event. Accordingly, for this purpose only, the total overstrength axial loads to be sustained by the walls should be estimated as follows:

i) For tension of minimum compression

$$P_1^o = P_{eq}^{\ o} - P_D$$

ii) For compression

$$P_2^o = P_{eq}^{\ o} + P_D$$

It is now possible to estimate the flexural overstrength capacity of each wall section, as detailed, that may be developed concurrently with the above axial forces. The moments of resistance, which may be based on material strengths defined by $1.25f_y$ and $1.25f_c'$, so derived for the tension and compression walls respectively, are M_1^o and M_2^o. In similarity to Eq. (B-17) the overstrength factor for the entire coupled shear wall structure may be obtained from

$$\phi_o = \frac{M_1^o + M_2^o + P_{eq}^{\ o}\ \ell}{M_o} \qquad \text{(B-24)}$$

In accordance with the assumed strength properties of "Relationship between strengths" the value of ϕ_o so obtained should not be less than 1.39. If it is, the design should be checked for the error.

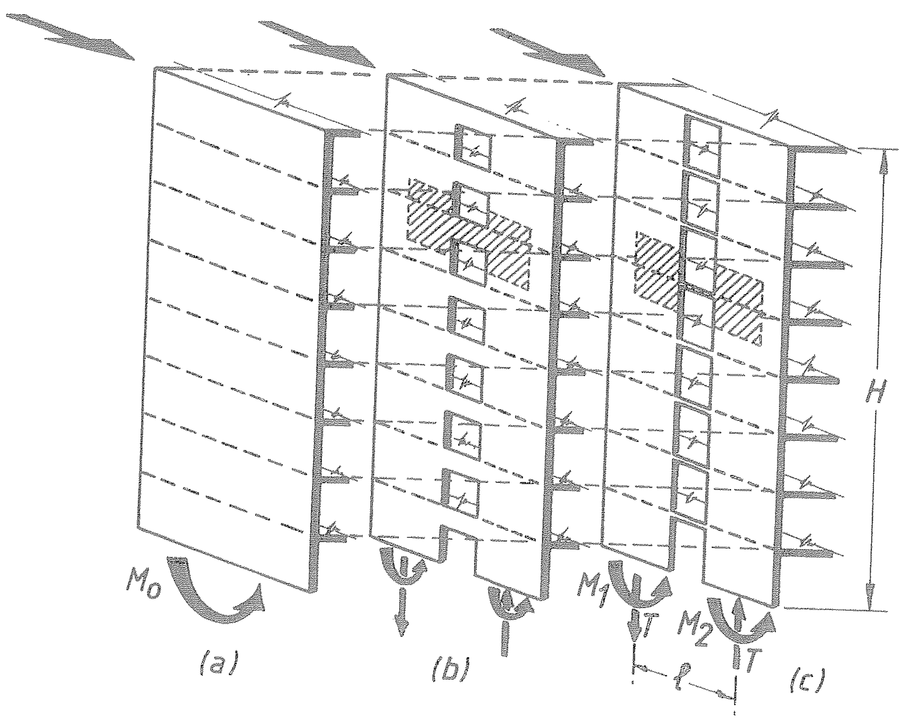

Fig. 17 - A Comparison of Ductil Walls (a) A Cantilever Wall (b) Wall Coupled by Strong Coupled Beam (c) Walls Coupled by Slabs Only

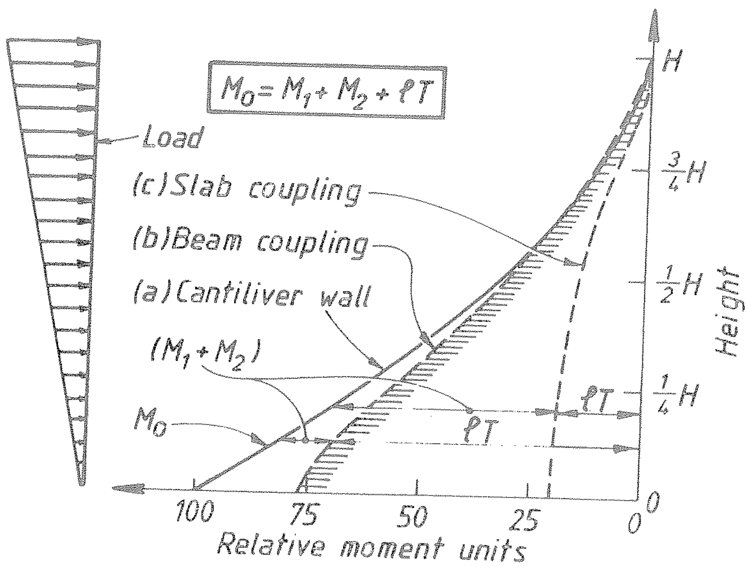

Fig. 18 - Contribution of Internal Coupling to the Resistance of Overturning Moments in Coupled Shear Walls.

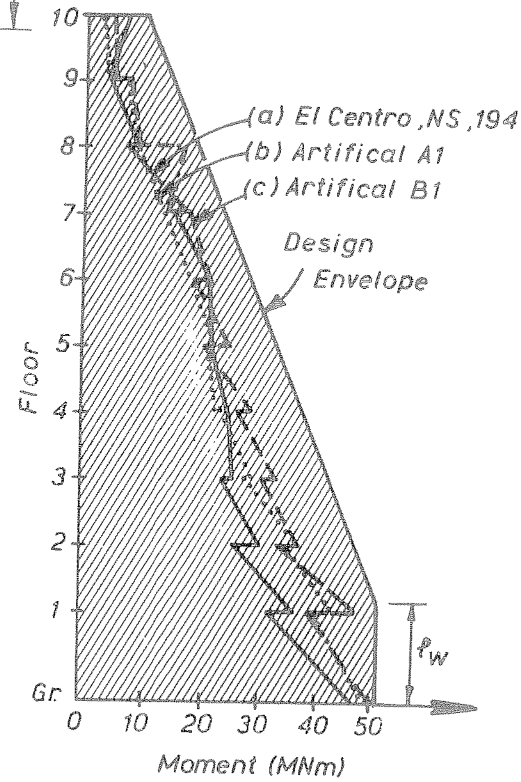

Fig. 19 - Bending Moment Envelopes for Coupled Shear Walls (a) Envelope Used in Design (b) Envelopes Observed in a Theoretical Study (15).

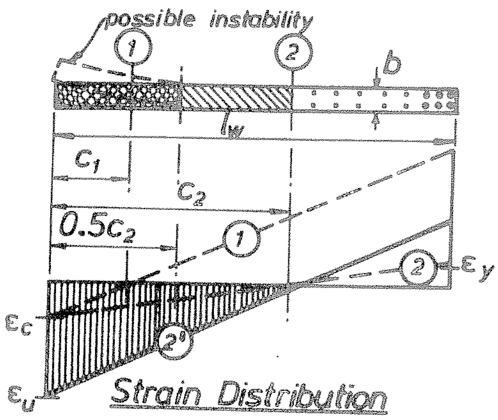

Fig. 20 - Strain Patterns for a Rect-angula Wall Section Subjected to Flexure and Axial Load.

Wall shear forces

In similarity to the approach employed in the section "Shear strength of cantilever walls" for cantilever shear walls, the maximum shear force for one wall of a coupled shear wall structure may be obtained from

$$V_{i,wall} = \omega_v \phi_o \left(\frac{M_i^o}{M_1^o + M_2^o} \right) V_{code} \; ; \; i = 1,\ 2 \qquad \text{(B-25)}$$

where ω_v = dynamic shear magnification factor in accordance with Eq. (B-20)

V_{code} = shear force on the entire shear wall structure at any level, derived by the initial elastic analysis for code loading[1] with the appropriate S factor.

$\omega_v \; \phi_o \leq 4/S$ in accordance with Eq. (B-22)

The bracketed term in Eq. (B-25) makes an approximate allowance for the distribution of shear forces between the two walls, which, at the development of overstrength, is likely to be different from that established with the initial elastic analysis. It also takes into account the approximate redistribution of shear forces that may have resulted from the deliberate redistribution of design moments from the tension to the compression wall.

The required horizontal shear reinforcement may be determined now. In assessing the contribution of the concrete shear resisting mechanism, the effects of the axial forces P_1^o and P_2^o, as appropriate should be taken into account.

Confinement of wall sections

From the load combinations considered above the positions of the neutral axes relative to the compressed edges of the wall sections are readily obtained. From the regions of the wall section over which, in accordance with the section "Confinement of Wall Regions" anti-buckling and/or confining transverse reinforcement is required, this reinforcement can now be determined.

Curtailment of vertical flexural reinforcement

For the purpose of establishing the curtailment of the principal vertical wall reinforcement, a linear bending moment envelope along the height of each wall should be assumed, as shown in figure 19a. This is intended to ensure that the likelihood of flexural yielding due to higher mode dynamic responses along the height of the wall is minimized. Details for the justification of such an envelope were examined in the section "Moment design envelopes". In a study, in which the inelastic dynamic response of a coupled shear wall was computed, the moment envelopes for responses to three different ground excitations, shown in figure 19, were obtained[15].

Foundation design

The actions at the development of the overstrength of the superstructure, P_1^o, P_2^o, M_1^o, M_2^o and wall shear forces V_1 and V_2, should be used as loading on the foundations. For ductile coupled shear walls, the foundation structure should be capable of absorbing these actions at its ideal strength capacity.

SATISFYING DUCTILITY DEMANDS

Stability

When part of a thin wall section is subjected to large compression strains, the danger of premature failure by instability arises. This is the case when a large neutral axis depth is required in the plastic hinge zone of the wall, as shown in figure 20, and the length of the plastic hinge is large i.e. one storey high or more. The problem is compounded when cyclic inelastic deformations occur. Instability should not be permitted to govern strength of ductile shear walls.

In the absence of information on the "compactness" of reinforced concrete wall sections, existing code rules[16], relevant to short columns, are best considered. For such columns the effective height to width ratio, ℓ_n/b, should not exceed 10[16].

The relevance of such a code requirement to a shear wall may be studied with the aid of figure 20. For a certain load combination the computed neutral axis depth may be c_2, so that a considerable portion of the wall section will be subject to compression. Near the extreme compression fibre, where, in accordance with accepted assumptions, the concrete strain at ideal flexural capacity is taken as $\varepsilon_c = 0.003$, instability may occur unless this strain pattern is restricted vertically to a very short plastic hinge length. Moreover, the strain profile marked (2) in figure 20 shows that very limited curvature ductility would be available at the attainment of the ideal strength of the section. To satisfy the intended displacement ductility demand for the shear wall system, a strain profile shown by line (2') may need to be developed. Such large concrete compression strains, ε_u, could only develop if the concrete in this zone is confined, and this will be examined in a later section. The phenomenon is fortunately rare, but it emphasizes the need for considering instability. It occurs more commonly when a wall has a large tension flange, such as shown in figure 22 and figure 35.

In the absence of experimental evidence intuitive judgement was used to recommend that, with the exceptions to be

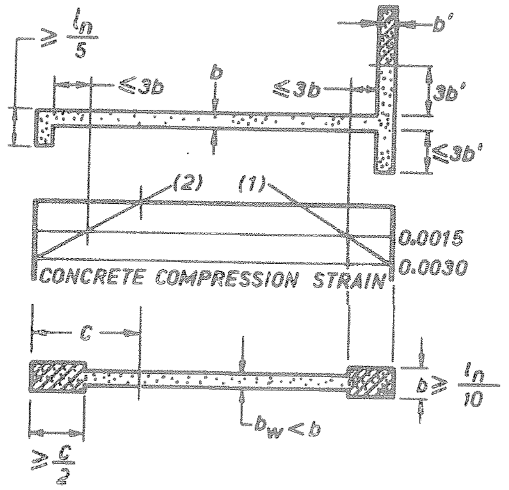

Fig. 21 - Parts of a Wall Section to be Considered for Instability and which Provides Lateral Support.

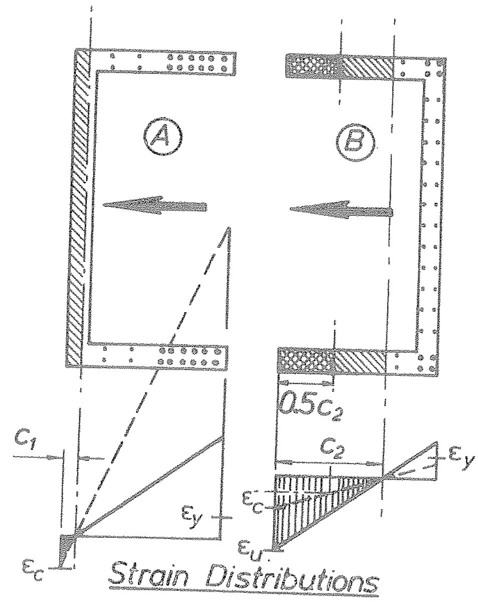

Fig. 22 - Strain Profiles for Channel Shaped Wall Sections.

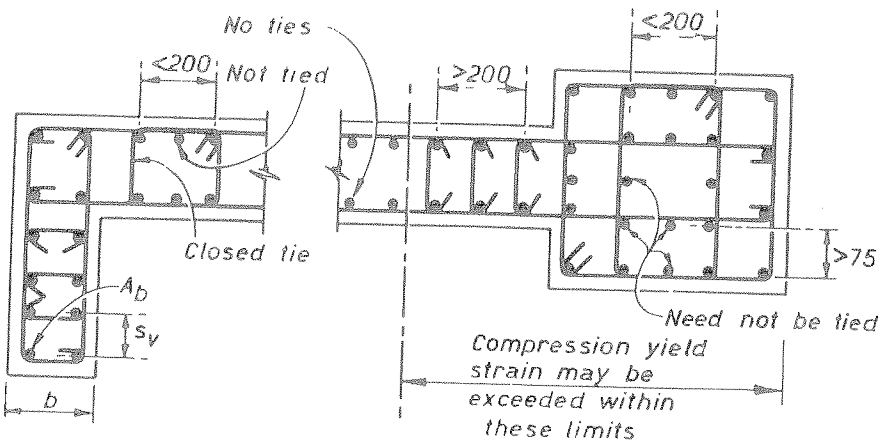

Fig. 23 - Transverse Reinforcement in Potential Yield Zones of Shear Wall Sections.

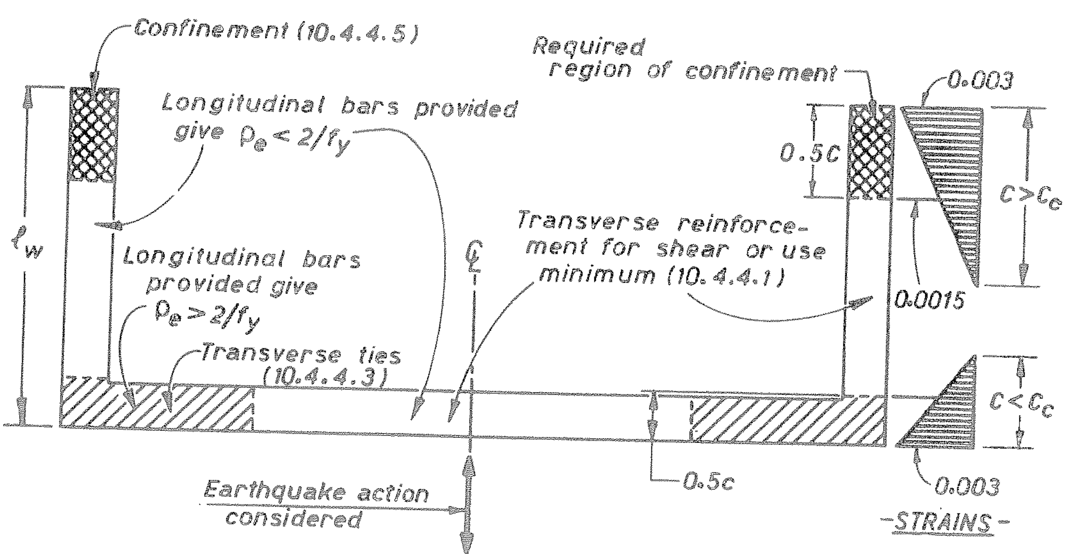

Fig. 24 - Regions of Different Transverse Reinforcement in a Shear Wall Section.

set out subsequently, in the outer half of the conventionally computed compression zone, the wall thickness b should not be less than one tenth of the clear vertical distance between floors or other effective lines of lateral support, ℓ_n. Considering the strain pettern (2) in figure 20, this zone extends over s distance of $0.5c_2$, as shown with cross shading. This is an area over which the concrete compression strain will exceed 0.0015 when the strain in the extreme compression fibre of the section, consistant with the determination of the ideal flexural strength, attains its assumed maximum value of 0.003.

When the computed neutral axis depth is small, as shown by the strain distribution (1) in figure 20, the compressed area may be so small that adjacent parts of the wall will stabilize it. Accordingly, when the fibre of 0.0015 compression strain is within a distance of the lesser of 2b or 0.15 ℓ_w from the compressed edge, the $b \geq \ell_n/10$ limit should not need to be complied with. In terms of neutral axis depth this criteria is met when $c \leqslant 4b_w$ or $c \leqslant 0.3\ \ell_w$, whichever is less. The strain profile (1), which occurs commonly in lightly reinforced walls with small gravity load, clearly satisfies this condition.

It may be assumed that only in buildings 3 storeys or higher would the plastic hinge length at the base, extending toward the first floor, be large enough to warrant an examination of instability criteria.

Certain components of walls, such as shown in figure 21, provide continuous lateral support to adjacent compressed elements. Therefore it is considered that any part of a wall, subjected to computed strains larger than 0.0015, which is within a distance of 3b of such a line of support, should be exempted from slenderness limitation. Figure 2 shows a number of locations that are exempt. The shaded part of the flange is considered to be too remote to be effectively restrained by the web portion of the wall and hence it should comply with the $b > \ell_n/10$ slenderness limitation. In the absence of a flange, the width of which is at least $\ell_n/5$, a boundary element may be formed that satisfies the slenderness limit. These latter two cases are also shown in figure 21.

Limitations on Curvature Ductility

By simple limitations of the amount of flexural tension reinforcement[8] in beam sections, it can be ensured that adequate curvature ductility, to meet the intents of seismic design, will be available. Because of the variety of cross sectional shapes and arrangements of reinforcements that can be used, and the presence of some axial load, the availability of ductility in shear walls cannot be checked by the simple process that is used for rectangular beams or sections.

In the analysis of wall sections for flexure and axial load, the neutral axis depth, c, is always determined. Hence the ratio of c/ℓ_w, an indicator of the curvature ductility required at the development of the ideal strength, (figure 21) can be readily found. Various strain profiles, associated with a maximum assumed concrete compression strain of $\varepsilon_c = 0.003$ are shown by dashed lines in figures 20 and 22. It is seen that different neutral axis depths, c_1 and c_2, for different wall configurations can give very different curvature ductilities.

The curvature ductility demand in the plastic hinge zone of cantilever walls was related to the displacement ductility in 'Flexural ductility of cantilever walls'. Typical relationships were also presented in figure 16. It will be seen that in a relatively slender shear wall with $h_w/\ell_w = 8$, a curvature ductility of approximately 11 is required if the displacement ductility is to be 4. The yield curvature of a section may be approximated by $\phi_y = (\varepsilon_y + \varepsilon_{ce})/\ell_w \simeq 0.0025/\ell_w$ where ε_y and ε_{ce} are the steel and concrete strains at the extreme edges when the yield strain of the reinforcement is just reached. Hence the desired ultimate curvature will be $\phi_u = 11\phi_y = 0.0275/\ell_w$. Current strength computations are based on the conservative assumption that $\varepsilon_c = 0.003$. It is found, however, that a strain of 0.004 can be readily attained in the extreme compression fibre of a section before crushing of the concrete commences[2]. By assuming that the maximum concrete strain will reach the value of 0.004 it is found that the neutral axis depth at this curvature needs to be $c = 0.004\ \ell_w/0.0275 = 0.145\ell_w$. As figure 16 shows however, for h_w/ℓ_w ratios less than 8 lesser curvature ductilities will suffice.

The above discussion was based on cantilevers, for which a structural type factor of S = 1 is relevant, and for which a displacement ductility demand of 4 might arise when the intended base overstrength, corresponding with $\phi_o = 1.39$, is developed. For walls with larger S factors or larger unintended overstrength (i.e. when $\phi_o > 1.39$), the displacement ductility requirement may be assumed to be proportionally reduced. Consequently the critical neutral axis depth can be conservatively assumed to be

$$c_c = 0.10\ \phi_o\ S\ell_w \qquad \text{(B-26)}$$

If desired, the designer could carry out a more refined analysis, using Eq. (B-27) which may show that a larger neutral axis depth would provide the desired curvature ductility.

$$c_c = \frac{8.6\ \phi_o\ S\ell_w}{(4 - 0.7S)\ (17 + h_w/\ell_w)} \qquad \text{(B-27)}$$

Whenever the computed neutral axis depth for the design loading on the given section exceeds the critical value c_c, given by Eq. (B-26), it will be necessary to assume that increased ductility can be attained only at the expense of increased concrete compression strains.

It is seen on the left hand side of figure 22, showing the channel shaped cross section of a single cantilever wall, that, because of the large available concrete compression area, very large curvature ductility is associated with the development of the flexural strength. A given displacement ductility, however, may require only a strain pattern shown by the heavy line. It is evident that this curvature could only be attained in the other wall section, shown on the right in figure 22, if the concrete compression strains increase considerably. The same relationship can be seen between the strain patterns (1) and (2') shown in figure 20. Excessive compression strains would lead to failure of the section unless the concrete in the core of the compression zone is suitably confined. This aspect of the design is examined in the next section.

Confinement of Wall Regions

From the examination of curvature relationships in the simple terms of c/ℓ_w ratio, it is seen that in cases when the computed neutral axis is larger than the critical value c_c, given by Eq. (B-26) or Eq. (B-27), the compression region of the wall needs to be confined. It does not seem necessary to confine the entire compression zone. It is suggested, however, that the outer half of it be confined. Accordingly the following simple rules are suggested.

Region of confinement

When the neutral axis depth in the potential yield regions of a wall, computed for the most adverse combination of design loadings, exceeds

$$c_c = 0.10\ \phi_o\ S\ell_w \qquad \text{(B-26)}$$

the outer half of the compression zone, where the compression strain, computed when the ideal flexural strength of the section is being determined, exceeds 0.0015, should be provided with confining reinforcement. This confining transverse reinforcement should extend vertically over the probable plastic hinge length, which for this purpose should be assumed to be equal to the length of the wall ℓ_w, as shown in figure 15 and figure 19.

Confining reinforcement

The principles of concrete confinement (2) to be used are those relevant to column sections, with the exceptions that very rarely will the need arise to confine the entire section of a shear wall. Accordingly it is recommended that rectangular or polygonal hoops and supplementary ties, surrounding the longitudinal bars in the region to be confined, should be used so that

$$A_{sh} = 0.3\ s_h\ h''\ \left(\frac{A_g^*}{A_c^*} - 1\right)\ \frac{f'_c}{f_{yh}}\ \left(0.5 + 0.9\frac{c}{\ell_w}\right) \qquad \text{(B-28)}$$

$$A_{sh} = 0.12\ s_h\ h''\ \frac{f'_c}{f_{yh}}\left(0.5 + 0.9\frac{c}{\ell_w}\right) \qquad \text{(B-29)}$$

whichever is greater, where the ratio c/ℓ_w need not be taken more than 0.8.

In the above equations:

A_{sh} = total effective area of hoops and supplementary cross ties in direction under consideration within spacing s_h, mm^2

s_h = vertical centre to centre spacing of hoop sets, mm

A_g^* = gross area of the outer half of wall section which is subjected to compression strains mm^2

A_c^* = area of concrete core in the outer half of section which is subjected to compression strains, measured to outside of peripheral hoop legs, mm^2

f'_c = specified compression strength of concrete, MPa

f_{yh} = specified yield strength of hoop or supplementary cross tie steel, MPa

h'' = dimension of concrete core of section measured perpendicular to the direction of the hoop bars, mm

These equations are similar to those developed by Park[17] for columns. The area to be confined is thus extending to $0.5c_2$ from the compressed edge as shown by cross hatching in the examples of figures 20 and 22.

For the confinement to be effective the vertical spacing of hoops or supplementary ties, s_h, should not exceed 6 times the diameter of vertical bars in the confined part of the wall section, one third of the thickness of the confined wall or 150 mm, whichever is less.

An application of this procedure is given in Appendix II.

Confinement of longitudinal bars

A secondary purpose of confinement is to prevent the buckling of the principal vertical wall reinforcement where the same may be subjected to yielding in compression. It is therefore recommended that in regions of potential yielding of the longitudinal reinforcement within a wall with two layers of reinforcement, where the longitudinal reinforcement ratio ρ_ℓ, computed from Eq. (B-31), exceeds $2/f_y$, transverse tie reinforcement, satisfying the following requirements, should be provided:

(a) Ties suitably shaped should be so arranged that each longitudinal bar or bundle of bars, placed close to the wall surface, is restrained against buckling by a 90^o bend or at least a 135^o standard hook of a tie. When two or more bars, at not more than 200 mm centres apart, are so restrained, any bars between them should be exempted from this requirement.

(b) The area of one leg of a tie, A_{te}, in the direction of potential buckling of the longitudinal bar, should be computed from Eq. (B-30) where ΣA_b is the sum of the areas of the longitudinal bars reliant on the tie including the tributary area of any bars exempted from being tied in accordance with (a) above.

$$A_{te} = \frac{\Sigma A_b\ f_y}{16\ f_{yh}}\ \frac{s_h}{100} \qquad \text{(B-30)}$$

Longitudinal bars centered more than 75 mm from the inner face of stirrup ties need not be considered in determining the value of ΣA_b.

(c) The spacing of ties along the longitudinal bars should not exceed six times the diameter of the longitudinal bar to be restrained.

(d) Where applicable, ties may be assumed to contribute to both the shear strength of a wall element and the confinement of the concrete core.

(e) The vertical reinforcement ratio that determines the need for transverse ties should be computed from

$$\rho_\ell = \frac{\Sigma A_b}{b s_v} \qquad \text{(B-31)}$$

where the terms of the equation, together with the interpretation of the above requirements are shown in figure 23. The interpretation of Eq. (B-31) with reference to the wall return at the left hand end of figure 23 is as follows : $\rho_\ell = 2A_b/bs_v$.

The requirements of transverse reinforcement is a shear wall section are summarized in figure 24 as follows:

(a) For the direction of loading the computed neutral axis depth c exceeds the critical value c_c, given by Eq. (26) or Eq. (27), hence confining reinforcement over the outer half of the compression zone, shown by cross hatching, should be provided in accordance with "Confining reinforcement".

(b) In the web portion of the channel shaped wall, within the outer half of the computed neutral axis depth, vertical bars need be confined (using antibuckling ties) in accordance with "Confinement of longitudinal bars" only if $\rho_\ell > 2/f_y$. The affected areas are shaded.

(c) In all other areas, which are unshaded the transverse (horizontal) reinforcement need only satisfy the requirements for shear and its ratio to the concrete area should not be less than 0.0025.

Longitudinal Wall Reinforcement

For practical reasons the ratio of longitudinal i.e. vertical reinforcement, ρ_ℓ, (Eq. (B-31)) over any part of wall should not be less than $0.7/f_y$ nor more than $17/f_y$.

In walls which are thicker than 200 mm or when the design shear stress exceeds $0.3\sqrt{f'_c}$ MPa, at least two layers of reinforcement should be used, one near each side of the wall.

The diameter of bars used in any part of a wall should not exceed one tenth of the thickness of the wall. The spacing between longitudinal bars should not exceed twice the thickness of the wall nor 400 mm.

In regions where the wall section is required to be confined the spacing of vertical bars should not exceed 200 mm.

Control of Shear Failure

Shear forces and shear stresses

The derivation of the design shear forces, using the principles of capacity design, have been outlined previously for cantilever walls ("Shear strength of cantilever walls") and in "Wall shear forces" for coupled shear wall structures. Shear strength provided in accordance with these shear forces is expected to ensure ductile flexural response of walls with an acceptable amount of reduction in energy dissipation during hysteretic response. For convenience and in keeping with traditional practice these forces may be converted into stresses thus

$$v_i = \frac{V_{wall}}{b_w d} \qquad \text{(B-32)}$$

where the effective depth need not be taken less than $0.8\ \ell_w$. Eq. (B-32) should be considered as an index rather than an attempt to quantify a stress level at any particular part of the wall section. From observed behaviour of walls, using this expression, certain limits have been set to ensure satisfactory performance.

Shear may lead to different types of failure, such as diagonal tension, diagonal compression and sliding, each of which are examined subsequently. In general the principles relevant to the design of ordinary reinforced concrete beams(2) are also applicable to structural walls.

Control of diagonal tension and compression

Two areas within a wall must be distinguished for which the design procedures

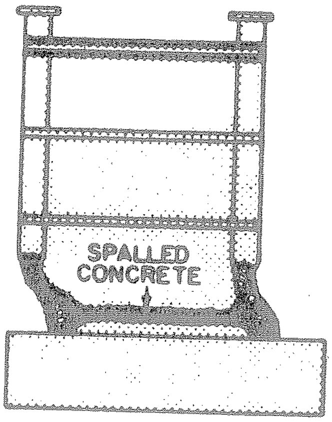

Fig. 25 - Sliding Shear Failure Initiated by Web Crushing.

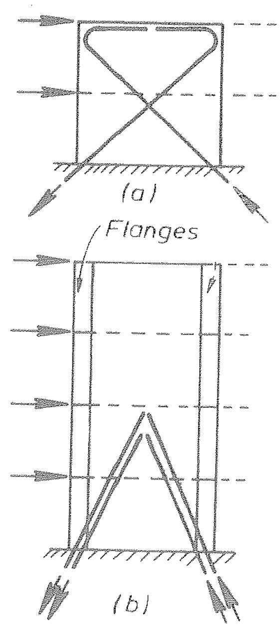

Fig. 26 - Suggestions for the Arrangement of Diagonal Reinforcement to Control Sliding Displacement at the Base.

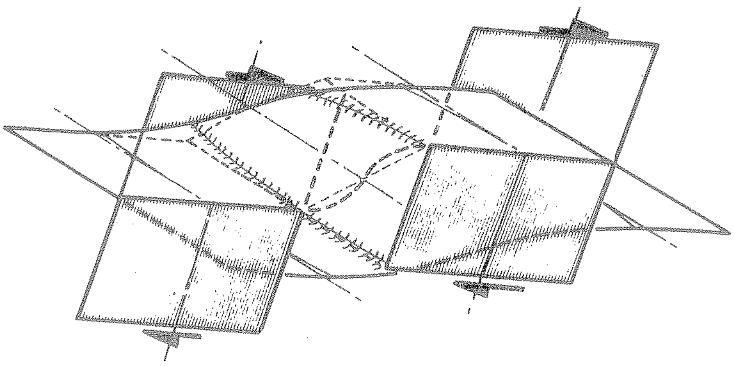

Fig. 27 - The Inelastic Deformations of a Slab Interconnecting two Laterally Loaded Shear Walls.

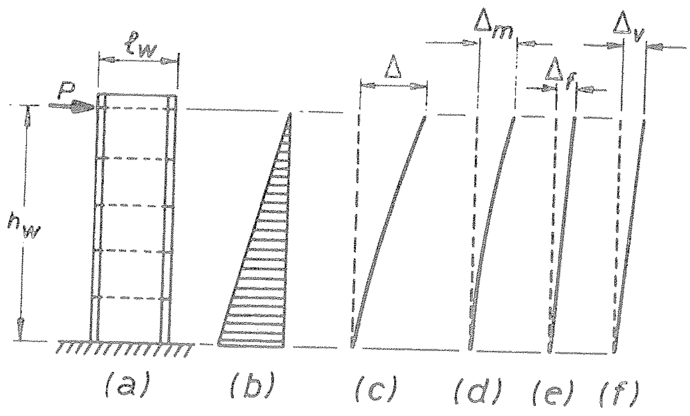

Fig. 28 - A Cantilever Wall and its Distortions.

are different. These are the potential plastic hinge zone and the remainder of the wall, which is expected to remain free of significant flexural yielding during any kind of dynamic excitation. In the design to control diagonal tension, one part of the shear strength is assumed to be provided by the shear reinforcement (v_s) and the other by mechanisms collectively designated as the contribution of the concrete (v_c). Accordingly

$$v_i = v_c + v_s \qquad \text{(B-33)}$$

In this the contribution of the "concrete" to shear resistance, v_c, is assumed to be zero in the potential plastic hinge zone, unless the minimum design axial load, N_u, produces an average compression stress of 0.1 f'_c or more over the gross concrete area, A_g, including flanges, in which case

$$v_c = \frac{2}{3}\sqrt{\left(\frac{N_u}{A_g} - \frac{f'_c}{10}\right)} \qquad \text{(B-34)}$$

The value of v_c outside the potential plastic hinge zone may be taken as that specified for beams[8] subjected to gravity (non-seismic) loading only. This will normally result in significant reduction in the web reinforcement in the upper parts of a shear wall.

Web reinforcement, consisting of horizontal bars, fully anchored at the extremities of the wall section, must be provided so that

$$A_v = \frac{v_s\, b_w\, s}{f_y} = \frac{(v_i - v_c)\, b_w s}{f_y} \qquad \text{(B-35)}$$

These provisions should ensure that diagonal tension failure across the wall will never occur. To guard against diagonal compression failure, which may occur in flanged walls, that are over-reinforced for shear, codes[8, 16] set an upper limit for the value of v_i. These values were based on tests with monotonic loading. Recent tests by the Portland Cement Association[11] and the University of Berkeley[18] have demonstrated, however, that web crushing in the plastic hinge zone may occur after only a few cycles of reversed loading involving displacement ductilities of 4 or more. When the imposed ductilities were only 3 or less, the shear stresses stipulated by existing codes[16] could be repeatedly attained. Web crushing may eventually lead to apparent sliding shear failure, as shown in figure 25. To prevent such failure the ideal shear strength of the wall should be such that

$$v_{i,max} \leqslant (0.3\, \phi_o\, S + 0.16)\sqrt{f'_c} \leqslant 0.8\sqrt{f'_c}\ \text{(MPa)} \qquad \text{(B-36)}$$

It is seen that for cantilever shear walls with $\phi_o = 1.39$ and a structural type factor of $S = 1.6$, in which limited displacement ductility demand is expected, the design shear stress will attain the maximum value considered for all structures i.e. $0.8\sqrt{f'_c}$ MPa. On the other hand for a coupled shear wall structure with $\phi_o = 1.39$ and $S = 0.8$, $v_{i,max} = 0.49\sqrt{f'_c}$.

Control of sliding shear

It is likely that sliding in the plastic hinges of walls is better controlled by conventional reinforcement than it is in beams where sliding, resulting from high intensity reversed shear loading, can significantly affect the hysteretic response (see figure 4). The reasons for this are that most shear walls carry some axial compression due to gravity and this assists in closing cracks across which the tension steel yielded in the previous load cycle, and that the more uniformly distributed and embedded vertical bars across a potential sliding plane provide better dowel shear resistance.

Also, more evenly distributed vertical bars across the wall section provide better crack control. In beams several small cracks across the flexural reinforcement may merge into one or two large cracks across the web, thereby forming a potential plane of sliding. Because of the better crack control and the shear stress limitation imposed by Eq. (B-36), it does not appear to be necessary to provide diagonal steel across the potential sliding planes of the plastic hinge zone, as it has been suggested[8] for beams. However, it is recommended that in low rise shear walls some of the shear should be resisted by diagonal bars, placed in the middle of the wall thickness, particularly when the minimum axial compression stress on such walls is less than 0.1 f'_c and the shear stress exceeds $0.4\sqrt{f'_c}$. Suggested arrangements are shown in figure 26. Such bars should be included in the evaluation of the flexural resistance and may be included in the resistance to diagonal tension.

Construction joints represent potential weaknesses where sliding shear displacement can occur. Therefore it is recommended that the design for shear transfer across construction joints be based on the shear friction mechanism[2]. Accordingly where shear is resisted at a construction joint by friction between carefully roughened surfaces and by dowel action of the vertical reinforcement, the ratio of reinforcement that crosses at right angles to the construction joint should not be less than

$$\rho_{vf} = \left(v_{wall} - \frac{N_u}{A_g}\right)\frac{1}{f_y} > 0.0025 \qquad \text{(B-37)}$$

where N_u is the minimum design compression force on the wall. For tension, N_u should be taken as negative. V_{wall} is obtained from Eq. (B-20) or Eq. (B-25).

Detailing of Coupling Beams

The ductility demand on coupling beams of coupled shear walls, such as examined in "Coupled Shear Walls", can be large.

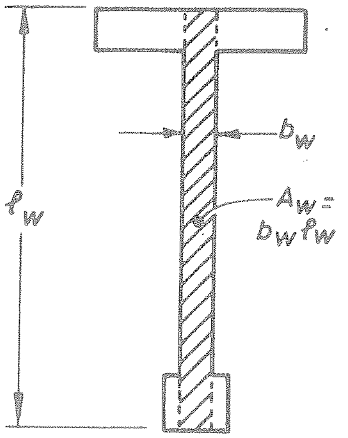

Fig. 29 - Effective Shear Area of a Flanged Wall Section.

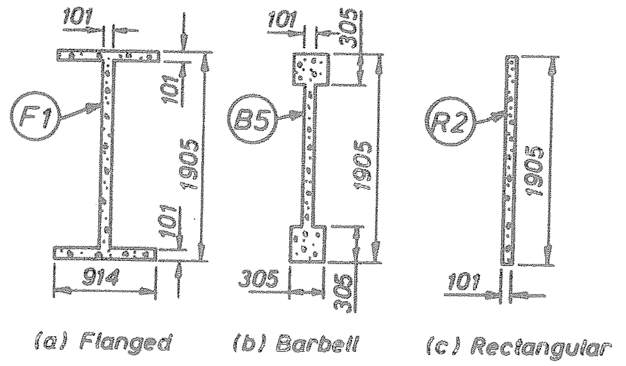

Fig. 30 - Nominal Cross Sectional Dimensions of the PCA Test Specimens (11).

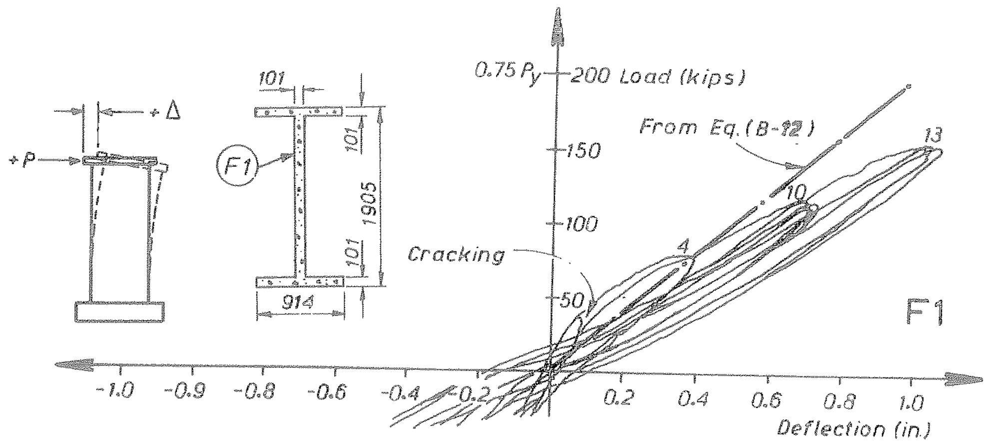

Fig. 31 - Continuous Load-Deflection Plot for Initial Cycles for the Flanged Wall Specimen FI.

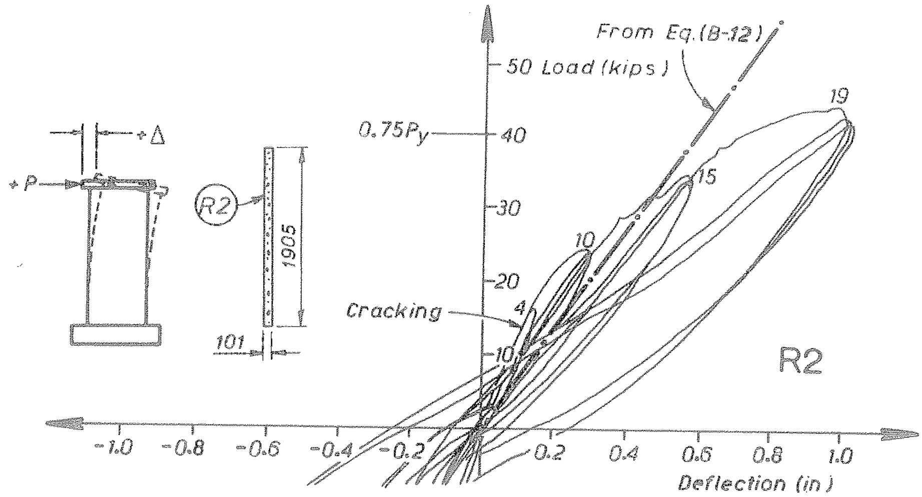

Fig. 32 - Continuous Load-Deflection Plot for Initial Cycles for the Rectangular Wall Specimen R2.

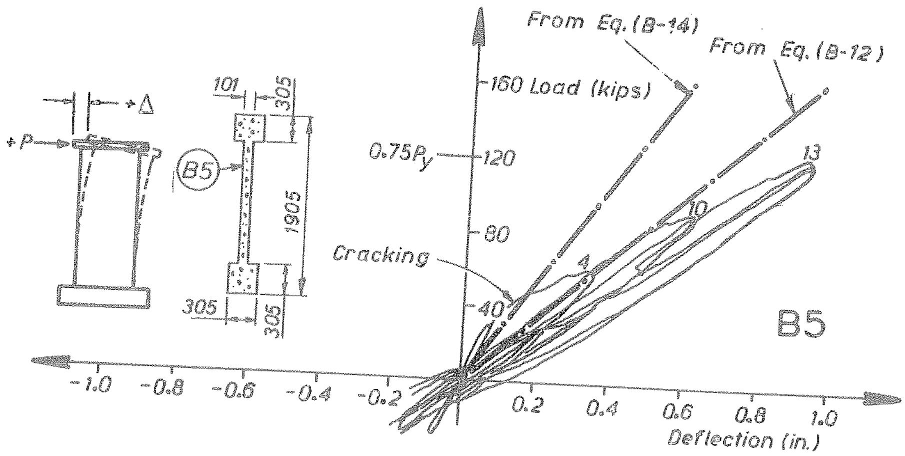

Fig. 33 - Continuous Load-Deflection Plot for the Initial Cycles for Specimen B5.

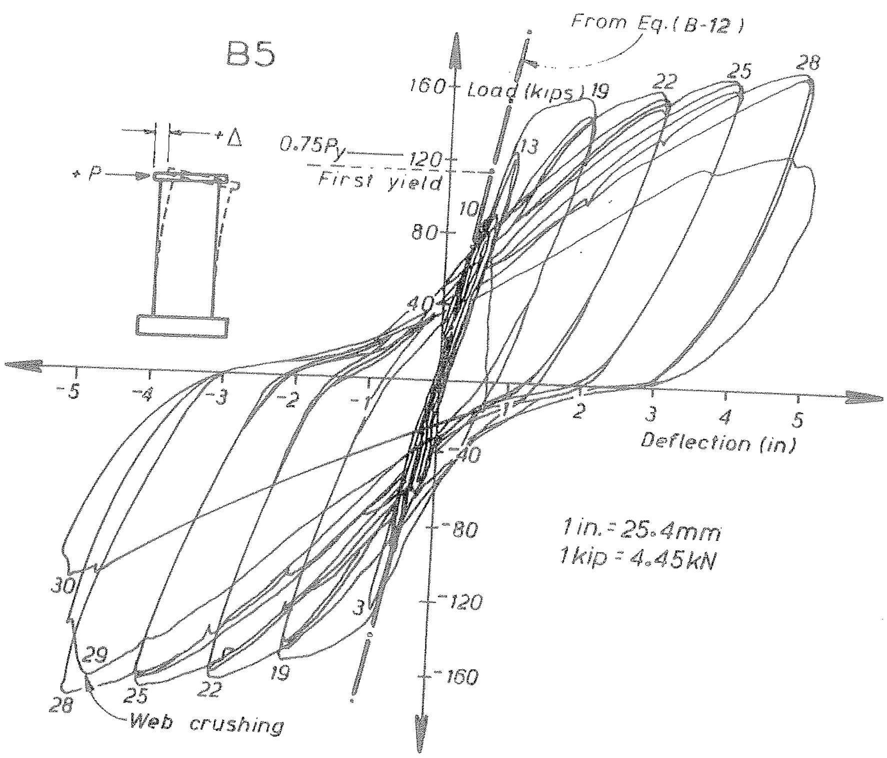

Fig. 34 - Continuous Load-Deflection Plot for all Cycles for Specimen B5

(See figure 7b). To preserve the energy dissipating properties of such beams, which are often relatively deep, diagonal reinforcement should be utilized(2) to resist simultaneously both the moments and the shear. Diagonal bars in cages should be confined to ensure that buckling of diagonal bars cannot occur. For this purpose Eq. (B-30) and the rules listed in 'Confinement of longitudinal bars' should be followed. However neither the spacing of ties nor the pitch of rectangular spirals should exceed 100 mm.

When coupling beams are as slender as normal beams, which are used in ductile frames, distinct plastic hinges will form at the ends and these can be detailed as for beams. The danger of sliding shear failure and the inhibition of flexural ductility increases with increased depth to span ratio, h/ℓ_n, and with increased shear stresses. Therefore it is recommended that in coupling beams of shear walls the entire seismic design shear and flexure should be resisted by diagonal reinforcement in both directions unless the earthquake induced shear stress is less than

$$v_i = 0.1 \frac{\ell_n}{h} \sqrt{f'_c} \qquad \text{(B-38)}$$

It should be noted that this severe limitation is recommended because coupling beams can be subjected to much larger rotational ductility demands than spandrel beams of similar dimensions in frames. There is no limitation on the inclination of the diagonal bars.

Slab Coupling of Walls

When walls are interconnected by slabs only, as shown in figure 17c, the stiffness and strength of the coupling between the two walls becomes difficult to define. In the elastic range of displacement a considerable width of the slab will participate in load transfer. However, when inelastic deformations occur in the doorway, as illustrated in figure 27, a dramatic loss of stiffness can be expected(15). Even when the flexural reinforcement is placed in a narrow band, with a width approximately equal to that of the doorway, and the band is confined by stirrup-ties enclosing the top and bottom slab bars in the band, it is difficult to control punching shear around the toes of the walls. From preliminary studies(15) it appears that the hysteretic response of slab coupling is poor and that this system does not provide good energy dissipation with reversed inelastic cyclic loading. As figure 18 indicates, the contribution of slab coupling to the total moment of resistance is not likely to be significant. For this reason its contribution to seismic strength should be neglected in most cases.

When shallow beams, projecting below the slab, are provided across doorways, it must be expected that they will fail in shear, unless the very significant contribution of the slab reinforcement, placed parallel to the coupled walls, is included in the evaluation of the flexural overcapacity of the relevant beam hinge, and thus in the evaluation of the imposed shear.

NOTATION:

A = moment parameter used for coupled shear walls

A_b = area of one bar, mm^2

$A_c{}^*$ = area of concrete core in the outer half of section which is subjected to compression strains, measured to outside of peripheral hoop legs, mm^2

A_e = effective area of the cross section of a wall subjected to axial load

A_g = gross area of section, mm^2

$A_g{}^*$ = gross area of the outer half of wall section which is subject to compression strains, mm^2

A_{sh} = total effective area of hoop bars and supplementary cross ties in directions under consideration within spacing s_h, mm^2

A_{te} = area of one leg of stirrup or stirrup tie, mm^2

A_v = area of shear reinforcement within a distance s, mm^2

A_w = effective web area of wall cross section, mm

b = width of compression face of member or thickness of rectangular wall section

b_w = web width or wall thickness

c = computed distance of neutral axis from compressive edge of the wall section

c_c = critical value of c

d = distance from extreme compression fibre to centroid of tension steel

ex, ey = eccentricity of centre of mass in x and y directions respectively

E_c = modulus of elasticity of concrete, MPa

f = form factor considered with shear deformation

f'_c = specified compressive strength of concrete, MPa

f_r = modulus of rupture of concrete, MPa

f_y = specified yield strength of steel reinforcement, MPa

f_{yh} = specified yield strength of hoop or supplementary cross tie steel, MPa

G_c = modulus of rigidity of concrete, MPa

h = overall thickness of member or depth of beam, mm

h_w = overall height of wall of horizontal length ℓ_w, mm

h'' = dimension of concrete core of section measured perpendicular to the direction of the hoop bars, mm

I = importance factor

I_{cr} = moment of inertia of cracked section transformed to concrete

I_e = effective moment of inertia for computation of flexural and shear deflections

I_g = moment of inertia of gross concrete section about centroidal axis

I_w = equivalent moment of inertia of wall section neglecting the reinforcement for computing total deflections

ℓ = distance between axes of shear walls

ℓ_n = length of clear span or distance, measured face to face of support

ℓ_w = horizontal length of wall

M_a = maximum moment in member at stage for which deflection is being computed

M_{cr} = cracking moment

M_{code} = moment induced by code specified static loading

M_i = ideal flexural strength of wall section

M_o = overturning moment at the base of a shear wall structure due to code load

M^o = moment developed at flexural overcapacity of member

, M_2 = moments due to code loading developed at the base of the wall concurrently with earthquake induced axial tension or compression respectively

M_1^o, M_2^o = flexural overcapacity developed in the tension and compression wall respectively

M_1', M_2' = design moments at the base after moment redistribution in the tension and compression walls respectively

n = number of floors above the section of wall being considered

N = number of storeys in a shear wall structure

N_u = design axial compression load normal to cross section occurring simultaneously with the design shear force, N

P_D = axial load on member due to deal load only

P_e = maximum design axial load due to gravity and seismic loading acting on the member during an earthquake, N

P_{eq} = axial load on member due to design earthquake loading only

P_{LR} = axial load on member due to reduced live load

P_{eq}^o = maximum axial load on member due to earthquake only at the development of flexural overcapacity

P_1^o, P_2^o = design axial tension and compression force acting on wall at the development of the flexural overstrength capacity of the structure

Q_i^o = shear overcapacity of a coupling beam

s = spacing of stirrups, mm

s_h = vertical spacing of horizontal reinforcement, mm

s_v = horizontal spacing of vertical reinforcement along length of wall, mm

S = structural type factor

T = tension force or period of vibration, seconds

v_c = nominal permissible shear stress carried by concrete, MPa

v_i = ideal shear stress, MPa

v_s = nominal shear stress allocated to resistance of web reinforcement, MPa

V_{code} = shear demand derived from code loading

V_i = ideal shear capacity of wall

V_{wall} = design shear force for a wall at the development of the flexural overcapacity of the structure

V^o = shear force developed at flexural overcapacity

y_t = distance from centroidal axis of gross section, neglecting the reinforcement, to the extreme fibre in tension

Z = modifier of structural type factor

Δ_f = wall deflection due to anchorage deformations only

Δ_m = wall deflection due to flexural deformations only

Δ_u = deflection at top of shear wall at ultimate state

Δ_v = wall deflection due to shear deformations only

Δ_y = deflection at top of shear wall at first yield

ϕ = strength reduction factor

ϕ_u = curvature at maximum displacement ductility

ϕ_y = curvature at first yield

ϕ_o = overstrength factor

ε_c = specified compression strain at extreme concrete fibre, 0.003

ε_{ce} = compression strain at extreme concrete fibre at first yield of tension steel

ε_u = compression strain at extreme concrete fibre at development of a maximum curvature

ε_y = yield strain of reinforcement

μ_Δ = displacement ductility factor

μ_ϕ = curvature ductility factor

ω_v = dynamic shear magnification factor

ρ_ℓ = ratio of vertical tension reinforcement in wall spaced at s_v

ρ_{vf} = ratio of reinforcement crossing unit area of construction joint

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APPENDIX I

THE ESTIMATION OF DEFLECTIONS OF CRACKED REINFORCED CONCRETE CANTILEVER WALLS

Assumptions

Deflection estimates generally used in seismic design should reflect the behaviour of the structure after the development of extensive cracking at a load level which, as yet, does not result in inelastic deformations. Therefore for the purpose of the derivations that follow, wall behaviour at 75% of the theoretical yield load will be considered. The yield load is that which causes the in part of the flexural reinforcement, .aced in boundary regions of walls, such as flanges, to yield. If for example the main flexural reinforcement in a wall section consists of seven layers of D28 bars, the yield load is that attained at the onset of yielding in the innermost (i.e. seventh layer of these D28 bars). This load will be close to the ideal flexural capacity.

In order to define the stiffness of any elastic member with given boundary conditions, a certain unit deformation must be related to a certain load pattern. For the purpose of this study the structure and the load on it are those shown in figure 28a and figure 28b, and the deformation to be determined is the lateral deflection at roof level, Δ, as shown in figure 28c.

The symbols used in the subsequent derivation are fully defined in the text or the list of symbols.

Flexural Deformations

The flexural deformations, being dominant, are normally the only ones that are considered in the design of flexural members. Accordingly the roof deflection for a homogeneous elastic cantilever wall of figure 28a is

$$\Delta = \frac{Ph_w^3}{3E_cI_g} \qquad \text{(I-1)}$$

The most appropriate approach to the estimation of cracking is to allow for a loss of effective resisting area in the cross section. The effective moment of inertia of the section, I_e, will be between that based on the uncracked section, I_g, and that obtained from the fully cracked section in which the steel area is transformed to concrete area, I_{cr}. An interpolation for I_e between the above limits has been developed by Branson and it has been adopted by the American Concrete Institute[16]. Its background is examined elsewhere[2, 8]. This is

$$I_e = \left(\frac{M_{cr}}{M_a}\right)^3 I_g + \left[1 - \left(\frac{M_{cr}}{M_a}\right)^3\right] I_{cr} \qquad \text{(B-14)}$$

The moment assumed to cause cracking is from first principles[2]

$$M_{cr} = \frac{f_r I_g}{y_t} \qquad \text{(B-15)}$$

It is seen that the relationship between the second moment of area and the moments are such that $I_g \geq I_e \geq I_{cr}$ where $1 \geq (M_{cr}/M_a) \geq 0$).

For beams and columns of normal proportions and reinforcement contents it is found that usua-ly $0.4 < I_{cr}/I_g < 0.6$, and hence the equivalent moment of inertia is such that $0.5 < I_e/I_g < 0.7$. Consequently in the elastic analysis of frames customarily the "gross moment of inertia", I_g of members is used, and this is reduced by 30 to 50% to allow for the effects of cracking.

In structural walls usually considerably less flexural reinforcement is being used than in beams of ductile earthquake resisting frames. The flexural tension steel content, $\rho = A_s/bd$, to be considered in the evaluation of flanged transformed wall sections can be as small as 0.05%. Consequently in such walls the "transformed moment of inertia", I_{cr} will be a smaller fraction of the "gross moment of inertia", I_g. Cracking has thus a more profound effect on the stiffness of normal walls than on that of beams.

The flexural deformation, shown in figure 28d can therefore be obtained thus

$$\Delta_m = \frac{Ph_w^3}{3E_cI_e} \qquad \text{(I-2)}$$

Anchorage Deformations

The analytical model commonly used is a cantilever. This is fully fixed against rotations at its base. (figure 28a). Under lateral load the vertical wall reinforcement is at its highest stress at the base. Consequently tensile strains along the flexural bars will only gradually decay in the foundation structure. The elongation of the vertical bars within the foundation structure and the slip due to high local bond stresses along the development length will result in an apparent "pull out" of such bars at the base of the wall. This can significantly increase the wall deflection, as shown in figure 28e. Based on the relative magnitudes of observed "pull out" deformations, it is suggested that its magnitude be estimated as

$$\Delta_f = 0.2\ \Delta_m \qquad \text{(I-3)}$$

Shear Deformations

It is well known that shear deformations

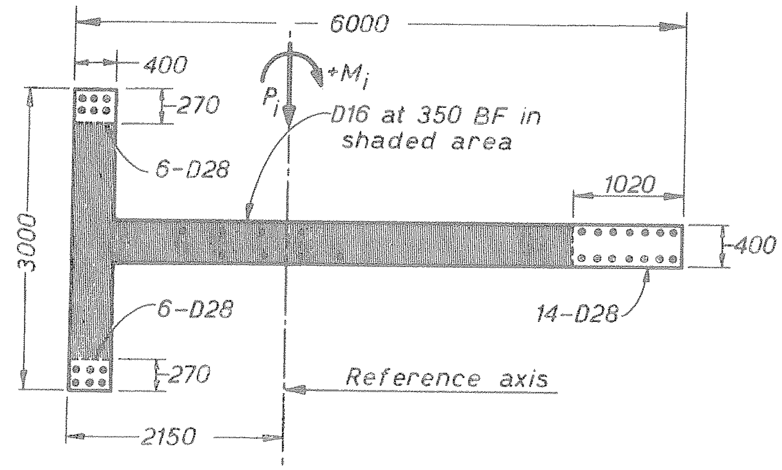

Fig. 35 - Sectional Properties of an Example Shear Wall Section.

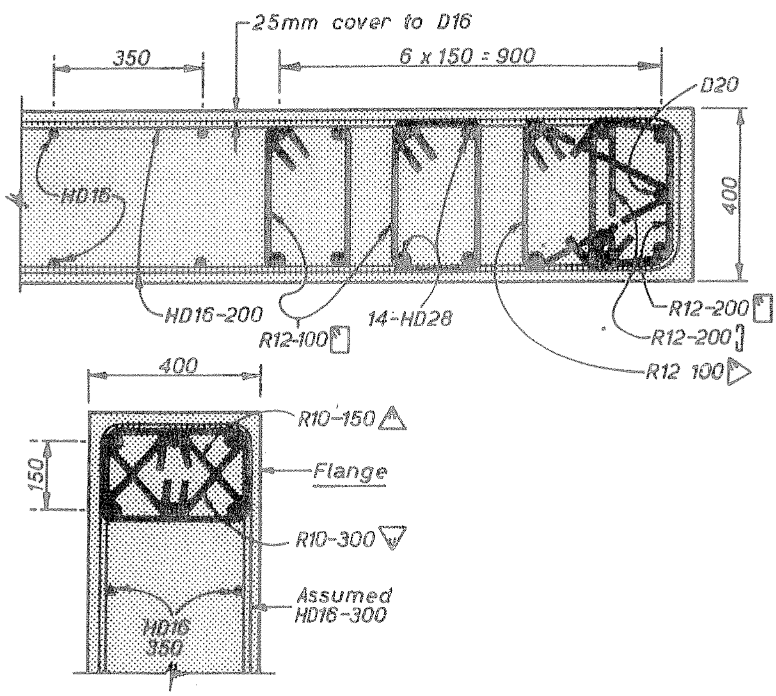

Fig. 36 - Arrangement of Transverse Reinforcment in the Critical Regions of the Example Shear Wall Section.

in slender flexural members are negligibly small in comparison with those due to flexure. Walls, however, may belong to the family of "deep beams", in which shear deformations are likely to be significant. Therefore shear deformations should be considered.

The shear deflection of a homogeneous elastic wall at roof level, shown in figure 28f, is known to be

$$\Delta_v = \frac{fPh_w}{G_c A_w} \qquad \text{(I-4)}$$

The area of the wall, effective in shear, A_w, is defined in figure 29. It will be assumed that $A_w = b_w \ell_w$ for the common type of walls used.

It has been found that in members in which diagonal cracks have developed as a result of shear stresses, the relative contribution of shear deformations is considerably larger than what Eq. (I-4) would predict. It will be appreciated that after the development of diagonal cracking a new form of shear transfer begins to operate i.e. the truss mechanism. In this new mechanism the web reinforcement (stirrups) contributes to large shear strains. It has been shown[2] that the shear stiffness of diagonally cracked beams is only 10-30% of that of uncracked beams, depending on the contribution of web reinforcement.

The estimation of shear deformation in a shear wall is complicated by the fact that the shear force in a real wall will decrease from a minimum at the top of the wall to a maximum of the base. Moreover, in the lower portions of the wall more extensive flexural and shear cracking will occur, and it can be expected that in these more heavily cracked zones the shear deformations will be larger. Taking these considerations into account it is suggested that the contributions of shear deformations along the height of a cantilever wall be estimated from the following simple expression:

$$\Delta_v = \frac{1.2\ Ph_w}{0.4\ E_c 0.3A_w} = \frac{10Ph_w}{E_c A_w} \qquad \text{(I-5)}$$

Combined Deformations

It is seen from figure 28 that the roof deflection of the cracked cantilever wall due to flexural, anchorage pull-out and shear deformations is $\Delta = \Delta_m + \Delta_f + \Delta_v$. Substituting from Eqs. (I-2), (I-3) and (I-5) we obtain

$$\Delta = \frac{Ph_w^3}{3E_c I_e} + \frac{0.2Ph_w^3}{3E_c I_e} + \frac{10Ph_w}{E_c A_w} \qquad \text{(I-6)}$$

It is convenient to express the deflection in terms of flexural deformations and an equivalent wall moment of inertia, I_w, so that

$$\Delta = \frac{Ph_w^3}{3E_c I_w} \qquad \text{(I-7)}$$

By equating the above two equations the equivalent wall moment of inertia, I_w, is obtained thus

$$I_w = \frac{I_e}{1.2 + F} \qquad \text{(B-12)}$$

where value of I_e is given by Eq. (B-14) and

$$F = \frac{30\ I_e}{h_w^2 b_w \ell_w} \qquad \text{(B-13)}$$

A Comparison with Experiments

Recently the Portland Cement Association in Skokie (US) carried out extensive testing with centilever shear walls[11]. Some observed results of this programme are compared with values obtained from Eq. (B-12) and Eq. (B-14). All the walls reported have the same aspect ratio of $h_w/\ell_w = 2.4$. This is in the range where shear deformations are likely to be significant.

The basic dimensions of the cross sections used for the 4752 mm high wall specimens are shown in figure 30. A comparison of predicted deflections with observed ones was made for all seven specimens reported. However, representative results for only three of the cases are presented here.

Figure 31 shows the initial cycles of the load displacement relationship for the flanged wall specimen (figure 30), when the load did not exceed approximately 60% of the yield load P_y. The straight line shows the idealized relationship that would have resulted from Eq. (B-12).

A similar relationship is shown in figure 32 for a wall with a rectangular cross section. In the response shown the maximum load reached approximately 83% of the yield load, Py.

Finally a comparison is made for a wall with a rectangular boundary element (barbell), B-5, in figure 33. Here Eqs. (B-12) and (B-14) are compared. It is seen that Eq. (B-14) generally recommended[8] for the prediction of beam deflection, overestimates the wall stiffness. The differences in deflections, as predicted by the two equations, result from the considerations of shear and anchorage deformations, which have been incorporated into Eq. (B-12). The full response, including the inelastic cycles, of this wall specimen, is shown in figure 34.

With respect to the PCA experiments used here, it may be said that the suggested deflection estimate procedure should be acceptable for design purposes.

APPENDIX II

DESIGN OF A CANTILEVER SHEAR WALL

Design Requirements and Properties

Preliminary design has indicated that one of several symmetrically arranged cantilever shear walls of a 11 storey Class III building, resisting the required seismic loading, may be dimensioned and reinforced at ground floor level as shown in figure 35. In this study seismic actions in the longitudinal direction of the wall sections are considered only. The first storey is 3.50 m high and the upper 10 storeys are 3.25 m each.

The strength properties to be used are as follows:

Concrete	f'_c = 25MPa
Vertical wall reinforcement	f_y = 380MPa
Horizontal wall shear reinforcement	f_y = 380MPa
Horizontal hoops and ties	f_y = 275MPa

The total loading at ground floor level from all the tributary areas of the upper floors is as follows:

Dead load	7000 kN
Reduced live load	3000 kN

The centre of the lateral static load, used in the preliminary design, was located at 23 m above ground floor. At ground level the wall is assumed to be fully fixed against rotations.

Minimum requirements with respect to

i) Section "Stability" i.e. $\ell_n/b < 3500/400 = 8.75 < 10$

ii) Section "Longitudinal Wall Reinforcement" i.e. $\rho_{\ell,min} = 0.7/380 < 2 \times 201/(400 \times 350) = 0.004$

and

iii) Bars spacing requirements are all satisfied

Flexural Capacities

The flexural capacities are to be evaluated for each direction of loading. The maximum axial compression to be considered for the evaluation of the available ideal flexural strength is from(1)

$$U_{ideal} = (D + L_R)/\phi = (7000 + 3000)/0.9 = 11,100 \text{ kN}$$

Loading causing compression in the flange

P_i = 11,100 kN M_i = ?

Using a trial and error process, the neutral axis depth will be estimated so that the internal compression forces less the tensile forces will give a compression resultant of approximately 11 MN. Then the moment about the reference axis (the centroid of the gross concrete section) will be computed.

Assume first c = 0.05 x 6000 = 300 mm

The D16 bars provide (2 x 201)380/(0.35 x 10^6) = 0.44 MN force per meter wall length.

Ignore contribution of reinforcement in the flange and the reduction of steel flexural contribution in the elastic core of the section then:

Compression C_c = (0.85 x 300)3000x(0.85 x 25)/10^6 ≃ 16.3MN

Tension T_{28} = 14 x 615 x 380/10^6 = 3.27

T_{16} = (6.0 − 0.4 − 1.02) 0.44 = 2.01

Total tension T = 5.28≃5.3MN

Therefore $C_c - T$ = P_i ≃11.0MN

M_i = 16.3(2.15 − 0.5 x 0.85 x 0.3) =33.0MNm
3.27(6.00− 2.15−0.5x1.02) =10.9MNm
2.01(0.5x4.58+0.4−2.15) = 1.1MNm

No new trial for c is required. Therefore M_i =45.0MNm

Loading causing tension in the flange

P_i = 11.1 MN M_i = ?

Assume first c = 0.35 x 6000 = 2100 mm

Compression C_c = (0.85x2100)400(0.85x25)/10^6 =15.2MN
C_{28}=14x615x380/10^6 ≃ 3.3MN
C_{16}=neglect = −

Total compression C =18.5MN

Tension in the flange T_{28}=(6x615)380/10^6 = 1.4MN
T_{16}=(3.0−2x0.27)0.44 = 1.1MN
in the web T_{16}=(6.0−0.4−2.1)0.44 = 1.5MN

Total tension T = 4.0MN

Net compression P_i = 11.1 < 14.5MN

Reduce a by Δ a = (14.5−11.1)10^6/(0.85x25x400) = say 370 mm

Hence c = 2100 − 370/0.85 = 1664 mm

by proportion
C_c = 1664x15.2/2100 =12.0MN
C_{28} = as before = 3.3MN
C_{16} = as before = −

15.3MN

T_{28} = as before = 1.4MN
in the flange
T_{16} = as before = 1.1MN
in the web T_{16} = (6.0−0.4−1.66)0.44 = 1.7MN

4.2MN

P_i = 11.1 =11.1MN

M_i = 12.0(6.0-2.15-0.85x1.66x0.5) = 37.7MNm
3.3(6.0-2.15-0.5x1.02) = 11.0MNm
(1.4+1.1)(2.15-0.5x0.4) = 4.9MNm
1.7{-(6.0-0.4-1.66)0.5+2.15-0.4} = 0.4MNm

Hence moment of resistance is M_i = 53.2MNm

Design for Shear

As the ideal moment capacity for the most adverse load combination is 53.2MN, the code required shear is close to 0.9x53.2/23 = 2.08MN.

For a 11 storey building the dynamic shear magnification from Table B-I is ω_v = 1.7. With a flexural overstrength of 125% of ideal strength, the design shear force for the wall is obtained from Eq. (B-18).

wall = 1.7 x 1.25 x 2.08 = 4.42 MN

Hence from Eq. (B-32)

v_i = 4.42 x 10^6/(400 x 0.8 x 6000) = 2.30MPa

From Eq. (B-36) the maximum allowable shear stress is

$v_{i,max}$ = (0.3 x 1.39 x 1.0 + 0.16) $\sqrt{25}$ = 2.89 > 2.30 MPa

N_u/A_g = 11.1 x 10^6/{6000x400+(3000-400)400} = 3.23MPa

From Eq. (B-34)

v_c = 0.25(1 + 25/25) $\sqrt{3.23-25/10}$ = 0.43MPa

From Eq. (B-33)

v_s = $v_i - v_c$ = 2.30 - 0.43 = 1.87MPa

From Eq. (B-35)

= 1.87x400xs/380 = 1.97s

Assume two legs of HD16 bars, A_v = 402mm^2

s = 402/1.97 = 204 ≃ 200 mm

Use HD16 at 200 mm crs for horizontal shear (stirrup) reinforcement

Confinement

It is evident that no confinement is required when the flange is in compression as the section is extremely ductile with c/ℓ_w = 0.05. However, when the flange is in tension the stem of the section will need to be confined. For this it was found in "Loading causing tension in the flanges", that c = 1664 mm.

From Eq. (B-26) with ϕ_o = 1.4

c_c = 0.10x1.4x1.0x6000 = 840 < 1664

Hence provide confinement over a length of 0.5 x 1664 = 832 mm

For Eqs. (B-28) and (B-29) to be used take the following values

h" = 832-41+0.5x12 = 797 ≈ 800,

A_g^* = 400x832 = 333000 mm^2, Assume R12 ties,

Assume cover to HD stirrups = 25 mm and to main bars 41 mm, hence

A_c^* = (400-2x41+2x12)(832-41+12) = 275000 mm^2

$(A_g^*/A_c^*$ - 1) = (333/275-1) = 0.21

0.3x0.21 = 0.063 < 0.12 hence Eq. (B-29) governs

From Eq. (B-29)

A_{sh}= 0.12s_h 800 x (25/275)(0.5 + 0.9 x 1664/6000) = 6.54s_h

With 6 R12 legs over 800 mm length

s_h = 6x113/6.54 = 104 mm

From the spacing requirements stated in "Confining reinforcement"

$s_{h,max}$ = 6 x 28 = 168 or 400/3 = 133 or 150 mm

Hence use R12 hoops and ties at 100 mm crs and for practical reason confine all 14 HD28 bars.

For the confinement in the longitudinal direction h" = 400 - 2 x 41 + 12 = 330 mm

As the distance between the 2 HD28 bars is more than 200 mm, it will be necessary to place in the confined region an intermediate (nominal) bar in between them. A D20 bar will enable another tie to be placed over the 400 mm width of the section. Hence by proportion from the above derivation of A_{sh} and s_h = 100

A_{sh} = (330/800)5.90 x 100 = 243 mm^2

R10 legs could be used, but for the sake of uniformity R12 ties will be provided as shown in figure 36.

To confine the HD28 bars against buckling at the ends of the flange, ties are required in accordance with 'Confinement of longitudinal bars' and Eq. (B-30)

From Eq. (B-31)

ρ_ℓ = 3 x 615/400 x 150 = 0.0308 > 0.0075

Hence

$$A_{te} = \frac{615}{16}\,\frac{380}{275}\,\frac{s_h}{100} = 0.53 s_h$$

The max^m spacing is 6 x 28 = 168 mm

R10 ties may be used, thus

s_h = 78.5/0.53 = 148 mm

Use R10 ties at 150 mm crs as shown in figure 36

The confining reinforcement as computed should extend, in accordance with figure 15, to a height of ℓ_w = 6000 mm, i.e. up to the 2nd floor of this structure.

Note that a more rigorous analysis, using Eq. (B-27) would have given the critical value for the neutral axis depth as follows:

With $S = 1.0$, $\ell_w = 6000$ and $h_w =$

$3.5 + 10 \times 3.25 = 36$ m

$$c_c = \frac{8.6x1.4x1x6000}{(4-0.7x1)(17+36/6)} = 952\text{mm} > 840 < 1664$$

Hence confinement is to be provided as computed above.

CURRICULUM VITAE

NAME: **ATHOL JAMES CARR**

PLACE OF BIRTH: Methven, New Zealand

ACADEMIC QUALIFICATIONS:

1. 1964 Bachelor of Engineering (Civil)(1st Class Honours) University of Canterbury. N.Z.
2. 1966 Master of Science in Engineering, University of California, Berkeley, U.S.A.
3. 1967 Doctor of Philosophy, University of California, Berkeley, U.S.A.

PROFESSIONAL QUALIFICATIONS:

1. Member, American Society of Civil Engineers
2. Fellow, Institution of Professional Engineers New Zealand
3. Life Member, New Zealand National Society for Earthquake Engineering

UNIVERSITY EXPERIENCE:

1.	1964–1966	Teaching Assistant, Department of Civil Engineering, Div. SESM, University of California, Berkeley, U.S.A.
2.	1968–1971	Lecturer, Department of Civil Engineering, University of Canterbury.
3.	1972–1986	Senior Lecturer, Department of Civil Engineering, University of Canterbury.
4.	1987–2006	Reader, Department of Civil Engineering, University of Canterbury.
5.	2007–2010	Professor, Department of Civil Engineering, University of Canterbury.
6.	2010-	Professor Emeritus, Department of Civil and Natural Resources Engineering, University of Canterbury.
7.	1975	Visiting Professor, Institutt for Statikk, Norges Tekniske Høgskole, Universitet i Trondheim, Norge.
8.	1982	Visiting Professor, Institutt for Statikk, Norges Tekniske Høgskole, Universitet i Trondheim, Norge.
8.	1989	Visiting Professor, Institutt for Konstruksjonsteknikk, Norges Tekniske Høgskole, Universitet i Trondheim, Norge
9.	2001	Visiting Professor, Department of Structural Engineering, University of California, San Diego, La Jolla, California, USA.

10. 2001 Visiting Professor, Section of Structural Mechanics, Faculty of Civil Engineering and Geosciences, Technical University of Delft, Delft, The Netherlands.

11. 2001 Visiting Professor, Earthquake Engineering Research Centre, University of Iceland, Selfoss, Iceland.

12. 2005- Appointed Member of Faculty of the Rose School (Advanced Studies in Reduction of Seismic Risk) at the University of Pavia, Pavia, Italy
Teach a course on 'Non-linear dynamic analysis', 2005,2008,2011

13. 2009- Adjunct Professor, Earthquake Engineering Research Centre, University of Iceland.

AWARDS:

1964 "The Travelling Scholarship in Engineering"

1964 Fulbright Travel Award for study at the University of California

1966 University Fellowship, University of California, Berkeley

1983 Otto Glogau Award, presented by the New Zealand National Society for Earthquake Engineering (as a member of the Bridge Study Group)

1985 Otto Glogau Award, presented by the New Zealand National Society for Earthquake Engineering (with B. Wood and P. Boardman for Paper "Union House – A Cross Braced Structure with Base Isolation")

1985 Freyssinet Award, presented by IPENZ (with B. Wood and P. Boardman for Paper "Union House – A Cross Braced Structure with Base Isolation").

1995 Structural Award, presented by IPENZ for the paper "Dynamic Analysis of Structures" published in the Bull. N.Z.National Society for Earthquake Engineering. v27, no2, June 1994.

2004 Erskine Grant to visit the USA, Germany, Norway and Iceland to discuss developments in the teaching of structural dynamics and mechanics.

2005 Japanese Society for the Promotion of Science (JSPS) Fellowship to spend 8 weeks in Japan to collaborate with Japanese researchers in earthquake engineering. This Fellowship was taken in January-February 2007, mostly at University of Ryukus, Okinawa,.

2010 Awarded Life Membership of the New Zealand Society for Earthquake Engineering.

UNIVERSITY COMMITTEES:

1971–1973	School of Engineering Computer Centre Liaison Committee
1972–1974	Faculty of Engineering Timetable Committee
1973–1974	Computer Centre Liaison Officer, Department of Civil Engineering
1976–1981	Computer Centre Liaison Officer, Department of Civil Engineering
1983–1988	Computer Centre Liaison Officer, Department of Civil Engineering
1976–1977	Committee, Sixth Australasian Conference on the Mechanics of Structures and Materials (held at University of Canterbury, August 1977)
1978–1979	Faculty of Engineering Timetable Committee
1981	Chairman, Faculty of Engineering Subcommittee on Computer Aided Design
1983–1988	Chairman, Faculty of Engineering Subcommittee on Computer Aided Design
1983–1988	Chairman, Faculty of Engineering Computer Committee
1990-2005	Faculty of Engineering, Committee on Computer Aided Design, Deputy Chairman, 1997 – Chairman
1991-2000	Academic Board, Civil Engineering Department representative
1997-2005	Faculty of Engineering, Associate Dean (Computing)
1997–1998	Faculty of Engineering, Statutes and Prescriptions Committee representative
1999--2000	Member Information Technology Services Committee Chairman, Electronics and Telecommunications Sub-committee Chairman, Working Party on Network Reliability
2001-2005	Chair, Department of Civil Engineering, Research Committee.
2002-2003	Chair, University of Canterbury IT Committee
2003-2007	Chair, University of Canterbury IT Advisory Committee
2003-2009	Chair, College of Engineering IT Committee
2006	Acting Chair, Department of Civil Engineering Postgraduate Committee
2006-2007	Department of Civil Engineering Executive Committee

EXTENSION STUDIES SEMINARS:

The Finite Element Method in Structural Mechanics, University of California, Berkeley, March 20–24, 1967. *Refined Shell Analysis and Dynamic Response.*

Use of Digital Computer in Civil Engineering, University of Canterbury, July 8–12, 1969. *Communication with the Computer*, and *The Computer in Structural Engineering and Structural Mechanics.*

Matrix Methods in Engineering Analysis. University of Auckland, September 2–5, 1970. *Stress Analysis Problems.* Faculty member in charge of Seminar.

Analysis of Shells, University of Canterbury, May 21–23, 1969. *Finite Element Analysis of Shells,* and *Hanging Roofs.*

Engineering Seismology and Fundamentals of Seismic Design of Earth Structures. University of Canterbury, May 8–9, 1978. *Site Effects.*

Dynamics of Structures, University of Auckland (with I.G. Buckle and B.J. Davidson), Part I, 6 weekly lectures from 16 July, 1980; Part II, 21–23 August, 1980.

Fundamentals of Earthquake Engineering, University of Canterbury, 6 May 1993. *Linear Seismic Responses of Multi-storey Structures,* and *Nonlinear Seismic Response of Multi-storey Structures.*

Earthquake Engineering, Concepts and Structural Modelling, June 20-22, 2001, Technical University of Delft, Delft, The Netherlands.

SHORT COURSES given by Athol J. Carr

“Non-linear Seismic Analysis of Reinforced Concrete Buildings”. Five evening course given for ACHISINA Asociación Chilena de Sismología e Ingeniería Antisísmica, Santiago, Chile, 2-6 June 2008.

“Earthquake Engineering”. A Joint Five afternoon Course of Reykjavik University and University of Iceland. Reykjavik, Iceland, August 2009

PROFESSIONAL COMMITTEES:

1972–1975	University of Canterbury representative: Long Span Bridges Research Coordinating Subcommittee of TC4 of the Road Research Unit of the National Roads Board.
1976–1977	University of Canterbury representative: Bridge Dynamics Research Coordinating Subcommittee of TC4 of the Road Research Unit of the National Roads Board.
1976–1977	Member Organising Committee, 1977 Annual Conference of the New Zealand Institution of Engineers, February 1977 – Convener Registration Subcommittee.
1978–1980	Member, Discussion Group on Seismic Design of Bridges, The New Zealand National Society for Earthquake Engineering. On Subcommittee on Soil-Structure Interaction, Special Structures.
1978–1988	University of Canterbury representative: Substructures Research Coordinating Subcommittee of TC4 of the Road Research Unit of the National Roads Board.
1986	Member, Discussion Group on Structures of Limited Ductility. The New Zealand National Society for Earthquake Engineering.
1991–1994	Member, Study Group on Dynamic Analysis. The New Zealand National Society for Earthquake Engineering.
1997	Standards New Zealand. Joint Earthquake Loading Standard, Sub-committee BD/6/4 Earthquake Loads.

INVITED LECTURES:

Earthquake Engineering. Address given to Norwegian Society for Earthquake Engineering, Oslo, 26 April 1989.

Earthquake Engineering in New Zealand. Lecture given, Division of Structural Mechanics, Luleå University of Technology, Sweden, 12 November 1989.

Three Decades of Earthquake Engineering Research at the University of Canterbury. Invited by The New Zealand Society for Earthquake Engineering to give an address at the 8th National Conference on Earthquake Engineering, held in San Francisco, 17-22 April 2006 to celebrate the 10th anniversary of the 1906 San Francisco earthquake.

KEYNOTE ADDRESSES:

Control of Deformation Under Seismic Excitation. *Fifth World Conf. Habitat and the High Rise.* Council on Tall Buildings and Urban Habitat, Amsterdam, 14–19 May 1995:

Structural Engineering in New Zealand – Trends in Practice and Recent and Current Research**,** *XXXIII Jornadas Sudamericanas di Ingieneria Estructural,* Santiago, Chile, 26-29 May 2008.

The Darfield and Christchurch Earthquakes. *International Symposium on Stron-motion Earthquake Effects, ISSEE2011*, University of Iceland, Reykjavik, Iceland. April 29, 2011.

PROFESSIONAL SEMINARS:

Seminar on Bridge Design and Research, *Elastic Soil-Structure Interaction* (with P.J. Moss). National Roads Board, Bridges and Structures, Wellington, October 3–4, 1974.

Civil Engineering – A Survey of New Developments. A joint seminar by NZIE and University of Canterbury Extension Studies, October 11–13, 1978. *Computers II – Structural Uses.*

Earthquake Engineering - Seminar presented at Det Norske Veritas, Høvik, Oslo, Norway, June 28–29, 1982.

Earthquake Engineering Seminar. (with Rajesh Dhakal) IPENZ one day course on earthquake engineering..

22nd August 2011: New Plymouth
23rd August 2011: Taupo
24th August 2011: Dunedin
29th August 2011: Auckland
30th August 2011: Wellington
31st August 2011: Christchurch
15th November 2011: Auckland
16th November 2011: Wellington
18th November 2011: Christchurch

ENGINEERING STRUCTURES:

In 1983 was invited to join the International Editorial Board for the new journal, *Engineering Structures,* published by IPC Science and Technology Press Ltd, Guildford, England. Resigned from Editorial Board 2000.

RESEARCH COUNCIL of NORWAY

Since 2004 I have been asked by the Research Council of Norway to review some of the applications from researchers in Norway for funding from the Research Council of Norway.

INTERNATIONAL CONFERENCE COMMITTEES:

1993	APVC '93 Asia Pacific Vibration Conference, Kitakynohu, Japan, Nov. 1993, International Steering Committee.
1996	XXV General Congress of European Seismological Commission, Reykjavik, Iceland, Sept. 9–14, 1996, International Steering Committee.
1997	APVC'97 Asia Pacific Vibration Conference, Kjongju, Korea, Nov. 1997, International Steering Committee.
1999	APVC '99 Asia Pacific Vibration Conference, Singapore, Nov. 1999, International Steering Committee.
1999	Civil and Environmental Engineering Conference New Frontiers and Challengers, Bangkok, Thailand, Nov. 1999, International Technical Committee.
2003	APVC '03 Asia Pacific Vibration Conference, Gold Coast, Australia, Nov. 2003, International Steering Committee.
2005	APVC '05 Asia Pacific Vibration Conference, Langkawi, Malaysia, Nov. 2005, International Steering Committee.
2007	APVC '07 Asia Pacific Vibration Conference, Sapporo, Japan, Aug. 2007, International Steering Committee.
2009	APVC'09 Asia Pacific Vibration Conference, Christchurch, New Zealand, Nov. 2007, Chairman, Conference Organizing Committee, International Steering Committee.

PROFESSIONAL EXPERIENCE:

1963–1964	Design engineer, Bill Lovell-Smith, Consulting Engineer, Christchurch.
	While a postgraduate student and since joining the University of Canterbury, I have carried out specialised computer consulting for:
1966–1967	State of California, Division of Water Resources, Analysis of pipe manifolds, Tehacaphi mountain pumping stations water project for Southern California (with Prof. R.W. Clough).
1966–1967	United States Steel Corporation. Analysis of ship hull vibration (with Prof. R.W. Clough)
1969	Edwards Clendon and Partners, Consulting Engineers, Wellington, to advise on choice of computer hardware and computational facilities for a proposed engineering computer bureau.
1969–1973	Engineers Computer Bureau, Wellington, structural analysis program development and modification, analysis of particularly difficult structures.
1969	A.E. Tyndall, Consulting Engineer, Christchurch. Analysis of frames for Timaru Hospital and multistorey hotel for Queenstown.
1970–1971	Marine Department. Analysis of a series of ferro-cement trawlers to facilitate a code of practice for the design of such vessels (with Dr J.C. Scrivener).
1971	Christchurch City Council, to set up traffic signal timing sequence design program on the University of Canterbury Computer.
1971	Holmes, Wood and Poole, Consulting Engineers, Christchurch, Shear wall finite element analysis for Queenstown hotel.
1972	Royds, Sutherland and McLeay, Consulting Engineers, Christchurch. Analysis of Waiau Ferry Bridge to facilitate strengthening without changing its appearance.
1972	Hardie and Anderson, Consulting Engineers, Christchurch. Floor slab analyses for Lincoln College.
1974	Edwards, Clendon and Partners. Inelastic seismic analysis of the Maui A offshore platform. Verification of ductile response characteristics. (with P.J. Moss).
1974	Dunedin City Corporation. Cumberland Street Overpass box girder bridge analysis.
1974	Beca, Carter, Hollings and Ferner, Consulting Engineers, Auckland. Seismic behaviour of North Rangitikei and Kawhatu Railway viaduct sites (with P.J. Moss).

1976–1977	Shell, BP, Todd Oil Services, New Plymouth. Pressure vessel seismic analysis. This included devising a method of analysis, justifying it against conventional "code" analyses, and training their engineers in the use of the finite element method. This eventually resulted in their purchase of a license for the computer program from the University of Canterbury.
1976	Beca, Carter, Hollings and Ferner. Review of program capability and manuals for the large EAC/EASE2 finite element program and for the analysis of new NAC hangar (Christchurch) for seismic analysis and analysis of box girder over doors.
1976	National Roads Board. Analysis of box girder bridge.
1976	Hardie and Anderson. Analysis of stainless steel standard "container" tanks (with P.J. Moss)
1977	Halliday, O'Loughlin and Associates, Consulting Engineers, Christchurch. Analysis of multi-storey bank building for Christchurch.
1977	Wilkins and Davies. Review of seismic aspects of a feasibility study for a concrete offshore production platform (with D.G. Elms).
1978	Brickell, Moss, Rankine and Hill, Consulting Engineers, Wellington. Dynamic analysis of twin tower building Britannic House.
1978	Holmes, Wood, Poole and Johnstone, Consulting Engineers, Christchurch. Establishment of suite of computer programs for analysis of multistorey framed structures.
1978–1982	Central Otago Electric Power Board. Finite element analysis of two concrete arch dams for the Teviot River hydroelectric power scheme (with P.J. Moss)
1980	Beca, Carter, Hollings and Ferner. Floor slab analyses for ANZ tower building in Wellington, (with T. Paulay).
1980	Beca, Carter, Hollings and Ferner. Todd Motors Ltd. Hyperbolic-paraboloid umbrella shell roof analyses (with M.J.N. Priestley).
1980	N.Z. National Society for Earthquake Engineering. To comment on the International Electro-Technical Commission Draft Document for N.Z. comment, guide for equivalent testing procedures (50A (Secretariat) 17 September 1979).
1981–1983	Holmes, Wood, Poole and Johnstone. Analysis of Union House, base isolated building, Auckland.

1982	SINTEF, AVD 71, Trondheim, Norway. Spherical tank support for LNG tanks. Review for Kvaerne, Moss Rosenborr F. Selmer. Joint venture tender proposal, Statoil Project E002.
1983	Edward Clendon and Partners. Analysis of hyperbolic paraboloid cooling towers.
1983	Holmes, Wood, Poole and Johnstone. Seismic analysis of silo structures.
1984	Frame, Harvey and West, Consulting Engineers, Boroko, Papua New Guinea. Dynamic analysis of a multi-storey apartment house for Port Moresby.
1984	Canterbury Frozen Meat. Dynamic analyses of stacked meat carcass pallets to investigate rocking stability during possible seismic excitation.
1984	New Zealand Electricity Department. Dynamic testing of switchgear support systems (with P.J. Moss).
1985	ACADS Melbourne. Review of proposed recommendations for standards of engineering software using FORTRAN 77.
1986	Holmes, Wood, Poole and Johnstone. Dynamic testing, laundry floor, Sunnyside Hospital.
1988	Holmes Consulting. Dynamic analyses of multi-storey buildings.
1988–1990	KRTA, Consulting Engineers, Wellington. Dynamic analyses for proposed base-isolated multi-storey structure for Prince's Wharf, Auckland.
1991	Duffill, Watts and King, Consulting Engineers, Dunedin. Review of analyses and dynamic analyses of spillway structure for Roxborough Dam.
1994–1995	Beca Carter Hollings and Ferner. Expert advisor on the dynamic analyses for the *Sky-Tower*, Auckland.
2000	AC Power Group, Consulting Engineers, Wellington. Generation of suite of synthetic earthquake accelerograms for the analysis and design of electrical equipment.
2001	NývERK, Consulting Engineers, Reykjavik, Iceland. With Ragnar Sigbjornsson of the University of Iceland. Advice on how to strengthen a 1970 lift-slab building to better resist earthquake excitation. The existing structure has poor quality welded connections between the slabs and precast wall panels and which were meant to provide resistance to lateral forces.

2010	Aurecon Ltd., Consulting Engineers, Wellington. Advice on problems in the earthquake analyses for a structure in the Hutt Valley that has to be fully operational in a 2500 return period earthquake. Under the strong horizontal shaking the base-isolated structure shows significant vertical floor accelerations as a consequence of a rocking motion on the isolation bearings and these accelerations are greater than those acceptable for the control systems in the structure.
2011	Royal Commission into the Christchurch Earthquakes - Asked to provide in-elastic response spectra for the 4th September 2010 Darfield earthquake and the 22nd February 2011 and 13th June 2011 earthquakes.
2011-2012	Dunning Thornton Consultants, Consulting Structural Engineers, Wellington, Victoria University of Wellington: Easterfield Seismic Retrofit Structural Engineering Peer Review. (With Greg MacRae)
2012	Royal Commission into the Christchurch Earthquakes - In-elastic analyses of the Hotel Grand Chancellor to investigate behavioural characteristics missed by the Expert panel and the Engineers reports on the building failure.
2012	Royal Commission into the Christchurch Earthquakes - In-elastic analyses of a six storey building with marked torsional responses.
2012	Royal Commission into the Christchurch Earthquakes - Expert witness to the Non-Linear Time-History Analyses (NLTHA) performed by consulting engineers for the Department of Building and Housing. To be presented to the Royal Commission hearings in June 2012.

Athol J. Carr

PUBLICATIONS:

1. Books (sole/joint author)

1. Bell, K., Carr, A.J. and Syvertsen, T. A. Handbook of Computer Programming, *SINTEF,* Trondheim, June 1983, ISBN 82-595-2874-6, 315p.

2 Carr, A.J. RUAUMOKO. Non-linear Dynamic Analysis of Framed Structures. *Department of Civil Engineering. University of Canterbury*.
October 2004, 3 Volumes:
Volume 1. Theory and User Guide to Associated Programs.173p.
Volume 2. User Manual for 2-Dimensional version, Ruaumoko2D. 161p.
Volume 3. User Manual for 3-Dimensional version, Ruaumoko3D. 1813p.

2. Articles/papers in refereed scholarly journals

1. Shepherd, R. *et al*. "The 1968 Inangahua Earthquake". *Bulletin of the Seismological Society of America,* v60 no5, October 1970: 1561–1605.

2. Moss, P.J. and Carr, A.J. "Aspects of the Analysis of Frame–panel Interaction". *Bulletin N.Z. Society Earthquake Engineering,* v4 no1, March 1971: 126–44

3. Carr, A.J. and Moss, P.J. "Elastic Soil Structure Interaction". *Bulletin of N.Z. Society Earthquake Engineering,* v4 no2, April 1971: 258–69.

4 Moss, P.J. and Carr, A.J. "The Use of Computers in Civil Engineering Education and Research". *N.Z. Engineering*, v27, Aug. 1972.

5. Sharpe, R.D. and Carr A.J. "The Seismic Response of Inelastic Structures". *Bulletin N.Z. Society for Earthquake Engineering.* v8 no3, September 1975: 192–203.

6. Carr, A.J. and Moss, P.J. "Maui Platform Analysis". *The Consulting Engineer*. London, August, 1976: 27–33.

7. Priestley, M.J.N., Crosbie, R.L. and Carr, A.J. "Seismic Forces in Base-Isolated Masonry Structures". *Bulletin of N.Z. National Society for Earthquake Engineering*. v10 no2, June 1977: 55–68.

8. Priestley, M.J.N., Evison, R.J. and Carr, A.J. "Seismic Response of Structures Free to Rock on Their Foundations". *Bulletin of the N.Z. National Society for Earthquake Engineering,* v11 no3, September 1978: 141–50.

9. Moss, P.J., Carr, A.J. and Cree-Brown, N.C. "Large Deflection Nonlinear Behaviour of Layered Timber Cylindrical Shells". *Proc. Struct. Div. ASCE,* October 1979: 2019–34.

10. Moss, P.J., Carr, A.J. and Cree-Brown, N.C. "Large Deflection Non-Linear Behaviour of Nailed Layered Timber Hyperbolic Paraboloid Shells". *Proc. I.C.E. Part 2*, v69, March 1980: 33–47.

11. Paulay, T., Carr, A.J. and Tompkins, D.N. "Response of Ductile Reinforced Concrete Frames Located in Zone C". *Bulletin of the N.Z. National Society for Earthquake Engineering,* v13 no3, Sept. 1980: 209–25.

12. Edmonds, F.D. *et al*. Seismic Design of Bridges – Section 4 – Bridge Foundations. *Bulletin of the N.Z. National Society for Earthquake Engineering,* v13 no3, Sept. 1980: 248–61.

13. Priestley, M.J.N., Stanford, P.R. and Carr. A.J. "Seismic Design of Bridges – Section 11 – Bridges Requiring Special Studies". *Bulletin of the N.Z. National Society for Earthquake Engineering,* v13 no4, Sept. 1980: 302–7.

14. Moss, P.J. and Carr, A.J. "The Effects of Large Displacements on the Earthquake Response of Tall Concrete Frame Structures". *Bulletin of the N.Z. Society for Earthquake Engineering,* v13 no4, Dec. 1980: 317–28.

15. Moss, P.J., Carr, A.J. and Cree-Brown, N.C. "The Influence of Planking Direction on the Behaviour of Nailed Layered Timber Hyperbolic Paraboloid Shells". *Bulletin of the International Association of Shell and Spatial Structures,* No. 75, April 1981: 19–34.

16. Moss, P.J., Carr, A.J. and Cree-Brown, N.C. "The Effects of Material Properties and Mesh Refinement on the Analysis of Timber Hyperbolic Shells". *Bulletin of the International Association of Shell and Spatial Structures,* No. 76, August 1981: 27-46.

17. Kivell, B.T. Moss, P.J. and Carr, A.J. "Hysteretic Modelling of Moment-Resisting Nailed Timber Joints". *Bulletin of the N.Z. National Society for Earthquake Engineering,* v14 no4, Dec. 1981: 233–43.

18. Kivell, B.T., Moss, P.J. and Carr, A.J. "The Cyclic Load Behaviour of Two Moment Resisting Nailed Timber Joints". *Trans. IPENZ,* v9, No. 2/CE, July 1982: 45–55.

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Research Student Supervison

Master of Engineering Students.

1. Climo, N.A. *Finite Element Modelling of Soil Continua.* 1972
2. Sinclair, P.J. *Finite Element Analysis of Steady State Seepage with a Free Surface.* 1972
3. Gormack, P.J. *Non-linear Finite Element Analysis of Shear Walls and two Dimensional Reinforced Concrete Structures.* 1974
4. Cameron, A.J. *The Response of Reinforced Concrete Bridge Piers to Seismic Motion.* (with R. Park) 1975
5. Lim, Chin Pau. *The Effects of Temperature on Reinforced Concrete Joints.* (with R. Park) 1975
6. Lindup, G.H. *Seismic Demands on Columns of Reinforced Concrete Multistorey Frames.*(with T. Paulay) 1975
7. Macdonald, A. *Non-linear Finite Element Analysis of Reinforced Concrete Structures Subject to Cyclic Loading.* 1975
8. Cree Brown, N.C. *Non-linear Finite Element Analysis of Layered Shells.* (with P. Moss) 1977
9. Evison, R.J. *Rocking Foundations.* (with N. Priestley) 1977
10. Crosbie, R.L. *Base Isolation for Brick Masonry Shear Wall Structures.* (with N. Priestley) 1977
11. Jury, R.D. *Seismic Load Demands on Columns of Reinforced Concrete Multi-storey Frames.* (with T. Paulay) 1978
12. Carter, B.H.P. *The Seismic Behaviour of Reinforced Concrete Frame-Shear Wall Structures.* (with T. Paulay) 1980
13. Crisp, D.J. *Damping Models for Inelastic Structures.* (with P. Moss) 1980
14. Tompkins, D.M. *The Seismic Response of Reinforced Concrete Multi-storey Frames.* (with T. Paulay) 1980
15. Kivell, B.T. *Hysteretic Modelling of Moment Resisting Nailed Timber Joints.* (with P. Moss) 1981
16. Goodsir, W.J. *The Response of Coupled Shear Walls and Frames.* (with T. Paulay) 1982
17. Lowe, P.J. *Non-linear Finite Element Analysis of Shell Structures.* (with P. Moss) 1982
18. Clendon, J.E. *Alternative Damping Models.* (with P. Moss) 1985
19. Smith, D.B.M. *The Response of Buildings with Coupled Shear Walls.* (with T. Paulay) 1985
20. Cooper, A.C. *Finite Element Analysis of Shell Structures Using Triangular Layered Elements.* (with P. Moss) 1986
21. Mallaly, K.W. *Gravity Dominated Reinforced Concrete Framed Buildings.* (with T. Paulay) 1986
22. Papakyriacou, M.A. *The Seismic Response of Coupled Shear walls.* (with T. Paulay) 1986
23. Tan, Fun Kwai. *Seismic Behaviour of Bridges on Isolating Bearings.* (with P. Moss) 1986
24. Turkington, D. *Seismic Design of Bridges on Lead-Rubber Bearings.* (with N. Cooke and P. Moss) 1987

25. Tjondro, J.A.	*Analytical Investigation of P-Delta Effects in Medium Height Steel Moment Resisting Frames.* (with P. Moss)	1988
26. Wijanto, L.S.	*Seismic Behaviour of Low-Rise Braced Steel Structures.* (with P. Moss)	1988
27. Chew, A.S.	*Seismic Response of Timber Structures.* (with P. Moss)	1989
28. Djaja, R.G.	*Finite Element Modelling of Fibrous Assemblies.* (with P. Moss and G. Carnaby (WRONZ))	1989
29. Lee, Poh Chuan.	*Seismic Response of Six Storey Braced Steel Frames.* (with P. Moss)	1989
30. Thomson, E.D.	*P-Delta Effects in Ductile Reinforced Concrete Frames Under Seismic Loading.* (with P. Moss)	1991
31. Sinclair, K.M.	*The Response of Multi-storey Frames to Seismic Pounding.* (with P. Moss)	1993
32. Stewart, N.L.	*An Analytical Study of the Seismic Response of Reinforced Concrete Frame-Shear Wall Structures.* (with D. Bull)	1995
33. Cho, J.H.	*Non-linear Geometric Effects in Framed Structures.* (with P. Moss)	1997
34. Rashidi, A.B.	*The Behaviour of Imperfect Shear Walls Under Earthquake Loading.* (with P. Moss)	1997
35. Chambers, D.J.	*A Distributed Spring Soil Model for Dynamic Soil-Structure Interaction Analysis.*	1998
36. Kao, Grace C.	*Design and Shaking Table Tests of a Four Storey Miniature Structure Built With Replaceable Plastic Hinges.*(with J. Restrepo)	1998
37. Pradono, M.H.	*Dynamic Amplification of Static Design Forces at Flexural Overstrength of Coupled Wall Structures.* (with P. Moss)	1998
38. Bishay-Girges, N.W.	*Damping Models for Inelastic Structures.* (with P Moss)	1999
39. Dong, Ping.	*Effects of Different Choice of Hysteresis Models on the Response of Framed Structures of Reinforced Concrete Subjected to Earthquake Excitation.* (with P. Moss)	1999
40. Chey, Min Ho.	*Parametric Control of Structures Using a Tuned Mass Damper System Under Earthquake Excitation.* (with P. Moss)	2000
41. Hou, Ming.	*Dynamic Behaviour of Bridges with Energy Absorbing Bearings.* (with P. Moss)	2000
42. Thompson, N.S.	*Curved Reinforced Concrete Shells.* (with P. Moss)	2000
43. Beyer, K	*Re-examination of the Seismic Behaviour of Ductile Coupled Walls* (with T. Paulay and H.Bachmann)	2001
44. Chu, K.H.	*Soil-Structure Interaction of Masonry In-filled Frames with Openings.*	2002
45. Robertson, K	*Probabilistic Seismic Design and assessment Methodologies for the New Generation of damage Resistant Structures.* (with J Mander)	2006
46. Hertanto, Eric	*Seismic Assessment of pre-1970s Reinforced Concrete Structures* (with S. Pampanin)	2006
47. Alistair Waller.	*The Effect of Mass Irregularity on the Response of Drift and Acceleration for Isolated and Un-isolated Structures,* (with Bruce Deam)	2010

Doctor of Philosophy Students.

1. Sharpe, R.D. *The Seismic Response of Inelastic Structures.* 1974
2. Moore, T.A. *Finite Element Analysis of Box-Girder Bridges.* 1975
3. Wilby, G.K. *Response of Concrete Structures to Seismic Motion.* (with R. Park) 1975
4. Taylor, R.G. *The Non-linear Seismic Response of Tall Shear Wall Structures.* (with T. Paulay) 1977
5. van Luijk, C.J. *Structural Analysis of Wool Yarns.* (with P. Moss and G. Carnaby(WRONZ)) 1981
6. Goodsir, W.J. *The Design of Coupled Frame-wall Structures for Seismic Actions.* (with T. Paulay) 1985
7. Whittaker, D. *Seismic Performance of Offshore Concrete Gravity Platforms.* (with R. Park) 1987
8. Andriono, T. *Seismic Resistant Design of Base Isolated Multi-storey Structures.* 1989
9. MacRae, G.A. *The Seismic Response of Steel Frames.* (with W. Walpole) 1989
10. Zhao, X. *Seismic Soil Structure Interaction.* (with P. Moss) 1990
11. Mori, A. *Investigation of the Behaviour of Seismic Isolation Systems for Bridges.* (with P. Moss) 1993
12. Munro, W.A. *Finite Elements for Yarn Mechanics.* (with P. Moss and G. Carnaby(WRONZ)) 1995
13. Widodo. *Rocking of Multi-storey Buildings.* 1995
14. Crisafulli, F.J. *Seismic Behaviour of Reinforced Concrete Structures with Masonry Infills.* (with R. Park) 1997
15. Charng, P.S. *Base isolation for Multi-storey Building Structures.* (with P. Moss) 1998
16. Rahman, A.M. *Seismic Pounding of Adjacent Multiple-Storey Buildings Considering Soil-Structure Interaction and Through-Soil Coupling.* (with P. Moss) 1998
17. Xi Lin. *Analysis and Design of Building Structures with Supplemental Lead Dampers Under Earthquake and Wind Loads.* (with P. Moss) 1999
18. Satyarno, I. *Adaptive Pushover Analysis for the Seismic Assessment of Older Reinforced Concrete Buildings.* (with J. Restrepo) 2000
19. Zhang, J.J. *Seismic Soil-Structure Interaction in the Time-Domain.* (with P. Moss) 2000
20. Liu, Aizhen. *Seismic Assessment and Retrofit of Pre-1970s Reinforced Concrete Frame Structures.* (with R. Park) 2002
21. Bishay-Girges, Nagui. *Seismic Protection of Structures Using Passive Control Systems.* (with P. Moss) 2004
22. Castillo-Barahona, Rolando. *Torsional Response of Ductile Structures.* (with T. Paulay) 2004
23. Dong, Ping. *Effect of Different Choice of Hysteresis Models and Damage Models on Seismic Damage Analysis for Reinforced Concrete Ductile Framed Structures.* (with P Moss) 2003
24. Saunders, Dean. *Refined Pushover Analysis for the Assessment of Reinforced Concrete Frames Inelastic Performance Under Seismic Attack and Investigation into the Performance of the Structural Mechanisms of the Civil Office Building, Christchurch.* (with J. Mander) 2005

25. Zaghlool, Baher. *Energy Modelling and Behaviour of Multi-storey Structures Under Concurrent Orthogonal Seismic Excitation.*
(with J. Mander) 2007
26. Chey, Min Ho. *Inelastic 3-Dimensional Analysis of Structures with Semi-active Non-linear Tuned Mass Dampers Under Earthquake Excitations.*
(with J. Mander and G.Chase) 2007
27. Wijanto, L. Sugeng. *Seismic Assessment and Performance of Historical Un-reinforced Masonry Buildings Built in Indonesia.*
(with J. Restrepo (UCSD)) 2008
28. Franco-Anaya, Roberto *Use of Semi-Active Devices to Control Deformation of Structures subjected to Seismic Excitation.*
(with J. Mander and G. Chase) 2008
29. Peng,Brian H.H. *Seismic Performance Assessment of Reinforced Concrete Buildings with Precast Concrete Floor Systems.*
(with R. Dhakal, R.Fenwich, D.Bull) 2009
30. Alejandro Amaris Mesa. *Developments of Advanced Solutions for Seismic Resisting Precast Concrete Frames.*
(with Stefano Pampanin, Des Bull and Alessandro Palermo) 2010
31. Kam Weng Yuen. *The development of selective retrofit strategy and techniques for reinforced concrete structures within a performance-based approach*
(with Stefano Pampanin and Des Bull) 2010
32. Iqbal. Asif "*Seismic Response and Design of Subassemblies for MultiStorey Prestressed Timber Buildings"*
(with Stefano Pampanin) 2011
33. Umat Akguzel *Seismic Assessment and Retrofit of Pre-1970s Reinforced Concrete Structures with Masonry Infill*
(with S. Pampanin and Constantin Christopulos (University of Toronto)) 2011
34. Debra Gardiner *Development of design recommendtions for the internal forces in concrete floor diaphragms*
(with Des Bull) 2012

BE(Hons) Research Project Students

1. Hills, Ian and Greenfield, Richard. *Structural Behaviour of Centreboard Yachts*. 1995 (with John Dean)
2. Williams, Alan. *Behaviour of Reinforced Concrete Beam-Column Joints and Their Effects of the Behaviour of Frames under Seismic Excitation.* 2005 (with Stefano Pampanin)
3. Gardiner, Debra. *The Forces in Floor Diaphragms under Earthquake Excitation.* 2006 (with Des. Bull)

Current Doctor of Philosophy Students.

1. K.Masoud.Moghaddasi *Performance-based seismic design and assessment of structures including SSI effects*
   (with Misko Cubrinovski, Stefano Pamapanin and Goeffrey.Chase)
2. Greg Cole *Quantifying the effects of building pounding*
   (with Rajesh Dhakal and Des Bull)
3. Patricio Quintana-Gallo *Performance-based retrofit and assessment of under-designed reinforced concrete frame buildings - a dynamic investigation*
   (with S.Pampanin and P.Bonneli (University Santa Maria, Valparaiso Chile))
4. Arun Puthanpurayil. *Development of optimal performance-based passive building control strategies for earthquakes.*
   (with Rajesh Dhakal and Greg Macrae)
5. Simona Giorgini *Non-linear Dynamic Soil-Foundation-Structure Interaction*
   (with Misko Cubrinovski and Stefano Pampanin)