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Appendices

Appendix 2A: Priority Factors

2A.1 Occupancy Classification

The occupancy classification (OC) should be determined by considering both the occupant load (O_L) and the intensity of occupation (O_I).

O_L = The maximum number of people exposed to risk during the normal functioning of the building.

O_I = $\dfrac{\text{Occupant Load}}{\text{Gross Floor Area (100s of m}^2\text{)}} \times \dfrac{\text{Weekly hours of normal occupancy}}{40}$

The occupancy classification is determined as follows:

- For essential buildings: $OC = 1$
- For all other buildings: OC is determined from Figure 2A.1.

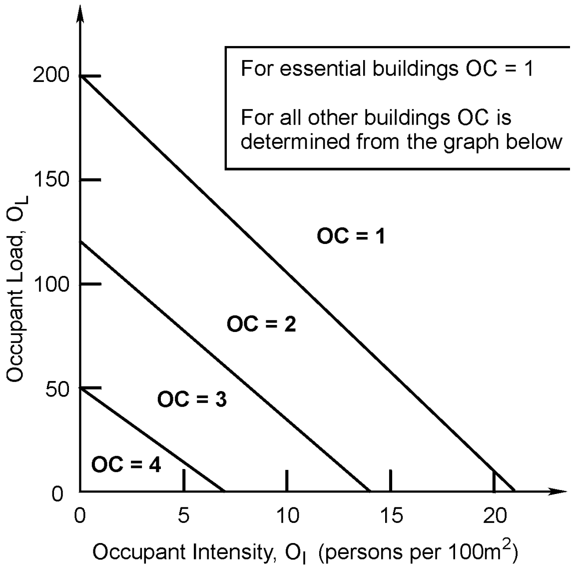


Figure 2A.1: Occupancy Classifications (non-essential buildings)

2A. 2 Risk to People Outside the Building

The risk to people outside the building is a function of building location, accessibility and use. The intention of this factor is to recognise that larger numbers of people, other than the occupants, may be at risk in the event that parts of a building may collapse during an earthquake. Examples are:

- high risk: inner city retail shopping areas adjacent to busy footpath, exitways, malls and public places
- medium risk: inner or outer city commercial business areas with street frontage
- low risk: outer city/suburb industrial warehouse areas not frequented by pedestrians.

2A.3 Prioritising for Detailed Evaluation

- The following relationship may be used to assist with prioritising buildings that have undergone the IEP procedure.
- The procedure should not be used for comparison of buildings in different earthquake zones, and is intended for use with buildings identified as potentially not safe in an earthquake.

$$PS = \frac{\%NBS}{(K1 \times K2)}$$

where:

PS = Prioritised Structural Performance Score
$\%NBS$ = Percentage of New Building Standard from the IEP analysis
$K1$, $K2$ = Factors from Table 2A.1

Table 2A.1: Modification factors *K*1 and *K*2

Description	Classification	Factor
Occupancy Classification (refer Figure 2.5)	1	$K1$ = 1.2
	2	1.0
	3	0.9
	4	0.8
Risk to people outside (refer commentary below)	High	$K2$ = 1.1
	Medium	1.0
	Low	0.9

2A.4 Timetable for Improvement

Time to complete performance improvement (T_c) to be:

$$T_c = \frac{\%NBS}{5 \times K1 \times K2}$$

where:

$1.0 < T_c < 20$ (years)
$\%NBS$ = Percentage of New Building Standard
$K1$, $K2$ = As above

Note:

- The *%NBS* is the earthquake performance of the building compared with requirements for a new building, expressed as a percentage. If a detailed evaluation of the building is available, this should be used to determine the *%NBS*. Otherwise, at the territorial authority's discretion, the IEP score may be used.
- For a change of use application, the work is to proceed immediately as part of the consent.

Appendix 2B: Factors to be considered when evaluating “as near as is reasonably practicable to that of a new building”

The following factors should be considered by TAs and designers/assessors when evaluating *“as near as is reasonably practicable to that of a new building”*;

a) *The size of the building;*
b) *The complexity of the building;*
c) *The location of the building in relation to other building, public spaces, and natural hazards;*
d) *The intended life of the building;*
e) *How often people visit the building;*
f) *How many people spend time in or in the vicinity of the building;*
g) *The intended use of the buildings, including any special traditional and cultural aspects of the intended use;*
h) *The expected useful life of the building and any prolongation of that life;*
i) *The reasonable practicality of any work concerned;*
j) *In the case of an existing building, any special historical or cultural value of that building;*
k) *Any other matter that the territorial authority considers to be relevant.”*

Appendix 3A: Typical ($\%NBS$)$_b$ values for Wellington, Auckland and Christchurch

Table 3A.1: ($\%NBS$)$_b$ Wellington, μ = 1.25, Importance Levels 2, 3 and 4

WELLINGTON

μ	1.25	T = 0.4s	T = 1.0s	T = 2.0s
IL	2	Short	Interm	Long
Code Era	Soil Type	(%NBS)b	(%NBS)b	(%NBS)b
WGTN 1931-1935	A or B Rock	17	38	62
	C Shallow Soil	14	30	49
	D Soft Soil	11	19	30
	E Very Soft Soil	12	12	19
1935-1965	A or B Rock	14	30	49
	C Shallow Soil	11	24	39
	D Soft Soil	9	15	24
	E Very Soft Soil	10	10	16
1965-1976	A or B Rock	29	40	52
	C Shallow Soil	23	32	41
	D Soft Soil	18	20	25
	E Very Soft Soil	20	13	16
1976-1992	A or B Rock	86	108	138
	C Shallow Soil	69	86	110
	D Soft Soil	54	59	74
	E Very Soft Soil	54	38	48
1976-1984 Reinforced Concrete	A or B Rock	103	129	165
	C Shallow Soil	82	104	132
	D Soft Soil	65	70	89
	E Very Soft Soil	65	45	58
1992-2004	A or B Rock	78	75	63
	C Shallow Soil	73	91	73
	D Soft Soil	72	84	69
	E Very Soft Soil	72	54	44

WELLINGTON

μ	1.25	T = 0.4s	T = 1.0s	T = 2.0s
IL	3	Short	Interm	Long
Code Era	Soil Type	(%NBS)b	(%NBS)b	(%NBS)b
WGTN 1931-1935	A or B Rock	14	30	49
	C Shallow Soil	11	24	39
	D Soft Soil	9	15	24
	E Very Soft Soil	10	10	16
1935-1965	A or B Rock	14	30	49
	C Shallow Soil	11	24	39
	D Soft Soil	9	15	24
	E Very Soft Soil	10	10	16
1965-1976	A or B Rock	30	41	53
	C Shallow Soil	24	33	42
	D Soft Soil	19	20	26
	E Very Soft Soil	21	13	17
1976-1992	A or B Rock	94	119	152
	C Shallow Soil	75	95	121
	D Soft Soil	59	64	82
	E Very Soft Soil	59	42	53
1976-1984 Reinforced Concrete	A or B Rock	113	142	182
	C Shallow Soil	90	114	145
	D Soft Soil	71	77	98
	E Very Soft Soil	71	50	63
1992-2004	A or B Rock	70	68	56
	C Shallow Soil	66	82	66
	D Soft Soil	65	76	62
	E Very Soft Soil	65	49	40

WELLINGTON

μ	1.25	T = 0.4s	T = 1.0s	T = 2.0s
IL	4	Short	Interm	Long
Code Era	Soil Type	(%NBS)b	(%NBS)b	(%NBS)b
WGTN 1931-1935	A or B Rock	10	21	34
	C Shallow Soil	8	17	27
	D Soft Soil	6	10	17
	E Very Soft Soil	7	7	11
1935-1965	A or B Rock	10	21	34
	C Shallow Soil	8	17	27
	D Soft Soil	6	10	17
	E Very Soft Soil	7	7	11
1965-1976	A or B Rock	22	30	38
	C Shallow Soil	17	24	30
	D Soft Soil	14	15	19
	E Very Soft Soil	15	9	12
1976-1992	A or B Rock	86	108	138
	C Shallow Soil	69	86	110
	D Soft Soil	54	59	74
	E Very Soft Soil	54	38	48
1976-1984 Reinforced Concrete	A or B Rock	103	129	165
	C Shallow Soil	82	104	132
	D Soft Soil	65	70	89
	E Very Soft Soil	65	45	58
1992-2004	A or B Rock	55	53	44
	C Shallow Soil	51	64	51
	D Soft Soil	51	59	48
	E Very Soft Soil	51	38	31

Table 3A.2: $(\%NBS)_b$ Wellington, μ = 2, Importance Levels 2, 3 and 4

WELLINGTON

μ IL	2 2	T = 0.4s Short	T = 1.0s Interm	T = 2.0s Long
Code Era	Soil Type	(%NBS)b	(%NBS)b	(%NBS)b
1935-1965	A or B Rock	25	64	104
	C Shallow Soil	20	51	83
	D Soft Soil	16	32	51
	E Very Soft Soil	17	20	33
1965-1976	A or B Rock	53	85	55
	C Shallow Soil	43	68	44
	D Soft Soil	34	42	27
	E Very Soft Soil	36	27	17
1976-1992	A or B Rock	113	143	182
	C Shallow Soil	91	114	145
	D Soft Soil	71	78	98
	E Very Soft Soil	71	50	63
1976-1984 Reinforced Concrete	A or B Rock	136	171	219
	C Shallow Soil	109	137	174
	D Soft Soil	86	93	118
	E Very Soft Soil	86	60	76
1992-2004	A or B Rock	103	100	83
	C Shallow Soil	97	121	97
	D Soft Soil	96	111	91
	E Very Soft Soil	96	72	59

WELLINGTON

μ IL	2 3	T = 0.4s Short	T = 1.0s Interm	T = 2.0s Long
Code Era	Soil Type	(%NBS)b	(%NBS)b	(%NBS)b
1935-1965	A or B Rock	25	64	104
	C Shallow Soil	20	51	83
	D Soft Soil	16	32	51
	E Very Soft Soil	17	20	33
1965-1976	A or B Rock	54	87	56
	C Shallow Soil	44	70	44
	D Soft Soil	34	43	27
	E Very Soft Soil	37	28	18
1976-1992	A or B Rock	125	157	201
	C Shallow Soil	100	126	160
	D Soft Soil	79	85	108
	E Very Soft Soil	79	55	70
1976-1984 Reinforced Concrete	A or B Rock	150	188	241
	C Shallow Soil	120	151	191
	D Soft Soil	94	102	130
	E Very Soft Soil	94	66	84
1992-2004	A or B Rock	93	90	75
	C Shallow Soil	87	109	87
	D Soft Soil	86	100	82
	E Very Soft Soil	86	65	53

WELLINGTON

μ IL	2 4	T = 0.4s Short	T = 1.0s Interm	T = 2.0s Long
Code Era	Soil Type	(%NBS)b	(%NBS)b	(%NBS)b
1935-1965	A or B Rock	18	45	73
	C Shallow Soil	14	36	58
	D Soft Soil	11	22	36
	E Very Soft Soil	12	14	23
1965-1976	A or B Rock	39	63	40
	C Shallow Soil	32	51	32
	D Soft Soil	25	31	20
	E Very Soft Soil	27	20	13
1976-1992	A or B Rock	113	143	182
	C Shallow Soil	91	114	145
	D Soft Soil	71	78	98
	E Very Soft Soil	71	50	63
1976-1984 Reinforced Concrete	A or B Rock	136	171	219
	C Shallow Soil	109	137	174
	D Soft Soil	86	93	118
	E Very Soft Soil	86	60	76
1992-2004	A or B Rock	72	70	58
	C Shallow Soil	68	85	68
	D Soft Soil	67	78	64
	E Very Soft Soil	67	50	41

Table 3A.3: $(\%NBS)_b$ Wellington, μ = 3, Importance Levels 2, 3 and 4

WELLINGTON

μ	3	T = 0.4s	T = 1.0s	T = 2.0s
IL	2	Short	Interm	Long
Code Era	Soil Type	(%NBS)b	(%NBS)b	(%NBS)b
1976-1992	A or B Rock	113	171	219
	C Shallow Soil	91	137	174
	D Soft Soil	71	93	118
	E Very Soft Soil	71	60	76
1976-1984 Reinforced Concrete	A or B Rock	136	205	263
	C Shallow Soil	109	164	209
	D Soft Soil	86	112	142
	E Very Soft Soil	86	72	91
1992-2004	A or B Rock	103	100	83
	C Shallow Soil	97	121	97
	D Soft Soil	96	111	91
	E Very Soft Soil	96	72	59

WELLINGTON

μ	3	T = 0.4s	T = 1.0s	T = 2.0s
IL	3	Short	Interm	Long
Code Era	Soil Type	(%NBS)b	(%NBS)b	(%NBS)b
1976-1992	A or B Rock	125	188	241
	C Shallow Soil	100	151	191
	D Soft Soil	79	102	130
	E Very Soft Soil	79	66	84
1976-1984 Reinforced Concrete	A or B Rock	150	226	289
	C Shallow Soil	120	181	230
	D Soft Soil	94	123	156
	E Very Soft Soil	94	79	100
1992-2004	A or B Rock	93	90	75
	C Shallow Soil	87	109	87
	D Soft Soil	86	100	82
	E Very Soft Soil	86	65	53

WELLINGTON

μ	3	T = 0.4s	T = 1.0s	T = 2.0s
IL	4	Short	Interm	Long
Code Era	Soil Type	(%NBS)b	(%NBS)b	(%NBS)b
1976-1992	A or B Rock	113	171	219
	C Shallow Soil	91	137	174
	D Soft Soil	71	93	118
	E Very Soft Soil	71	60	76
1976-1984 Reinforced Concrete	A or B Rock	136	205	263
	C Shallow Soil	109	164	209
	D Soft Soil	86	112	142
	E Very Soft Soil	86	72	91
1992-2004	A or B Rock	72	70	58
	C Shallow Soil	68	85	68
	D Soft Soil	67	78	64
	E Very Soft Soil	67	50	41

Table 3A.4: $(\%NBS)_b$ Auckland, μ = 1.25, Importance Levels 2, 3 and 4

AUCKLAND

μ	1.25	T = 0.4s	T = 1.0s	T = 2.0s
IL	2	Short	Interm	Long
Code Era	Soil Type	(%NBS)b	(%NBS)b	(%NBS)b
1935-1965	A or B Rock	43	94	151
	C Shallow Soil	34	75	120
	D Soft Soil	27	46	74
	E Very Soft Soil	30	30	48
1965-1976	A or B Rock	60	83	106
	C Shallow Soil	48	66	84
	D Soft Soil	38	41	52
	E Very Soft Soil	42	26	33
1976-1992	A or B Rock	133	167	213
	C Shallow Soil	106	134	170
	D Soft Soil	84	91	115
	E Very Soft Soil	84	59	74
1976-1984 Reinforced Concrete	A or B Rock	159	200	256
	C Shallow Soil	127	160	204
	D Soft Soil	100	109	138
	E Very Soft Soil	100	70	89
1992-2004	A or B Rock	120	116	97
	C Shallow Soil	113	141	113
	D Soft Soil	111	130	106
	E Very Soft Soil	111	84	68

AUCKLAND

μ	1.25	T = 0.4s	T = 1.0s	T = 2.0s
IL	3	Short	Interm	Long
Code Era	Soil Type	(%NBS)b	(%NBS)b	(%NBS)b
1935-1965	A or B Rock	43	94	151
	C Shallow Soil	34	75	120
	D Soft Soil	27	46	74
	E Very Soft Soil	30	30	48
1965-1976	A or B Rock	61	84	108
	C Shallow Soil	49	67	86
	D Soft Soil	39	41	53
	E Very Soft Soil	42	27	34
1976-1992	A or B Rock	146	184	235
	C Shallow Soil	117	147	187
	D Soft Soil	92	100	127
	E Very Soft Soil	92	64	82
1976-1984 Reinforced Concrete	A or B Rock	175	220	282
	C Shallow Soil	140	176	224
	D Soft Soil	110	120	152
	E Very Soft Soil	110	77	98
1992-2004	A or B Rock	108	104	87
	C Shallow Soil	102	127	102
	D Soft Soil	100	117	95
	E Very Soft Soil	100	75	62

AUCKLAND

μ	1.25	T = 0.4s	T = 1.0s	T = 2.0s
IL	4	Short	Interm	Long
Code Era	Soil Type	(%NBS)b	(%NBS)b	(%NBS)b
1935-1965	A or B Rock	30	65	106
	C Shallow Soil	24	52	84
	D Soft Soil	19	32	52
	E Very Soft Soil	21	21	34
1965-1976	A or B Rock	44	61	78
	C Shallow Soil	36	49	62
	D Soft Soil	28	30	38
	E Very Soft Soil	31	19	25
1976-1992	A or B Rock	133	167	213
	C Shallow Soil	106	134	170
	D Soft Soil	84	91	115
	E Very Soft Soil	84	59	74
1976-1984 Reinforced Concrete	A or B Rock	159	200	256
	C Shallow Soil	127	160	204
	D Soft Soil	100	109	138
	E Very Soft Soil	100	70	89
1992-2004	A or B Rock	84	81	68
	C Shallow Soil	79	98	79
	D Soft Soil	78	91	74
	E Very Soft Soil	78	59	48

Table 3A.5: $(\%NBS)_b$ Auckland, $\mu = 2$, Importance Levels 2, 3 and 4

AUCKLAND

μ	2	T = 0.4s	T = 1.0s	T = 2.0s
IL	2	Short	Interm	Long
Code Era	Soil Type	(%NBS)b	(%NBS)b	(%NBS)b
1935-1965	A or B Rock	78	198	320
	C Shallow Soil	63	158	255
	D Soft Soil	49	97	157
	E Very Soft Soil	53	63	101
1965-1976	A or B Rock	109	175	112
	C Shallow Soil	88	140	89
	D Soft Soil	69	86	55
	E Very Soft Soil	75	56	35
1976-1992	A or B Rock	175	221	282
	C Shallow Soil	140	177	224
	D Soft Soil	111	120	152
	E Very Soft Soil	111	77	98
1976-1984 Reinforced Concrete	A or B Rock	211	265	339
	C Shallow Soil	168	212	269
	D Soft Soil	133	144	183
	E Very Soft Soil	133	93	118
1992-2004	A or B Rock	159	153	128
	C Shallow Soil	150	186	150
	D Soft Soil	147	172	140
	E Very Soft Soil	147	111	90

AUCKLAND

μ	2	T = 0.4s	T = 1.0s	T = 2.0s
IL	3	Short	Interm	Long
Code Era	Soil Type	(%NBS)b	(%NBS)b	(%NBS)b
1935-1965	A or B Rock	78	198	320
	C Shallow Soil	63	158	255
	D Soft Soil	49	97	157
	E Very Soft Soil	53	63	101
1965-1976	A or B Rock	112	179	114
	C Shallow Soil	89	143	91
	D Soft Soil	70	88	56
	E Very Soft Soil	76	57	36
1976-1992	A or B Rock	193	243	310
	C Shallow Soil	154	194	247
	D Soft Soil	122	132	168
	E Very Soft Soil	122	85	108
1976-1984 Reinforced Concrete	A or B Rock	232	292	373
	C Shallow Soil	185	233	296
	D Soft Soil	146	159	201
	E Very Soft Soil	146	102	130
1992-2004	A or B Rock	143	138	115
	C Shallow Soil	135	167	135
	D Soft Soil	133	154	126
	E Very Soft Soil	133	100	81

AUCKLAND

μ	2	T = 0.4s	T = 1.0s	T = 2.0s
IL	4	Short	Interm	Long
Code Era	Soil Type	(%NBS)b	(%NBS)b	(%NBS)b
1935-1965	A or B Rock	55	139	224
	C Shallow Soil	44	111	178
	D Soft Soil	35	68	110
	E Very Soft Soil	37	44	71
1965-1976	A or B Rock	81	130	83
	C Shallow Soil	65	104	66
	D Soft Soil	51	64	41
	E Very Soft Soil	55	41	26
1976-1992	A or B Rock	175	221	282
	C Shallow Soil	140	177	224
	D Soft Soil	111	120	152
	E Very Soft Soil	111	77	98
1976-1984 Reinforced Concrete	A or B Rock	211	265	339
	C Shallow Soil	168	212	269
	D Soft Soil	133	144	183
	E Very Soft Soil	133	93	118
1992-2004	A or B Rock	111	107	90
	C Shallow Soil	105	130	105
	D Soft Soil	103	120	98
	E Very Soft Soil	103	77	63

Table 3A.6: $(\%NBS)_b$ Auckland, μ = 3, Importance Levels 2, 3 and 4

AUCKLAND

μ	3	T = 0.4s	T = 1.0s	T = 2.0s
IL	2	Short	Interm	Long
Code Era	Soil Type	(%NBS)b	(%NBS)b	(%NBS)b
1976-1992	A or B Rock	263	397	508
	C Shallow Soil	210	318	404
	D Soft Soil	166	216	274
	E Very Soft Soil	166	139	177
1976-1984 Reinforced Concrete	A or B Rock	315	477	609
	C Shallow Soil	252	381	484
	D Soft Soil	199	259	329
	E Very Soft Soil	199	167	212
1992-2004	A or B Rock	159	153	128
	C Shallow Soil	150	186	150
	D Soft Soil	147	172	140
	E Very Soft Soil	147	111	90

AUCKLAND

μ	3	T = 0.4s	T = 1.0s	T = 2.0s
IL	3	Short	Interm	Long
Code Era	Soil Type	(%NBS)b	(%NBS)b	(%NBS)b
1976-1992	A or B Rock	289	437	558
	C Shallow Soil	231	350	444
	D Soft Soil	182	238	301
	E Very Soft Soil	182	153	194
1976-1984 Reinforced Concrete	A or B Rock	347	524	670
	C Shallow Soil	278	420	533
	D Soft Soil	219	285	362
	E Very Soft Soil	219	184	233
1992-2004	A or B Rock	143	138	115
	C Shallow Soil	135	167	135
	D Soft Soil	133	154	126
	E Very Soft Soil	133	100	81

AUCKLAND

μ	3	T = 0.4s	T = 1.0s	T = 2.0s
IL	4	Short	Interm	Long
Code Era	Soil Type	(%NBS)b	(%NBS)b	(%NBS)b
1976-1992	A or B Rock	263	397	508
	C Shallow Soil	210	318	404
	D Soft Soil	166	216	274
	E Very Soft Soil	166	139	177
1976-1984 Reinforced Concrete	A or B Rock	315	477	609
	C Shallow Soil	252	381	484
	D Soft Soil	199	259	329
	E Very Soft Soil	199	167	212
1992-2004	A or B Rock	111	107	90
	C Shallow Soil	105	130	105
	D Soft Soil	103	120	98
	E Very Soft Soil	103	77	63

Table 3A.7: $(\%NBS)_b$ Christchurch, μ = 1.25, Importance Levels 2, 3 and 4

CHRISTCHURCH

μ	1.25	T = 0.4s	T = 1.0s	T = 2.0s
IL	2	Short	Interm	Long
Code Era	Soil Type	(%NBS)b	(%NBS)b	(%NBS)b
1935-1965	A or B Rock	25	55	90
	C Shallow Soil	20	44	71
	D Soft Soil	16	27	44
	E Very Soft Soil	18	18	28
1965-1976	A or B Rock	43	59	75
	C Shallow Soil	34	47	60
	D Soft Soil	27	29	37
	E Very Soft Soil	29	19	24
1976-1992	A or B Rock	122	153	196
	C Shallow Soil	98	123	156
	D Soft Soil	77	83	106
	E Very Soft Soil	77	54	68
1976-1984 Reinforced Concrete	A or B Rock	146	184	235
	C Shallow Soil	117	147	187
	D Soft Soil	92	100	127
	E Very Soft Soil	92	65	82
1992-2004	A or B Rock	94	90	75
	C Shallow Soil	88	110	88
	D Soft Soil	87	101	83
	E Very Soft Soil	87	65	53

CHRISTCHURCH

μ	1.25	T = 0.4s	T = 1.0s	T = 2.0s
IL	3	Short	Interm	Long
Code Era	Soil Type	(%NBS)b	(%NBS)b	(%NBS)b
1935-1965	A or B Rock	25	55	90
	C Shallow Soil	20	44	71
	D Soft Soil	16	27	44
	E Very Soft Soil	18	18	28
1965-1976	A or B Rock	44	60	77
	C Shallow Soil	35	48	61
	D Soft Soil	27	29	38
	E Very Soft Soil	30	19	24
1976-1992	A or B Rock	134	169	216
	C Shallow Soil	107	135	172
	D Soft Soil	85	92	116
	E Very Soft Soil	85	59	75
1976-1984 Reinforced Concrete	A or B Rock	161	203	259
	C Shallow Soil	129	162	206
	D Soft Soil	101	110	140
	E Very Soft Soil	101	71	90
1992-2004	A or B Rock	84	81	68
	C Shallow Soil	79	99	79
	D Soft Soil	78	91	74
	E Very Soft Soil	78	59	48

CHRISTCHURCH

μ	1.25	T = 0.4s	T = 1.0s	T = 2.0s
IL	4	Short	Interm	Long
Code Era	Soil Type	(%NBS)b	(%NBS)b	(%NBS)b
1935-1965	A or B Rock	18	39	63
	C Shallow Soil	14	31	50
	D Soft Soil	11	19	31
	E Very Soft Soil	12	12	20
1965-1976	A or B Rock	32	43	56
	C Shallow Soil	25	35	44
	D Soft Soil	20	21	27
	E Very Soft Soil	22	14	18
1976-1992	A or B Rock	122	153	196
	C Shallow Soil	98	123	156
	D Soft Soil	77	83	106
	E Very Soft Soil	77	54	68
1976-1984 Reinforced Concrete	A or B Rock	146	184	235
	C Shallow Soil	117	147	187
	D Soft Soil	92	100	127
	E Very Soft Soil	92	65	82
1992-2004	A or B Rock	66	63	53
	C Shallow Soil	62	77	62
	D Soft Soil	61	71	58
	E Very Soft Soil	61	46	37

Table 3A.8: $(\%NBS)_b$ Christchurch, μ = 2, Importance Levels 2, 3 and 4

CHRISTCHURCH

μ	2	T = 0.4s	T = 1.0s	T = 2.0s
IL	2	Short	Interm	Long
Code Era	Soil Type	(%NBS)b	(%NBS)b	(%NBS)b
1935-1965	A or B Rock	46	117	190
	C Shallow Soil	37	94	151
	D Soft Soil	29	58	93
	E Very Soft Soil	32	37	60
1965-1976	A or B Rock	78	124	80
	C Shallow Soil	62	99	63
	D Soft Soil	49	61	39
	E Very Soft Soil	53	39	25
1976-1992	A or B Rock	161	203	259
	C Shallow Soil	129	162	206
	D Soft Soil	102	110	140
	E Very Soft Soil	102	71	90
1976-1984 Reinforced Concrete	A or B Rock	194	244	311
	C Shallow Soil	155	195	248
	D Soft Soil	122	132	168
	E Very Soft Soil	122	85	108
1992-2004	A or B Rock	124	120	100
	C Shallow Soil	117	145	117
	D Soft Soil	115	134	109
	E Very Soft Soil	115	86	70

CHRISTCHURCH

μ	2	T = 0.4s	T = 1.0s	T = 2.0s
IL	3	Short	Interm	Long
Code Era	Soil Type	(%NBS)b	(%NBS)b	(%NBS)b
1935-1965	A or B Rock	46	117	190
	C Shallow Soil	37	94	151
	D Soft Soil	29	58	93
	E Very Soft Soil	32	37	60
1965-1976	A or B Rock	79	127	81
	C Shallow Soil	63	101	65
	D Soft Soil	50	62	40
	E Very Soft Soil	54	40	26
1976-1992	A or B Rock	177	223	285
	C Shallow Soil	142	179	227
	D Soft Soil	112	121	154
	E Very Soft Soil	112	78	99
1976-1984 Reinforced Concrete	A or B Rock	213	268	342
	C Shallow Soil	170	214	272
	D Soft Soil	134	146	185
	E Very Soft Soil	134	94	119
1992-2004	A or B Rock	112	108	90
	C Shallow Soil	105	130	105
	D Soft Soil	103	120	98
	E Very Soft Soil	103	78	63

CHRISTCHURCH

μ	2	T = 0.4s	T = 1.0s	T = 2.0s
IL	4	Short	Interm	Long
Code Era	Soil Type	(%NBS)b	(%NBS)b	(%NBS)b
1935-1965	A or B Rock	32	82	133
	C Shallow Soil	26	66	106
	D Soft Soil	20	40	65
	E Very Soft Soil	22	26	42
1965-1976	A or B Rock	58	92	59
	C Shallow Soil	46	74	47
	D Soft Soil	36	45	29
	E Very Soft Soil	39	29	19
1976-1992	A or B Rock	161	203	259
	C Shallow Soil	129	162	206
	D Soft Soil	102	110	140
	E Very Soft Soil	102	71	90
1976-1984 Reinforced Concrete	A or B Rock	194	244	311
	C Shallow Soil	155	195	248
	D Soft Soil	122	132	168
	E Very Soft Soil	122	85	108
1992-2004	A or B Rock	87	84	70
	C Shallow Soil	82	101	82
	D Soft Soil	80	94	77
	E Very Soft Soil	80	60	49

Table 3A.9: $(\%NBS)_b$ Christchurch, μ = 3, Importance Levels 2, 3 and 4

CHRISTCHURCH

μ	3	T = 0.4s	T = 1.0s	T = 2.0s
IL	2	Short	Interm	Long
Code Era	Soil Type	(%NBS)b	(%NBS)b	(%NBS)b
1976-1992	A or B Rock	161	244	311
	C Shallow Soil	129	195	248
	D Soft Soil	102	132	168
	E Very Soft Soil	102	85	108
1976-1984 Reinforced Concrete	A or B Rock	194	292	374
	C Shallow Soil	155	234	297
	D Soft Soil	122	159	202
	E Very Soft Soil	122	102	130
1992-2004	A or B Rock	124	120	100
	C Shallow Soil	117	145	117
	D Soft Soil	115	134	109
	E Very Soft Soil	115	86	70

CHRISTCHURCH

μ	3	T = 0.4s	T = 1.0s	T = 2.0s
IL	3	Short	Interm	Long
Code Era	Soil Type	(%NBS)b	(%NBS)b	(%NBS)b
1976-1992	A or B Rock	177	268	342
	C Shallow Soil	142	214	272
	D Soft Soil	112	146	185
	E Very Soft Soil	112	94	119
1976-1984 Reinforced Concrete	A or B Rock	213	322	411
	C Shallow Soil	170	257	327
	D Soft Soil	134	175	222
	E Very Soft Soil	134	113	143
1992-2004	A or B Rock	112	108	90
	C Shallow Soil	105	130	105
	D Soft Soil	103	120	98
	E Very Soft Soil	103	78	63

CHRISTCHURCH

μ	3	T = 0.4s	T = 1.0s	T = 2.0s
IL	4	Short	Interm	Long
Code Era	Soil Type	(%NBS)b	(%NBS)b	(%NBS)b
1976-1992	A or B Rock	161	244	311
	C Shallow Soil	129	195	248
	D Soft Soil	102	132	168
	E Very Soft Soil	102	85	108
1976-1984 Reinforced Concrete	A or B Rock	194	292	374
	C Shallow Soil	155	234	297
	D Soft Soil	122	159	202
	E Very Soft Soil	122	102	130
1992-2004	A or B Rock	87	84	70
	C Shallow Soil	82	101	82
	D Soft Soil	80	94	77
	E Very Soft Soil	80	60	49

Appendix 3B: Assessment of Attribute Score for URM Buildings

For URM buildings built prior to 1935, the IEP can be carried out using the attribute scoring method outlined in this Appendix. The *%NBS* is then determined directly from the Total Attribute Score as described below.

The recommended procedure is;

1. Complete the attribute scoring Table 3B.1 using the guidance provided in Table 3B.2.
2. From the Total Attribute Score determine the *%NBS* from Table 3B.3

Interpolation may be used for intermediate attribute scores. While attributes may differ for each principal direction, it is the intention that the attribute score apply to the building as a whole. Given that local collapse is viewed as having the same implications as total collapse, attributes should correspond to the weakest section of a building where relevant.

The derivation of *%NBS* using the attribute scoring method outlined, assumes that all appendages likely to present a hazard have been adequately secured or measures taken to remove the risk to life, e.g. provision of appropriately designed canopies or designated “no go” zones adjacent to the building.

Table 3B.1: Assessment of Attribute Score

Item		Attribute ranking				Assessed score	
		0	1	2	3	Long	Trans
1	**Structure continuity**	Excellent	Good	Fair	Poor or none		
2	**Configuration**						
2a	Horizontal regularity	Excellent	Good	Fair	Poor		
2b	Vertical regularity	Excellent	Good	Fair	Poor		
2c	Plan regularity	Excellent	Good	Fair	Poor		
3	**Condition of structure**						
3a	Materials	Sound	Good	Fair	Poor		
3b	Cracking or movement	Not evident	Minor	Moderate	Severe		
4	**Wall (URM) proportions**						
4a	Out of plane	Good			Poor		
4b	In-plane	Excellent	Good	Fair	Poor		
5	**Diaphragms**						
5a	Coverage	Excellent	Good	Fair	Poor		
5b	Shape	Excellent	Good	Fair	Poor		
5c	Openings	None			Significant		
6	**Engineered connections between floor/roof diaphragms and walls, and walls and diaphragms** capable of spanning between	Yes			No		
7	**Foundations**	Excellent	Good	Fair	Poor		
8	**Separation from neighbouring buildings**	Adequate			Inadequate		
		Total Attribute Score;:		***for each direction***			
				for building as a whole:			

Notes:

For definition of grading under each attribute refer Table 3B.2

Table 3B.2: Definition of attributes and scores

	Atribute score[1]
Attribute Item (1): Structure continuity	
Totally un-reinforced masonry	3
Some continuity, e.g. un-reinforced masonry with a concrete band at roof *or* floor level	2
Good continuity, e.g. un-reinforced masonry with reinforced bands at *both* roof and floor levels	1
Full continuity (i.e. vertical stability not reliant on URM), e.g. reinforced concrete or steel columns and beams with un-reinforced masonry walls/infill or separate means of vertical support provided to floors and roof	0

	Atribute score[1]
Attribute Item (2): Configuration	
(a) Horizontal regularity	
Severe eccentricity, i.e. distance between storey centre of rigidity and the centre of mass for all levels above that storey, $e_d \geq 0.3\ b$ (b = longest plan dimension of building perpendicular to direction of loading)	3
$e_d \leq 0.3\ b$	2
$e_d \leq 0.2\ b$	1
Building symmetrical in both directions	0
(b) Vertical regularity	
Vertical stiffness discontinuities or discontinuities in load paths present	3
All walls continuous to foundations	2
and no soft storeys *and* minimal vertical stiffness changes	1
and no weak storeys *and* no significant mass irregularities	0
where:	
soft storey is a storey where the lateral stiffness is less than 70% of that in the storey above or less than 80% of the average stiffness of the three storeys above	
weak storey is a storey where the storey strength is less than 80% of the strength of the storey above	
a *mass irregularity* exists if the mass varies by more than 50% from one level to another (excluding light roofs which should be considered as a *part* of the building).	
(c) Plan regularity	
Sharp re-entrant corners present where the projection of the *wing* beyond the corner > 0.15 *b*	3
Regular in plan	0
Attribute Item (3): Condition of structure	
(a) Materials	
Poor, i.e. considerable deterioration, fretting or spalling, etc., or lime or other non-competent mortar or rubble wall construction	3
Fair, i.e. deterioration leading to reduced strength	2
Good, i.e. minor evidence of deterioration of materials	1
Sound	0
(b) Cracking or movement	
Severe, i.e. a considerable number of cracks or substantial movement leading to reduced strength *or* isolated large cracks	3
Moderate	2
Minor	1
Non-evident	0

<table>
<tr><th colspan="5"></th><th>Atribute score[1]</th></tr>
<tr><td colspan="5">Attribute Item (4): Wall (URM) proportions
(a) Out of plane performance
Poor,
for one storey buildings: $h_w/t \geq 14$ and $l_w/t \geq 7$
for multistorey buildings:
top storey: $h_w/t \geq 9$ and $l_w/t \geq 5$
other storeys: $h_w/t \geq 20$ and $l_w/t \geq 10$
Good (not poor)
Where h_w = height of wall between lines of positive lateral restraint
and l_w = length of wall between lines of positive lateral restraint</td><td>3

0</td></tr>
<tr><td rowspan="3">(b) In plane performance[2]</td><td colspan="4">A_p/A_w</td><td rowspan="3"></td></tr>
<tr><td rowspan="2">One storey building</td><td colspan="3">2 and 3 storey buildings</td></tr>
<tr><td>Top storey</td><td colspan="2">Other stories</td></tr>
<tr><td>Poor</td><td>≥25</td><td>≥20</td><td colspan="2">≥17</td><td>3</td></tr>
<tr><td>Fair</td><td>>20</td><td>>15</td><td colspan="2">>12</td><td>2</td></tr>
<tr><td>Good</td><td>>15</td><td>>10</td><td colspan="2">>7</td><td>1</td></tr>
<tr><td>Excellent</td><td>≤15</td><td>≤10</td><td colspan="2">≤7</td><td>0</td></tr>
<tr><td colspan="5">Where A_w = cross sectional area of all URM walls/wall sections extending over full height of storey
A_p = plan area of building above storey of interest.
For buildings of greater than 3 stories take attribute score = 3</td><td></td></tr>
<tr><td colspan="5">Attribute Item (5): Diaphragms (Refer Figure 3B.1)
(a) Coverage
No diaphragm
Full diaphragm</td><td>3
0</td></tr>
<tr><td colspan="5">To achieve an attribute ranking of 0 requires a diaphragm to be present at each level, including roof level, covering at least 90% of the building plan area at each level. Interpolation for attribute rankings of 1 and 2 may be made using judgement on the extent of coverage. Note that unless the diaphragm is continuous between walls, its effectiveness may be minimal.</td><td></td></tr>
<tr><td rowspan="2">(b) Shape</td><td colspan="4">Limiting span to depth ratios for diaphragms of different construction material</td><td rowspan="2"></td></tr>
<tr><td>Concrete</td><td>Sheet materials</td><td>T&G timber</td><td>Steel roof bracing</td></tr>
<tr><td>Poor</td><td>> 4</td><td>> 4</td><td>> 3</td><td>> 5</td><td>3</td></tr>
<tr><td>Fair</td><td>< 4</td><td>< 4</td><td>< 3</td><td>< 4</td><td>2</td></tr>
<tr><td>Good</td><td>≤ 3</td><td>≤ 3</td><td>≤ 2</td><td>≤ 3.5</td><td>1</td></tr>
<tr><td>Excellent</td><td colspan="4">As for good, but in addition the projection of “wings” beyond sharp re-entrant corners < 0.5b.</td><td>0</td></tr>
</table>

	Atribute score[1]
(c) Openings	
Significant openings	3
No significant openings	0
Interpolation for attribute rankings of 1 and 2 may be made using judgement.	
Significant openings are those which exceed the limiting values given below.	

Diaphragm construction material	Limiting values of	
	X/b	**Y/D**
Concrete	0.6	0.5
Sheet material	0.5	0.4
T&G timber	0.4	0.3

Refer Figure 3B.1 for definition of terms	
Attribute Item (7): Foundations	
Separate foundations with no interconnection *or* un-reinforced piles (unless ramification of pile failure is assessed to be minor).	3
Pads, strips or piles with some interconnection. Concrete piles to be reinforced unless ramification of pile failure is assessed to be minor.	2
Pads, strips or piles with good interconnection in both directions.	1
Concrete raft with sound connections to walls	0
Attribute Item (8): Separation	
Inadequate – no separation provided or obviously inadequate provisions for separation	3
Adequate – separation provided	0

Notes

1 Individual attribute scores may be interpolated.

2 This is an index describing the extent of brick walls within the building. The numbers given are only loosely related to lateral load capacity.

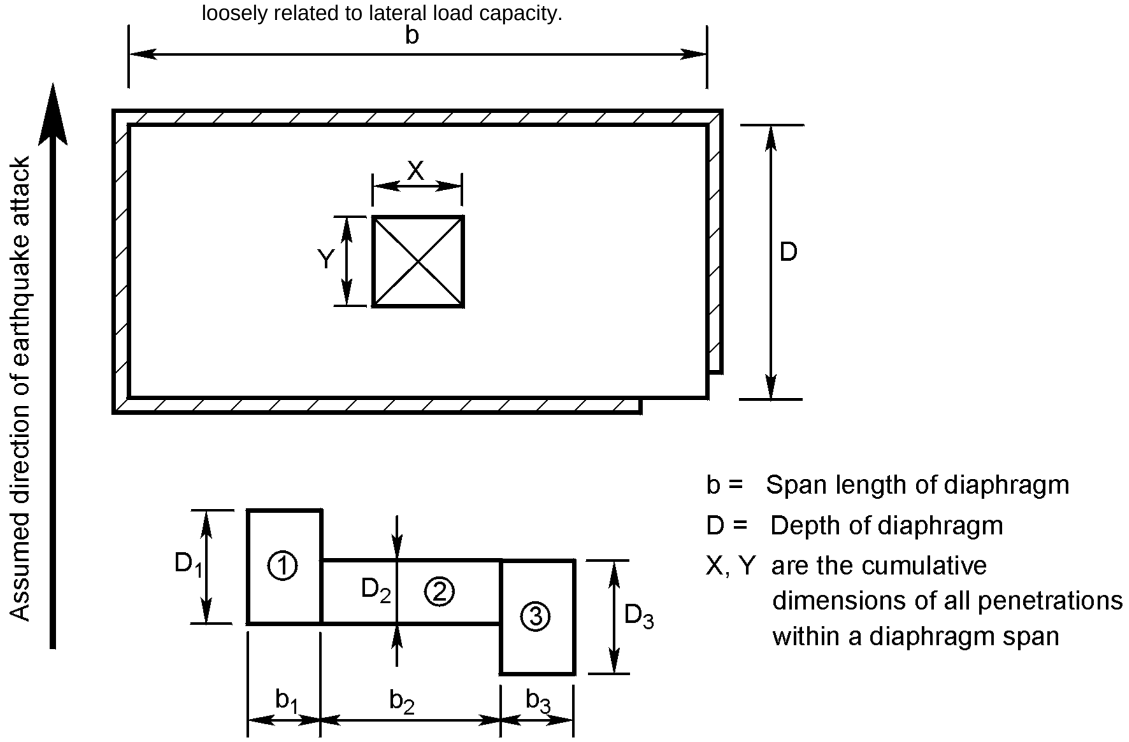


Figure 3B.1: Diaphragm parameters

Table 3B.3: Assessment of *%NBS* from Attribute Score

Item	Attribute Score	*%NBS*
1	A score of 0 for all attribute scoring items	67
2	Less than or equal to 1 for all of attribute scoring items 1 to 6 inclusive, and less than 2 for each of attribute scoring items 7 and 8	35
3	As for 2 but a score of 0 for attribute scoring item 1	40
4	5 < Total Attribute Score $\leq$ 10	20
5	10 < Total Attribute Score $\leq$ 15	15
6	15 < Total Attribute Score $\leq$ 25	10
7	Total Attribute Score > 25	5

Appendix 4A: Typical Pre-1976 Steel Building Systems Used in New Zealand

4A.1 General

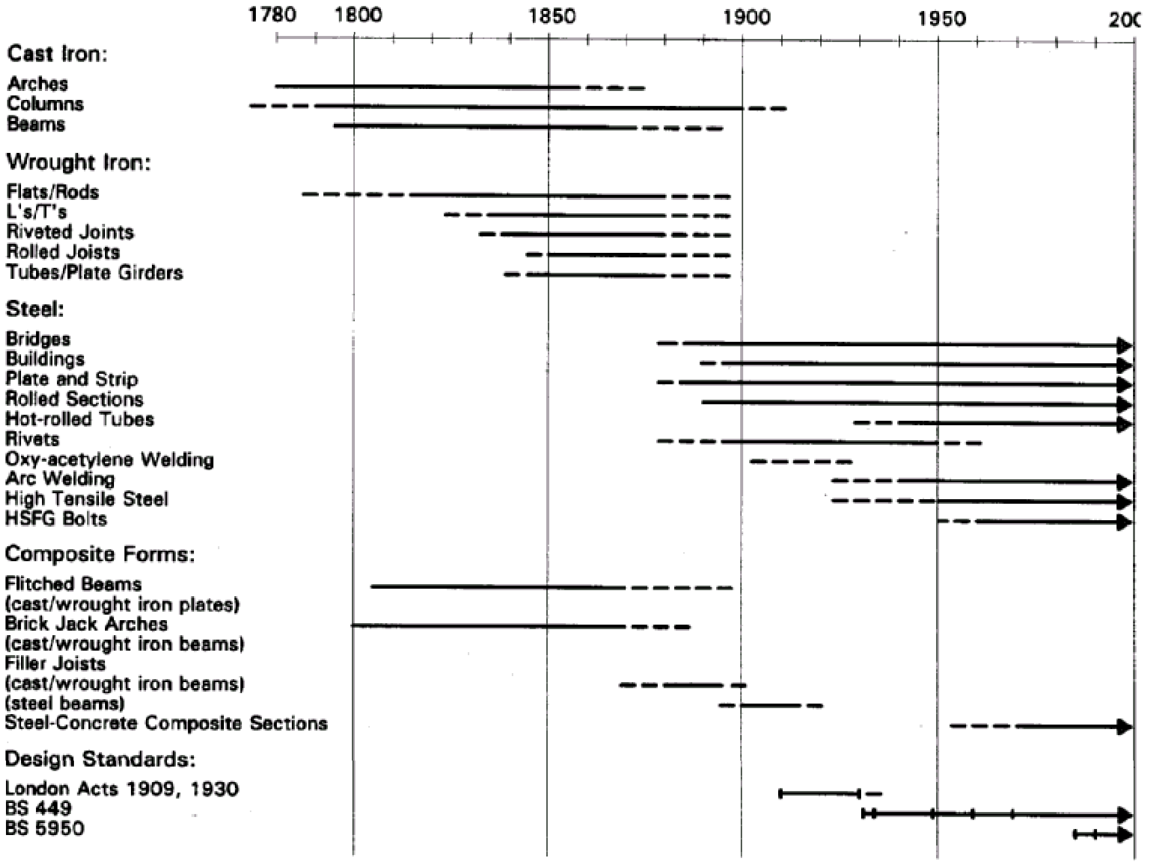
This section gives general guidance on the typical pre-1976 steel building systems used in New Zealand. The material presented is based on published material and details supplied by design engineers. It is intended that this section be extended as more buildings are assessed in the future.

4A.2 Use of iron and steel in existing buildings

Bussell (1997) gives a good summary of the use of iron and steel in structures from 1780 to the present day. In the New Zealand context, the relevant period covers ≈ 1900 to 1976. The main periods of use of the various materials is summarised in Table 4A.1.

Most ferrous material in existing New Zealand buildings will be steel, which was the preferred material for structural members in buildings from 1880 onwards. The exception is columns, especially gravity carrying columns functioning as vertical props for the floor. Cast iron was used for these through to just after 1900 and cast iron columns are found in some of the oldest New Zealand buildings. How to identify such columns is identified in Appendix 4C, section 4C.1.

Table 4A.1: Main periods of the structural use of cast iron, wrought iron and steel



Source: Bussell (1997).

4A.3 Moment-resisting frames

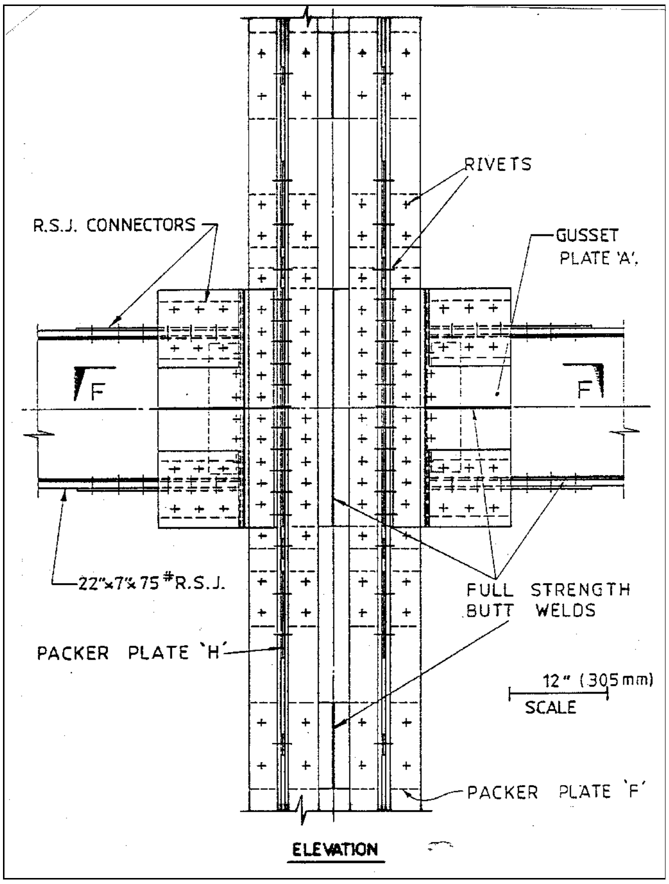
1 Beams: these were typically rolled steel joist (RSJ) sections, which are I-sections where the inside face of the flanges is not parallel to the outside face, being at a slope of around 15%. This makes the flanges thicker at the root radius than at the tips.

The flange slenderness ratios of RSJ sections are always compact when assessed to NZS 3404:1997.

These beams were typically encased in concrete for fire resistance and appearance, with this concrete containing nominal reinforcement made of plain round bars or, sometimes, chicken wire.

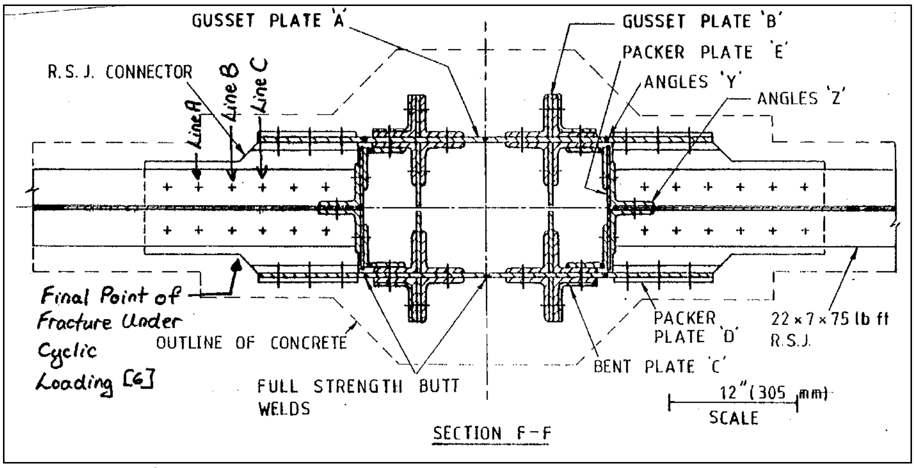
2 Columns formed from hot-rolled sections used either hot-rolled steel columns (RSCs) or box columns formed by connecting two channels, toes out, with a plate to each flange. The columns were encased in lightly reinforced concrete containing nominal reinforcement made of plain round bars.

3 Compound box columns were also formed from plates, joined by riveted or bolted angles into a box section and encased in concrete. Examples of this type of construction are shown in Figures 4A.1 and 4A.2.



Source: Wood 1987.
Note: See also Figure C9.2.

Figure 4A.1: Riveted steel fabrication details, Government Life Insurance Building, 1937



Source: Wood 1987.

Figure 4A.2: Riveted steel fabrication details, Government Life Insurance Building, 1937

4 Beam to column connections in the earlier moment frames typically comprised semi-rigid riveted or bolted connections. The RSJ beam flanges were bolted to Tee-stubs or angles bolted to the column flanges or to lengths of RSJ bolted to side extensions of the column plates. An example of the latter is shown in Figure 4A.2.

The RSJ beam web was connected by a double clip angle connection to the column flanges, again as shown in Figure 4A.2.

A simpler version of a semi-rigid connection used in some pre-1976 buildings is shown in Figure 8A.1 of Appendix 8A.

These joints generally involved the use of rivets up to 1950 and HSFG bolts after 1960, with a changeover from rivets to bolts from 1950 to 1960.

5 Beam to column connections from about 1940 onwards were also arc welded. The strength and ductility available from welded connections requires careful evaluation and attention to load path. This topic is addressed in section 8.4.2 and its importance is illustrated in Figure 4A.3. That figure, taken from a building collapsed by the Kobe earthquake of 1995, shows a failed beam to column minor axis connection, forming part of a moment-resisting frame in that direction. The beam was welded to an endplate which was fillet welded to the column flange tips. Unlike the connection detail shown in Figure 4A.2, there was no way to reliably transfer the concentrated axial force in the beam flanges, that is induced by seismic moment, from the beam into the column, with the weld between endplate and column flange unzipping under the earthquake action.

While this example is from Japan, the detail is also relevant to some early New Zealand buildings and the concept is certainly relevant.

Figure 4A.3: Failed beam to column weak axis welded connection from the 1995 Kobe earthquake

Figure 4A.4: Braced frame with light tension bracing showing damage but no collapse from the 1995 Kobe earthquake

6 Splices in columns. These typically involved riveted (pre-1950) or bolted (post-1950) steel sections, with the rivets or bolts transferring tension across the splice and compression being transferred by direct bearing. Figures 4A.1 and 4A.2 show plated box columns connected by riveted angles, while Figure 4A.3 shows a bolted UC splice detail in the column, this being a fore-runner to the bolted column splice details of HERA Report R4-100 (Hyland 1999). Such bolted splices generally perform well.

4A.4 Braced frames

For the pre-1976 buildings covered by this document, braced frames incorporating steel bracing involve concentrically braced framing (CBF), either x-braced CBFs or V-braced CBFs.

Figure 4A.4 shows an X-braced CBF with relatively light bracing and Figure 4A.5 V-braced CBF. Both are from Kobe, Japan but are similar to details used in early New Zealand buildings.

Figure 4A.5: V-braced CBF showing damage but no collapse from the 1995 Kobe earthquake

Appendix 4B: Relationships Between Structural Characteristics and Steel Building Performance in Severe Earthquakes

A small number of pre-1975 steel framed buildings (older steel-framed buildings) were damaged in the 1994 Northridge earthquake and a significant number in the 1995 Hyogo-ken Nanbu (Kobe) earthquake. From the pattern and extent of damage observed, some general recommendations can be made in order to guide the evaluation of this type of building. A background to these recommendations is now given, followed by details of the recommendations themselves.

The Los Angeles Northridge earthquake, in January 1994, caused considerable damage to modern, ductile moment-resisting steel frames (DMRSFs). This damage took the form of fracture between the beam flange to column flange connection of the rigid beam to column connections. Further details on the nature of the damage and reasons for it are given in Clifton (1996b).

The failures turned the initially rigid connections into semi-rigid connections, with the connection as the weakest flexural link relative to the moment capacity of the beam or the column. The vertical load-carrying capacity remained adequate and the connections retained a reduced moment capacity. Thus the inelastic demand on the frame was concentrated into the connections, which in semi-rigid form retained appreciable ductility.

The hysteretic performance (cyclic moment-rotation curves) representative of the damaged connections is described in Astaneh-Asl (1995). The nature of these curves can be described as being:

a) pinched hysteretic loops with little energy absorption
b) broadly elastoplastic in nature, but not symmetrical, due to the influence of the floor slab
c) susceptible to minor degradation over successive cycles.

While over 100 buildings suffered joint damage in this earthquake, the general response of these buildings was good. Most showed no outward non-structural signs of distress after the earthquake, such as permanent lateral drift, nor were there indications of unexpectedly large interstorey lateral deflections developed during the earthquake. Thus the nature of MRSF response, where the weak link was in the connections, was satisfactory under the high-intensity Northridge Earthquake, which had maximum spectral accelerations in the 0.2–0.8 second period range. (This is reasonably representative of the NZS 1170.5:2004 (or NZS 4203:1992) design spectra for intermediate and stiff soil sites.)

The Hyogo-ken Nanbu (Kobe) earthquake in Japan, in January 1995, caused damage to a range of steel framed buildings, but principally to older, medium – rise commercial and industrial buildings. Large numbers of these older (pre-1981) buildings suffered damage. Their poor performance was due to one or more of the following reasons (Clifton 1996b):

(i) poor distribution of strength/stiffness over successive storeys, leading to soft storey formation

(ii) lack of provision for an adequate load path through the connections, leading to partial or complete connection failure, especially loss of vertical load-carrying capacity

(iii) inadequate strength of the overall seismic-resisting system

(iv) inadequate stiffness of the overall seismic-resisting system

(v) in the case of some older residential buildings, corrosion of the steel frame due to long-term build up of condensation in the external walls envelope.

The pattern of damage from both earthquakes has showed that, for seismic-resisting systems which exhibited inelastic response, three factors are important in order to achieve a good performance of the overall building. These are:

1) the beam to column connections retain their integrity, with regard to carrying shear and axial force, if their moment capacity is reduced

2) inelastic demand is minimised in the columns: both member rotational demand due to general plastic hinging and localised deformation due to local buckling or tearing failure. The former demand can arise from soft-storey formation, as for example is illustrated in Figure 4B.1. In this instance, the soft storey demand has arisen due to the bracing system encompassing all except the bottom storey, resulting in the ductility demand being concentrated into that level. The latter demand is most typically caused by inappropriate detailing for transfer of forces through the connection of incoming beam or brace members into the column. An example is shown in Figure 4B.2 and this concept is covered in detail in section 8.4.2.

3) the inelastic response is essentially symmetrical in nature and does not lead to a progressive displacement of the building in one direction.

These three factors are embodied in the guidelines for evaluation which follow.

Figure 4B.1: Example of soft storey generated by change from braced to moment frame at bottom storey, 1995 Kobe earthquake

Figure 4B.1: Local column crippling failure due to lack of stiffener adjacent to incoming beam flange in a welded, moment-resisting beam to column connection, 1995 Kobe earthquake

Appendix 4C: Assessing the Mechanical Properties of Steel Members and Components

4C.1 Is it cast iron, wrought iron or steel?

The earliest steel framed buildings likely to be requiring a seismic assessment would have been built in the 1880s. As shown in Table 4A.1 of Appendix 4A, the use of cast iron from that time, until its discontinuance around 1910, was confined to columns. These would have typically been used for gravity load carrying columns only. They are typically “chunky” with thick sections, often ornate or complex profile (fluted or plain hollow circular or cruciform columns). Their surface is typically pitted with small blowholes. More detailed visual characteristics are given in Table 7.1 of the SCI Publication 138 (Bussell 1997).

Cast iron is a low-strength, low ductility material not suitable for incorporation into a seismic-resisting system. However, if used as a propped gravity column, with the supports for the beams assessed and reinforced if necessary (e.g. with steel bands) to avoid local fracture under seismic-induced rotation, they can be dependably retained. For more guidance on their assessment for this application (see Bussell 1997).

Wrought iron has good compressive and tensile strength, good ductility and good corrosion resistance. Its performance in this regard is comparable to that of steels from the same era, which largely ended around the 1880s and 1890s. The principal disadvantage of wrought iron was the small quantities made in each production item (bloom), being only 20 to 50 kg. This meant that the use of wrought iron in structural beam and column members required many sections to be joined by rivets. For that reason it was rarely used in building structures in New Zealand. If a building being assessed contains members built up from many small sections of I sections, channels and/or flats and which dates from earlier than 1900, then the use of wrought iron in these members should be further assessed, using the guidance in Sections 3.4 and 7 of Bussell (1997).

All other ferrous components in buildings under assessment can be considered as being made from steel.

If in doubt, the visual assessment criteria in Table 7.1 of Bussell (1997) can be used for more detailed visual consideration.

4C.2 Expected yield and tensile strengths of steels, fasteners and weld metals

The following information is taken from Bussell (1997) and Ferris. The values given are minimum values, being consistent with the requirements from NZS 3404 for the material properties used to be the minimum specified values. This information is given in Table 4C.1 for steels from America and Table 4C.2 for steels from the UK. In the case of the UK, the minimum properties given should be used in the assessment. Properties of UK steels and rivets prior to 1906 can be obtained from Bussell (1997).

Table 4C.1: Minimum material properties for steels and rivets manufactured in the USA

Time period	Application	Minimum yield stress (MPa)	Minimum tensile strength (MPa)
< 1900	Buildings Rivets	240 205	400 340
1900–10	Buildings Rivets	240 205	410 340
1910–25	Buildings Rivets	190 170	380 330
1925–32	Buildings Rivets	210 170	380 314
1932–50	Buildings Rivets	225 195	410 355
1950–76	Buildings (mild steel)	250	410
	Buildings (HT steel)	350	480

Source: Ferris (year?).

Table 4C.2: Typical properties of structural steels from the UK for the period 1906–68

Property (values in N/mm² unless noted)	Typical value (or range of values)	Notes
Ultimate tensile strength:		
BS 15: 1906	432-494	BS 15 covered mild steel
BS 15: 1912-1941	432-509	
BS 15: 1948-1961		
Rivet bar	386-463	
Other	432-509	
BS 548: 1934-1942		BS 548 covered high tensile steel
Rivet bar	463-540	
Other	571-664	
BS 968: 1941	As BS 548: 1934	BS 968 covered weldable high tensile steel
BS 968: 1943	509-633	
BS 968: 1962	494-602	
Yield strength:		
BS 15: 1948-1961	225-235	
BS 15: 1961-1968	230-250	No change in UTS.
BS 548: 1934-1942	293-355	No requirements for rivet bar; values depended on steel thickness, being lower for thicker sections
BS 968: 1941	As BS 548: 1934	
BS 968: 1943	293-324	
BS 968: 1962	340-355	
Elongation at failure (%):		
BS 15: 1906-1941		
Rivet bar	25 (min.)	
Other	20 (min.)	
BS 15: 1948-1961		
Rivet bar	26-30	Cold bend test
Other	16-24	
BS 548: 1934-1942		
Rivet bar	22-27	
Other	14-18	
BS 968: 1941-1943		
Plates	14-18	
Sections and bars	14-22	
BS 968: 1962		
Standard test pieces	15-23	

Source: Bussell (1997).

4C.3 Confirming tensile strength by test

Older steels have an inherently greater variability than modern steels, so it is important to undertake a minimum degree of non-destructive testing to gain sufficient assurance that the materials have the properties used in the assessment.

This testing should also be able to identify material that may exhibit brittle behaviour under seismic condition.

There is an approximate relationship between material hardness and tensile strength. Material hardness is represented in a number of ways, however the best relationship for the range of material strengths of interest (400 to 700 MPa) is given by the *Vickers Hardness, H_v*. Testing for Vickers Hardness is carried out to AS 1817 *Metallic Materials – Vickers Hardness Test* (1991).

That relationship is tabulated in ASM International (1976) and can be expressed in equation form as:

$$f_u = 3.09\,H_v + 21.2 \qquad \ldots 4C(1)$$

where H_v = Vickers Hardness from test.

This expression is valid for $100 \le H_v \le 300$, corresponding to $330 \le f_u \le 950$ MPa.

Vickers Hardness tests are readily undertaken on the insitu steel elements and there are a number of materials testing organisations which can perform this task.

The purpose of the tests is to:

- determine the general material strengths of the critical components
- identify components which have unexpectedly high or low strengths and hence need further investigation
- identify components that might be subject to brittle fracture under seismic conditions.

The steps involved in determining which elements to test and the number of tests to conduct are as follows:

Step 1: Determine the components to be tested, i.e. beams, columns, critical connection components and connectors. Those elements identified as critical from the connection evaluation in section 8.4.2 and the strength heirachy evaluation in section 8.5.2 should be subject to the most detailed testing, plus a lesser frequency of testing for other beam, column and brace members.

Step 2: Determine a frequency of testing. Use the guidance in Section 7.5 of Bussell (1997) and DCB No. 44, pp. 2–3 [Clifton (ed.)], aimed at covering 15% of the total sample of each type of component being tested for critical components; increasing this to 25% if the results show a significant number of suspect samples.

Step 3: Use eqn 4C(1) to obtain the tensile strength.

Step 4: Compare with the expected strengths from Section 4C.2 and make a judgement on the material's suitability. Any materials with $H_v < 100$ or $H_v > 230$ should be investigated more thoroughly by tensile sampling and visual inspection. Any materials with $H_v >$ 230 should also be treated as potentially prone to brittle fracture.

4C.4 Suppression of brittle fracture

This becomes a issue for further investigation if the testing from Section 4C.2 shows up a steel with a H_v of over 230 and/or if the thickness of any element of existing steelwork is over 32 mm thick, when that element is in the “principal load-carrying path through the seismic-resisting system” (NZS 3404:1997) and is carrying axial or bending induced tension force. In those cases material from those elements needs to be removed for Charpy Impact Testing, as specified from NZS 3404, to determine the energy absorption. These tests should be conducted at 0^{o}C for elements of external steelwork and at 20^{o}C for elements of internal steelwork.

There is not a direct relationship between tensile strength and brittle fracture, however the susceptibility to brittle fracture increases with increasing tensile strength. The elongation also decreases with increasing strength. This guidance is therefore a threshold, requiring more appropriate testing for potential brittle fracture performance if it is not met.

Appendix 4D: Potential for Pounding

4D.1 Evaluation of Potential for Pounding

The effects of pounding need to be considered where both of the following criteria apply.

a) Either of the following conditions exist:

   i) Adjacent buildings are of different heights and the height difference exceeds two storeys or 20% of the height of the taller building, whichever is the greater.

   ii) Floor elevations of adjacent buildings differ by more than 20% of the storey height of either building.

b) Separation between adjacent buildings at any level is less than a distance given by:

   $S = \sqrt{U_1^2 + U_2^2}$

   where U_1 = estimated lateral deflection of Building 1 relative to ground under the loads used for the assessment.

   and U_2 = estimated lateral deflection of Building 2 relative to ground under two-thirds of the loads used in the assessment.

However, the value of 'S' calculated above need not exceed 0.028 times the height of the building at the possible level(s) of impact.

Where adjacent buildings are of similar height and have matching or similar floor levels, no account need be taken of the effects of pounding on either building irrespective of the provided separation clearances.

4D.2 Assessment of Pounding Effects

Where required to account for the effect of pounding in 1 above, the following alternative approaches may be adopted.

4D.2.1 Analytical approach

A proper substantiated analysis shall be undertaken that accounts for the transfer of momentum and energy between the buildings as they impact. Elements and components of the building structures shall be capable of resisting the forces resulting from impact, giving due consideration to their ductility capacity and need to sustain vertical forces under such impact loading.

4D.2.2 Approximate approach

i) For the case of two unequal height buildings where their floor elevations align, the impact-side columns of the taller building should have sufficient strength to resist the following design actions.

   - 175% of the column design actions (shear, flexural and axial) occurring under the application of the seismic lateral loading of NZS 1170.5:2004, assuming the building is free standing, over the height of the building corresponding to that of the adjacent shorter building.
   - 125% of the column design actions occurring under the application of the seismic lateral loading of NZS 1170.5:2004, assuming the building is free standing, over the height of the building corresponding to that of the adjacent shorter building.

All other columns remote from the building side suffering impact shall have sufficient strength to resist 115% of the column design actions occurring under the application of the seismic lateral loading of NZS 1170.5:2004, assuming the building is free standing, over the full height of the building.

ii) For the case where the floor elevations of adjacent buildings differ, with the potential for mid-storey hammering of each building, the impact-side columns of the building(s) which may be impacted between storeys should have sufficient strength to resist design actions resulting from imposition of a displacement on the columns, at the point of impact, corresponding to one half of the value of '*S*' derived in 4A.1(b) above.

The imposed displacements need only be applied at any one level. However critical design actions shall be derived considering application of the imposed displacements at any level over the building height where impact could occur.

In addition, where the buildings are of unequal heights, in accordance with 4D.1(a)(i) above, the requirements of 4D.2.2 (i) shall also apply.

4D.3 Alternative Mitigation Approaches

Alternative means to mitigate the effects of pounding may be considered. These include:

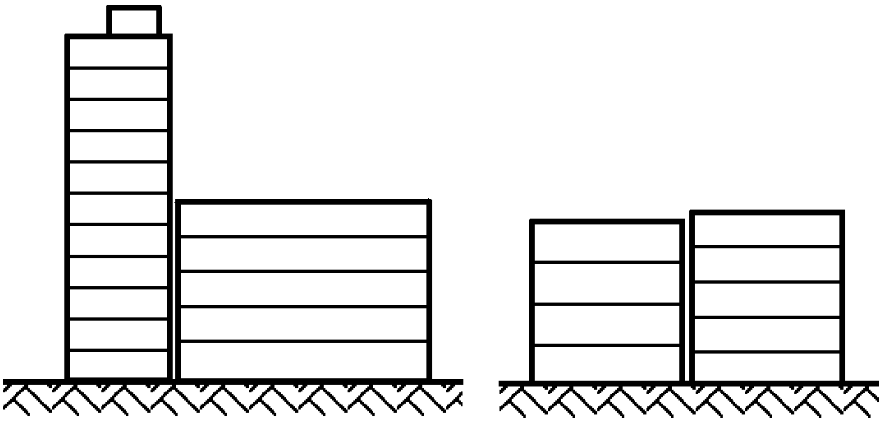
- permanent connection of adjacent buildings. This approach may prove practical for a row or block of buildings of similar height and configuration.
- provision of additional structural elements and components away from the points of impact to compensate for components that may be severely damaged due to impact.
- provision of strong collision shear walls to act as buffer elements to protect the rest of the building (Anagnostopoulas and Spiliopoulos 1992). The use of collision shear walls would prevent mid storey impact to columns of adjacent buildings, reducing potential for local damage and partial or total collapse.

Older buildings have often been built up to property boundary lines, with little or no separation to adjacent buildings. Buildings with inadequate separation may consequently impact each other or pound during an earthquake. Such impacts will transmit short duration, high amplitude forces to the impacting buildings at any level where pounding occurs with the following consequential effects:

- *High "in-building" accelerations in the form of short duration spikes.*
- *Modification to the dynamic response of the buildings, the pattern and magnitude of inertial demands and deformations induced on both structures. Response may be amplified or de-amplified and is dependent on the relative dynamic characteristics of the buildings, including their relative heights, masses and stiffness', as well as ground conditions that may give rise to soil-structure interaction and the magnitude and direction of travel of the earthquake motions.*
- *Local degradation of strength and/or stiffness of impacting members.*

Numerous pounding damage surveys and numerical and analytical pounding studies have been undertaken in the last 10–15 years, especially after the 1985 Mexico City earthquake that caused an unusually large number of building failures. It is clear that pounding is a complex problem with numerous circumstances under which it can be encountered. The results of the studies that have been undertaken are sensitive to the many parameters related to the building structures (and their numerical modelling) in addition to the prevalent soil conditions and the characteristics and *direction of seismic attack. However based on these studies and evidence from past earthquakes, it is possible to draw the following general conclusions.*

- *Where buildings are significantly different in height, period and mass, large increases in response from pounding can be expected.*
- *Differences in height in particular between neighbouring buildings can result in significant pounding effects, producing large response increases in the upper part of the taller building (refer Figure 4D.1(a)). The shears in the impact-side columns for the taller building can be up to 50–70% higher than in the no pounding case at the levels immediately above the lower building, and 25–30% at levels higher up, as the shorter building acts as a buttress to the taller building. In soft ground conditions where soil-structure interaction and through-soil coupling occurs, the impact-side shears can be enhanced by a further 25–50%.*
- *For buildings of similar height and having similar mass and stiffness, in most cases the effects of pounding will be limited to some local damage, mostly non-structural and nominal structural, and to higher in-building accelerations in the form of short duration spikes. In such conditions, from a practical viewpoint, the effects of pounding on global responses can be considered insignificant.*
- *Where building floors are at different elevations, the floor slabs of one structure can impact at the mid-storey of the columns of the others, shearing the columns and initiating partial or total collapse (refer Figure 4D.1(b)). Particularly susceptible to such action are buildings overtopping a shorter neighbouring building whose columns may be impacted at mid-storey by the uppermost level of the shorter building.*
- *The local high amplitude, short duration accelerations induced by colliding buildings will increase the anchoring requirements for the contents of the buildings as well as architectural elements.*



(a) Buildings of unequal height **(b) Buildings of equal height**

Figure 4D.1: Example of differing floor elevations in adjacent buildings

The potential or likelihood of pounding needs to be evaluated, using calculated drifts for both buildings. The SRSS combination of structural lateral deflections of both buildings is proposed, as adopted in FEMA 273 (NEHRP Guidelines), to check the adequacy of building separation. This approach has been adopted to account for the low probability of maximum drifts occurring simultaneously in both buildings whilst they respond completely out of phase. It is not intended that detailed analysis or modeling be undertaken to determine building drifts but rather general estimates be used.

Approximate analytical methods have been proposed for assessing the effects of pounding, including time history analyses (Johnson, Conoscente and Hamburger 1992) and elastic response spectrum analyses (Kasai, Maison and Patel 1990). Use of such approaches however may not prove practical for many buildings or within the capability of many design practitioners.

An alternative simplified approach has been proposed, based on simple factoring of earthquake design forces applicable to the building, to ensure some account of pounding effects is made. Both moment/shear capacities and p-delta effects need to be considered. Studies (Kasai, Maison and Patel 1990; Kasai, Jeng, Patel, et al 1992; Carr and Moss 1994) have shown that column and storey shears in the taller building above the pounding level can be increased by anywhere up to or exceeding 100%. The level of increase is dependent on many factors including initial separation distances and relative mass and stiffness of the adjacent buildings. A midrange increase in design shear has been adopted for the simplified approach at this stage. Whilst it is recognised that this approximate approach is relatively crude it has the benefit of ease of application without the need for use and familiarity with sophisticated analyses tools. It is expected that as further research on pounding is undertaken more appropriate and practical means to evaluate and mitigate pounding will become available.

Appendix 4E: Analysis Procedures

NOTE

This Appendix is based on material contained in FEMA 356.

Other background information can be found in FEMA 273 and 274.

This information is presented as commentary material to assist assessors in the application of the analysis procedures outlined in Section 4 and 6.

4E.1 Introduction and Scope

This appendix sets out the requirements for analysis of buildings and describes the general analysis requirements for mathematical modelling including basic assumptions, consideration of torsion, diaphragm flexibility, and P-Δ effects. Five methods that can be used to analyse a building are then described in detail.

Section 4.3.2and Table 4.2, summarise several elastic and inelastic analysis methods that can be used to assess strength and displacement demands that a building might be subjected to during and earthquake. Of the elastic methods, the Equivalent Static Method is a linear elastic procedure, while the Modal Response Spectrum Method is a linear dynamic procedure. In the case of the inelastic methods, the SLaMA and the Pushover Method are nonlinear static procedures whereas the Inelastic Time History Method is a nonlinear dynamic procedure.

Linear procedures are appropriate when the expected level of nonlinearity is low. Static procedures are appropriate when higher mode effects are not significant. This is generally true for short, regular buildings. Dynamic procedures are required for tall buildings, buildings with torsional irregularities, or non-orthogonal systems.

The Nonlinear Static Procedure is acceptable for most buildings, but should be used in conjunction with the Linear Dynamic Procedure if mass participation in the first mode is low.

The term "linear" in linear analysis procedures implies "linearly elastic." The analysis procedure, however, may include geometric nonlinearity of gravity loads acting through lateral displacements and implicit material nonlinearity of concrete and masonry components using properties of cracked sections. The term "nonlinear" in nonlinear analysis procedures implies explicit material nonlinearity or inelastic material response, but geometric nonlinearity may also be included.

4E.2 Mathematical Modelling

A building should be modelled, analysed, and evaluated as a three dimensional assembly of elements and components. However, use of a two dimensional model can be justified when:

1. The building has rigid diaphragms and horizontal torsion effects are not large or the horizontal torsion effects have been accounted for, or
2. The building has flexible diaphragms.

If two dimensional models are used, the three-dimensional nature of components and elements should be taken into account when calculating stiffness and strength properties.

If the building contains out-of-plane offsets in vertical lateral force-resisting elements, the model should explicitly account for these offsets when determining the demands on the diaphragms.

For nonlinear procedures, a connection should be modelled explicitly if the connection is weaker, has less ductility than the connected components, or the flexibility of the connection results in a change in the connection forces or deformations greater than 10%.

For two-dimensional models, the three-dimensional nature of components and elements should be recognized in calculating their stiffness and strength properties. For example, shear walls and other bracing systems may have "L" or "T" or other three dimensional *cross-sections where contributions of both the flanges and webs should be accounted for in calculating stiffness and strength properties.*

In these recommendations, component stiffness is generally taken as the effective stiffness based on the secant stiffness to yield level forces.

Examples of where connection flexibility may be important to model include the panel zone of steel moment-resisting frames, the "joint" region of perforated masonry or concrete walls, and timber diaphragms.

4E.3 Horizontal Torsion

The effects of horizontal torsion should be considered. Torsion need not be considered in buildings with flexible diaphragms as defined in Section 5(a) herein. The total horizontal torsional moment at a storey is given by the sum of the actual torsional moment and the accidental torsional moment as given in NZS 1170.5:2004, Clause 6.3.5.

Actual torsion is due to the eccentricity between the centres of mass and stiffness. Accidental torsion is intended to cover the effects of the rotational component of the ground motion, differences between computed and actual stiffnesses, and unfavourable distributions of dead and live load masses.

4E.4 Primary and Secondary Elements and Components

Elements and components may be classified as primary or secondary. Elements and components that affect the lateral stiffness or distribution of forces in a structure, or are loaded as a result of the lateral deformation of a structure should be classified as primary or secondary, even if they were not intended to be part of the lateral force resisting system.

Primary elements and components are those that provide the capacity of the structure to resist collapse under the seismic forces induced by the ground motion in any direction. Other elements and components can be classified as secondary. Primary elements and components should be checked for earthquake induced forces and deformations in combination with gravity load effects. Secondary elements and components should be checked for earthquake deformations in combination with gravity load effects.

NOTE

This definition of primary and secondary elements is not the same as used in NZS 3404 for steel structures.

4E.5 Diaphragms

4E.5.1 Classification of Diaphragms

Diaphragms should be classified as flexible when the maximum horizontal deformation of the diaphragm along its length is more than twice the average interstory drift of the vertical lateral-force-resisting elements of the story immediately below the diaphragm. For diaphragms supported by basement walls, the average interstory drift of the story above the diaphragm should be used.

Diaphragms should be classified as rigid when the maximum lateral deformation of the diaphragm is less than half the average interstory drift of the vertical lateral-force-resisting elements of the associated story.

Diaphragms that are neither flexible nor rigid should be classified as stiff.

For the purpose of classifying diaphragms, interstory drift and diaphragm deformations should be calculated using the pseudo lateral load specified in Equation (3-10). The in-plane deflection of the diaphragm should be calculated for an in-plane distribution of lateral force consistent with the distribution of mass, and all in-plane lateral forces associated with offsets in the vertical seismic framing at that diaphragm level.

4E.5.2 Mathematical Modelling

Mathematical modelling of buildings with rigid diaphragms should account for the effects of horizontal torsion as specified in Section 4E.3 above. Mathematical models of buildings with stiff or flexible diaphragms should account for the effects of diaphragm flexibility by modelling the diaphragm as an element with an in-plane stiffness consistent with the structural characteristics of the diaphragm system. Alternatively, for buildings with flexible diaphragms at each floor level, each lateral force-resisting element in a vertical plane may be permitted to be designed independently, with seismic masses assigned on the basis of tributary area.

Evaluation of diaphragm demands should be based on the likely distribution of horizontal inertia forces. For flexible diaphragms, such a distribution may be given by eqn 4E(1)) and illustrated in Figure 4E.1.

$$f_d = \frac{1.5F_d}{L_d}\left[1-\left(\frac{2x}{L_d}\right)^2\right] \qquad \ldots 4E(1)$$

where:
f_d = Inertial load per foot
F_d = Total inertial load on a flexible diaphragm
x = Distance from the centre line of flexible diaphragm
L_d = Distance between lateral support points for diaphragm

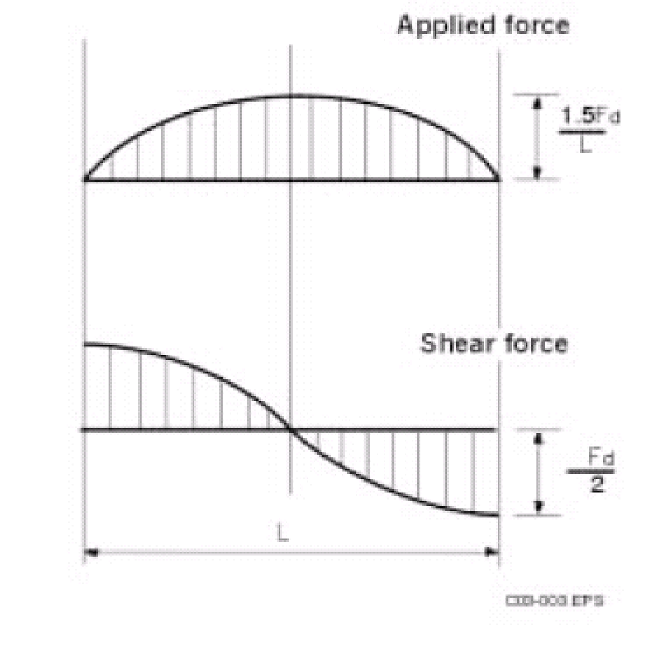


Figure 4E.1: Plausible force distribution in a flexible diaphragm

4E.6 P-Δ Effects

Buildings should be checked for P-Δ effects as set out in Section 6.5 of NZS 1170.5:2004.

P-Δ effects are caused by gravity loads acting through the deformed configuration of a building and result in increased lateral displacements.

A negative post-yield stiffness may significantly increase interstory drift and the target displacement. Dynamic P-Δ effects are introduced to consider this additional drift.

The degree by which dynamic P-Δ effects increase displacements depends on the following:

1. *The ratio α of the negative post-yield stiffness to the effective elastic stiffness;*
2. *The fundamental period of the building;*
3. *The strength ratio, R, (being the ratio of the yield strength to the ultimate strength);*
4. *The hysteretic load-deformation relations for each story;*
5. *The frequency characteristics of the ground motion; and*
6. *The duration of the strong ground motion.*

4E.7 Methods of Analysis

Selection of an appropriate analysis method should be based on Table 4.2.

4E.8 Equivalent Static Analysis

4E.8.1 Period Determination

The fundamental period of the building can be calculated for the direction under consideration using one of the following analytical, empirical, or approximate methods.

a) Method 1 – Analytical

Dynamic (eigenvalue) analysis of the mathematical model of the building can be carried out to determine the fundamental period of the building.

For many buildings, including multi-storey buildings with well-defined framing systems, the preferred approach to obtaining the period for design is Method 1. In this method, the building is modelled using the modelling procedures of Section 5 through 8 and 11, and the period is obtained by Eigenvalue analysis. Flexible diaphragms may be modelled as a series of lumped masses and diaphragm finite elements.

b) Method 2 – Empirical

The fundamental period of the building shall be determined in accordance with:

1. $$T_1 = 1.25\, k_t\, h_n^{0.75} \qquad \ldots 4E(2)$$

where ;

k_t = 0.075 for moment resisting concrete frames
0.11 for moment-resisting steel frames
0.06 for eccentrically braced steel frames
0.05 for all other frame structures

h_n = height in m from the base of the structure to the uppermost seismic weight or mass.

2. Alternatively, the value k_t for structures with concrete shear walls may be taken as

$$k_t = 0.075 / \sqrt{A_c} \qquad \ldots 4E(3)$$

where

$$A_c = [A_i \{0.2 + (l_{wi} / h_n)\}^2]$$

and

A_c = total effective area of the shear walls in the first storey in the building, in m^2,

A_i = effective cross-sectional area of shear wall i in the first storey of the building, in m^2,
h_n = as in item 1 above,
l_{wi} = length of shear wall i in the first storey in the direction parallel to the applied forces, in m, with the restriction that l_{wi}/h_n shall not exceed 0.9.

3. The estimation of T_1 may be made using the following expression:

$$T_1 = 2\sqrt{d} \qquad \ldots 4E(4)$$

where

d = the lateral elastic displacement of the top of the building, in m, due to gravity loads applied in the horizontal direction.

Empirical equations for period, such as that used in Method 2, intentionally underestimate the actual period and will generally result in conservative estimates of pseudo lateral load. Studies have shown that depending on actual mass or stiffness distributions in a building, the results of Method 2 may differ significantly from those of Method 1.

c) Method 3 - Approximate

1. For any building, the Rayleigh-Ritz method can be used to approximate the fundamental period.

The largest translational period in the direction under consideration, T_1, may be calculated from eqn 4E(5).

$$T_1 = 2\pi\sqrt{\frac{\sum_{i=1}^{n}\left(W_i d_i^{\,2}\right)}{g\sum_{i=1}^{n}\left(F_i d_i\right)}} \qquad \ldots 4E(5)$$

where

d_i = the horizontal displacement in m of the centre of mass at level i, ignoring the effects of torsion
F_i = the displacing force in kN at level i
g = acceleration due to gravity in m/s^2
i = the level under consideration of structure
n = number of levels in a structure
W_i = the seismic weight in kN at level i.

2. For one-story buildings with single span flexible diaphragms, eqn 4E(6) may be used to approximate the fundamental period.

$$T = \left(3.94\,U_w + 3.07\,U_d\right)^{0.5} \qquad \ldots 4E(6)$$

where U_w and U_d are in-plane wall and diaphragm displacements in metres, due to a lateral load in the direction under consideration, equal to the weight of the diaphragm.

3. For one-story buildings with multiple-span diaphragms, eqn 4E(6) may be used as follows: a lateral load equal to the weight tributary to the diaphragm span under consideration is applied to calculate a separate period for each diaphragm span. The period that maximizes the pseudo lateral load is used for design of all walls and diaphragm spans in the building.

4. For unreinforced masonry buildings with single span flexible diaphragms, six stories or less in height, eqn 4E(7) may be used to approximate the fundamental period.

$$T = (3.07 U_d)^{0.5} \qquad \text{...4E(7)}$$

where U_d is the maximum in-plane diaphragm displacement in metres, due to a lateral load in the direction under consideration, equal to the weight tributary to the diaphragm.

Method 3 is appropriate for systems with rigid vertical elements and flexible diaphragms in which the dynamic response of the system is concentrated in the diaphragm. Use of Method 2 on these systems to calculate the period based on the stiffness of the vertical elements will substantially underestimate the period of actual dynamic response and overestimate the pseudo lateral load.. Eqn 4E(7) is a special case developed specifically for URM buildings. In this method, wall deformations are assumed negligible compared to diaphragm deflections. For illustration of wall and diaphragm displacements see Figure 4E.2. When calculating diaphragm displacements for the purpose of estimating period using eqns 4E(6) or 4E(7), the diaphragm should be considered to remain elastic under the prescribed lateral loads.

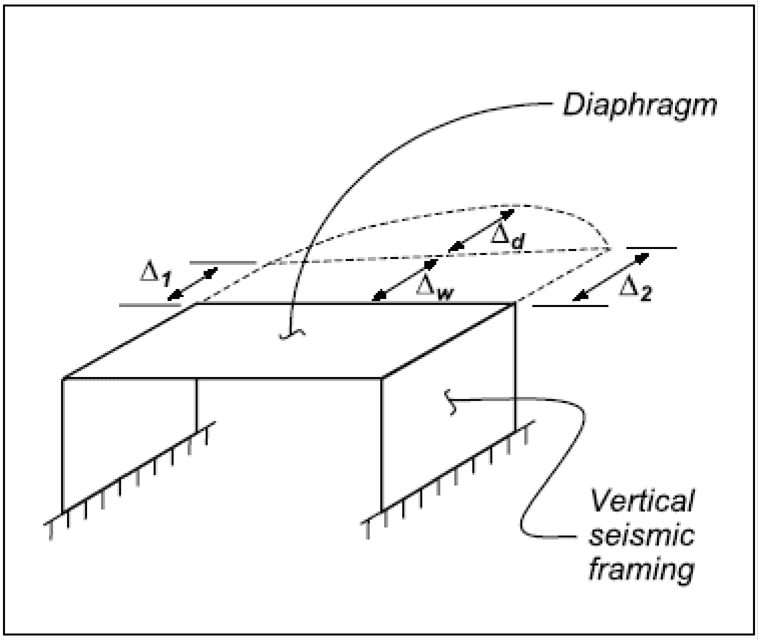


Figure 4E.2 Diaphragm and wall displacement terminology

4E.8.2 Pseudo Lateral Load

The pseudo lateral load in a given horizontal direction can be determined from eqn 4E(8). This load is applied to the vertical elements of the lateral force resisting system.

$$V = C_1 C_2 C_3 C_m S_a W_t \qquad \text{...4E(8)}$$

where:

V = Pseudo lateral load

C_1 = Modification factor to relate expected maximum inelastic displacements to those calculated for linear elastic response. Values suggested in FEMA 356 are:

$C_1 = 1.5$ for $T < 0.10$ second.

$C_2 = 1.0$ for $T \geq T_s$ second.

Linear interpolation may be used to calculate C_1 for intermediate values of T.

T = The fundamental period of the building in the direction under consideration, calculated as in Section 8.1 herein.

T_s = The characteristic period of the response spectrum, defined as the period associated with the transition from the constant acceleration segment of the spectrum to the constant velocity segment of the spectrum.

C_2 = Modification factor to represent the effects of pinched hysteresis shape, stiffness degradation, and strength deterioration on the maximum displacement response. C_2 should be taken as 1.0 for the case of linear elastic analysis.

C_3 = Modification factor to represent increased displacements due to dynamic P-Δ effects listed in Section 4B.6 herein. For values of the stability index, 2_i , (see Section 6.5 of NZS 1170.5:2004), less than 0.1 in all stories, C_3 shall be taken as 1 + 5(2 - 0.1)/T using 2 equal to the maximum value of 2_i of all stories.

C_m = Effective mass factor to account for higher mode mass participation effects and can be taken as 1.0 for one and two storey structures, or if the fundamental period, T, is greater than 1.0 seconds. In the case of steel or concrete buildings of three or more stories, a value of 0.9 can be used for C_m .

S_a = Response spectrum acceleration at the fundamental period and damping ratio of the building in the direction being considered and taken from Section 3 of NZS 1170.5:2004.

W_t = The effective seismic weight of the building.

Coefficient C_1. *This modification factor is to account for the difference in maximum elastic and inelastic displacement amplitudes in structures with relatively stable and full hysteretic loops. The values of the coefficient are based on analytical and experimental investigations of the earthquake response of yielding structures. See FEMA 356, Section 3.3.3.3 for further discussion.*

Coefficient C_2. *This coefficient adjusts design values based on component hysteresis characteristics, stiffness degradation, and strength deterioration. See FEMA 274 for additional discussion.*

Coefficient C_3. *For framing systems that exhibit negative post-yield stiffness, dynamic P-Δ effects may lead to significant amplification of displacements. Such effects cannot be explicitly addressed with linear procedures. No measure of the degree of negative post-yield stiffness can be explicitly included in a linear procedure.*

4E.8.3 Vertical Distribution of Seismic Forces

The vertical distribution of the pseudo lateral load should be as specified in this section for all buildings except unreinforced masonry buildings for which the pseudo lateral loads should be distributed as setout below. The lateral load F_x applied at any floor level x should be determined in accordance with Eqn 4E(8) and Eqn 4E(10):

$$F_x = C_{vx} V \qquad \ldots 4E(9)$$

$$C_{vx} = \frac{w_x h_x^k}{\sum_{i=1}^{n} w_i h_i^k} \qquad \ldots 4E(10)$$

where:

C_{vx} = Vertical distribution factor
k = 2.0 for $T \geq 2.5$ seconds
= 1.0 for $T \leq 0.5$ seconds
Linear interpolation shall be used to calculate values of k for intermediate values of T.
V = Pseudo lateral load
w_i = Portion of the total building weight W located on or assigned to floor level i
w_x = Portion of the total building weight W located on or assigned to floor level x
h_i = Height (in m) from the base to floor level i
h_x = Height (in m) from the base to floor level x

For unreinforced masonry buildings with flexible diaphragms for which the fundamental period is calculated using Eqn 4E(10), the pseudo lateral loads can be calculated and distributed as follows:

1. For each span of the building and at each level, calculate period
2. Calculate pseudo lateral load for each span.
3. Apply the lateral loads calculated for all spans and calculate forces in vertical seismic-resisting elements using tributary loads.
4. Diaphragm forces for evaluation of diaphragms are determined from the results of step 3 above and distributed along the diaphragm span considering its deflected shape.
5. Diaphragm deflection should not exceed 300 mm for this method of distribution of pseudo lateral loads to be applicable.

4E.8.4 Horizontal Distribution of Seismic Forces

The seismic forces at each floor level of the building should be distributed according to the distribution of mass at that floor level.

4E.8.5 Diaphragms

Diaphragms should be designed to resist the combined effects of the inertial force, F_{px}, calculated in accordance with eqn 4E(11), and horizontal forces resulting from offsets in or changes in the stiffness of the vertical seismic framing elements above and below the diaphragm. Forces resulting from offsets in or changes in the stiffness of the vertical seismic framing elements should be taken as the forces due to the pseudo lateral load without reduction, unless smaller forces are justified by a limit-state or other rational analysis, and should be added directly to the diaphragm inertial forces.

$$F_{px} = \sum_{i=x}^{n} F_i \frac{w_x}{\sum_{i=x}^{n} w_i} \qquad \ldots 4E(11)$$

where:

F_{px} = Total diaphragm inertial force at level x
F_i = Lateral load applied at floor level i given
w_i = Portion of the effective seismic weight W located on or assigned to floor level i
w_x = Portion of the effective seismic weight W located on or assigned to floor level x

The seismic load on each flexible diaphragm is then distributed along the span of that diaphragm, proportional to its displaced shape.

4E.9 Modal Response Spectrum Analysis

The horizontal ground motion should be either a response spectrum taken from Section 3 of NZS 1170.5:2004, or else a response spectrum determined by a site-specific investigation.

Modal spectral analysis is carried out using linearly elastic response spectra that are not modified to account for anticipated nonlinear response. It is expected that the method will produce displacements that approximate maximum displacements expected during the design earthquake, but will produce internal forces that exceed those that would be obtained in a yielding building. Calculated internal forces typically will exceed those that the building can sustain because of anticipated inelastic response of components and elements

4E.9.1 Response Spectrum Method

Should be carried out in accordance with Clause 6.3 of NZS 1170.5:2004.

4E.10 Simple Lateral Mechanism Analysis (SLaMA)

A hand analysis is carried out to determine the likely collapse mechanism and its lateral strength and displacement capacity. This is then compared to the earthquake demand on the structure determined using either a force- or displacement-based method. The following sets out a possible SLaMA procedure for a framed building.

4E.10.1 Lateral frame capacities

For each lateral frame (with or without walls):

1. Calculate the beam gravity moments, M_{BG}, and the gravity shear forces, V_{BG} (approximately).

2. Calculate the column and wall gravity loads, N_G.

3. Determine the beam moment capacities, MBN. Where the reinforcing comprises smooth bars, assume both top and bottom reinforcement is in tension regardless of the position of the neutral axis.

4. Determine the beam shears at the moment capacities as illustrated in Figure 4E.3.

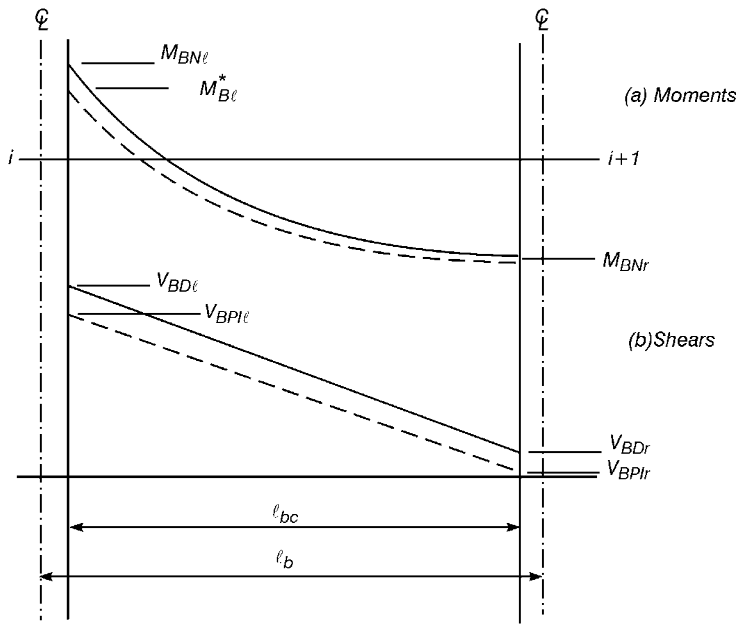


Figure 4E.3 Beam shears

i.e. $$V_{BDl} = V_{BGl} + V_{BEl} = V_{BGl} + \left(M_{BNl} + M_{BNr}\right)/\,l_{bc} \qquad 4E(12)$$

5. Determine the initial probable beam shear capacity, V_{BPI}, using eqn 7(5).

6. Check the initial beam shear strength to determine whether it is greater than the beam shear, V_{BD}, at the beam moment capacity. If $V_{BPI} > V_{BD}$, then reduce the effective beam moment capacity to (se Fig. 4E.3):

$$M^*_{Bl} = \left(V_{BPIl} - V_{BGl}\right)l_{bc} - M_{BNr} \qquad 4E(13)$$

7. Check the beam/column joint capacity demand as follows:

   (a) assume the top beam forms beam ‘hinges’ based on the moments, M_{BN}, from Step 3 or the reduced moments , M_B, from Step 6, i.e. Equation 4E(13).

   (b) Determine the joint shear strength using Equation 7(11), and the principal tensile stress, $p_t = k\sqrt{f_c}$’

   (c) If the joint capacity demand is too high, the beam moment capacity will need to be reduced.

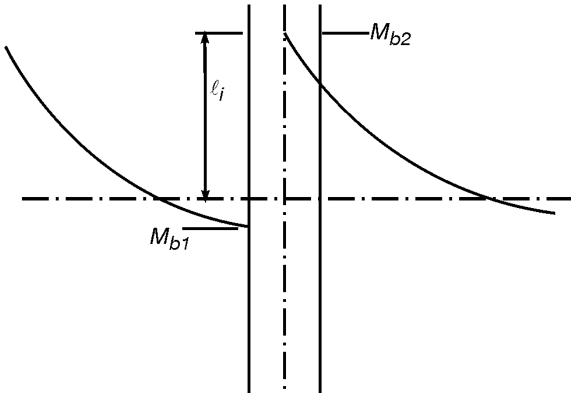


Figure 4E.4 Beam hinges

$$V_{ij} \approx \frac{(M_{b1}+M_{b2})}{0.9h_b} - V_{col}$$

$$= \frac{\sum M_b}{0.9h_b} - V_{col}$$

where h_b is the beam depth

$$\therefore V_{col} \approx 0.5\sum M_b \frac{l_b}{l_{bc}}\frac{1}{l_c}$$

$$\approx \frac{1.2\sum M_b}{l_c}$$

where l_b = beam length
l_{bc} = clear beam length
l_c = column height, between beam centrelines

$$\therefore V_{jh} \approx \frac{1.1\sum M_b}{h_b} - \frac{1.2\sum M_b}{l_c}$$

$$= \sum M_b \left(\frac{1.1l_c - 1.2h_b}{h_b l_c}\right) \qquad \text{4E(14)}$$

$$\therefore \sum M_b = (M_{b1}+M_{b2}) = V_{jhc}\left[\frac{h_b l_c}{1.1l_c - 1.2h_b}\right]$$

where $$V_{jhc} = p_t\sqrt{1+\frac{N^*}{A_g p_t}}$$

(d) Determine the column seismic axial forces, N_E*, below the beam arising from the seismic beam shears using the reduced beam moments (if necessary).

(e) Repeat (a) to (d) for each floor level down to the lowest level.

8. Determine the column shears and check the column shear demand/capacity.

(a) Using values of N* from Step 7 above (seismic plus gravity), calculate the column shear strength, V_{CPI}, using Equation 7(6).

(b) Calculate the column flexural strength under N*.

(c) Check whether the footings will rock or not. If they will, then reduce the column base moment capacity.

(d) Check the joint sway potential.

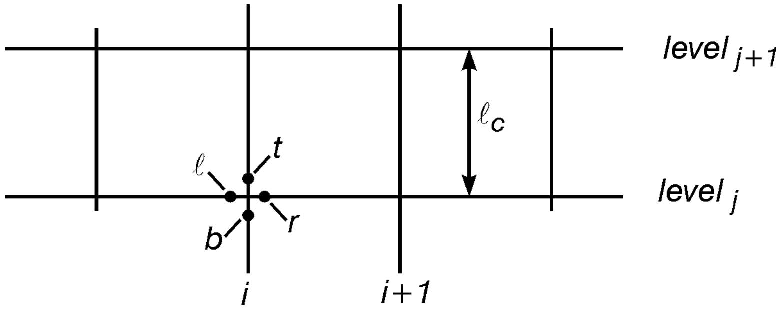


Figure 4E.5 Sway potential

The sway potential at the joint on column i at level j (see Fig. 4E.5) is given by

$$S_{Pij} = \frac{M_{bijl} + M_{bijr}}{M_{cijt} + M_{cijb}}$$

based on the full moment capacity at the joint centroid. If S_{Pij} > 0.85, assume that the column hinges at t and/or b.

(e) The column shear demand is given by:

$$V_{CD} = w_V \frac{\left(M_{bijl} + M_{bijr} + M_{bij+1,l} + M_{bij+1,r}\right)}{2kl_c}$$
$$\leq \frac{\left(M_{cijt} + M_{cij+1,b}\right)}{kl_c}$$

At the column base, use M_{CiO} instead of the beam moments.

(f) Check the initial column shear failure, i.e. is $V_{\mathrm{CPI}} > V_{\mathrm{CD}}$?

If the check is satisfactory, go to the next frame.
If $V_{\mathrm{CPI}} < V_{\mathrm{CD}}$, then the column is likely to fail in a brittle manner. In this case, μ_s = 1, and the beam moments and N* must be reduced proportionally.

(g) Check the next frame.

4E.10.2 Check the storey sway potential at each level.

1. Determine the storey sway potential for each frame where

$$S^*_{pjk} = \frac{\sum_i \sum_k \left(M_{bijkl} + M_{bijkr}\right)}{\sum_i \sum_k \left(M_{cijkt} + M_{cijkb}\right)}$$

where i = the column number
j = the storey number, and
k = the frame number

The beam and column moments are those extrapolated to the joint centroid.

2. Check whether $S_{pjk}^* > 0.85$ k.
If it does, then sway potential exists.

3. Check possible sway mechanisms as illustrated in Fig. 4E.6.

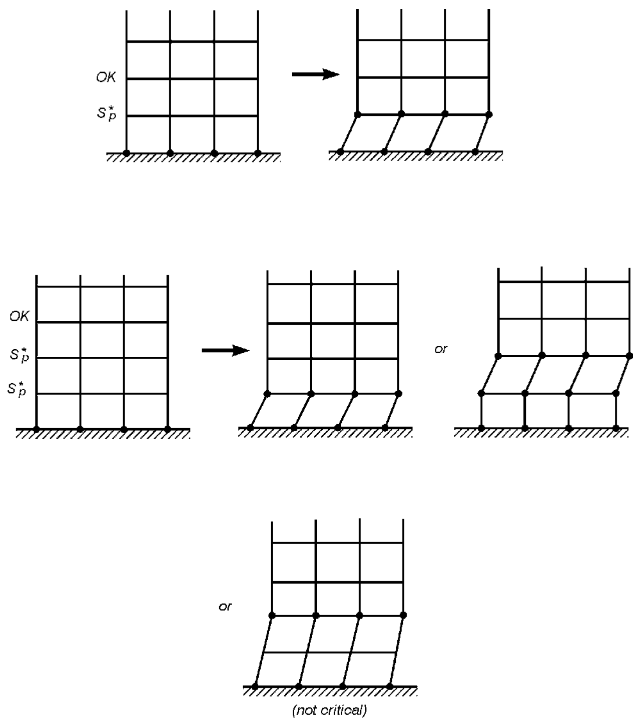


Figure 4E.6 Mechanisms

4E.10.3 Force-based Assessment of Demand

1. Calculate the overturning moment capacity of each frame in the structure (see Fig. 4E.7).

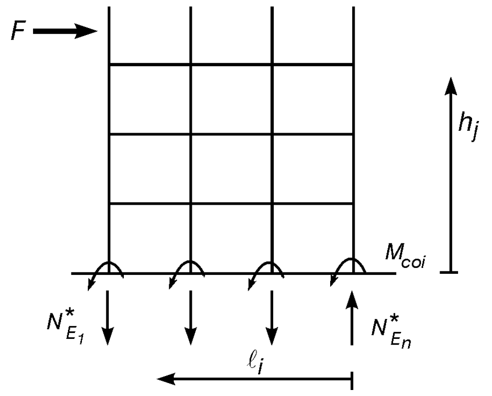


Figure 4E.7 Overturning capacity

Note: determine OTM[1] for unreduced beam moments, or
OTM[2] for beam moments reduced for ultimate joint shear, or
OTM[3] for beam moments reduced for the collapse mechanism.

2. Calculate the overturning moment capacity of the whole structure as:

$$V_{CD} = w_V \frac{\left(M_{bijl} + M_{bijr} + M_{bij+1,l} + M_{bij+1,r}\right)}{2kl_c}$$

$$\leq \frac{\left(M_{cijt} + M_{cij+1,b}\right)}{kl_c}$$

$$\text{Total}\, OTM = \sum_k OTM_k \quad \text{for k frames}$$

3. Determine the height of the lateral force resultant from

$$h_{eff} = \sum m_j h_j^2 \Big/ \sum m_j h_j$$

where m_j = mass at storey j.

$$OTM_n = \sum_i M_{coi} + \sum N_{Ei} l_i$$

4. The base shear capacity can be determined from

$$V_B = OTM / h_{eff}$$

5. The yield displacement, μ_y, is given by

$$\Delta_y = \left[0.5\varepsilon_y \frac{l_b}{h_b} h_{eff}\right] \frac{OTM_1}{OTM_2}$$

where l_b = full beam length (see Fig. 4E.3) and d_b is the beam depth.

6. Calculate the frame ultimate displacement capacity for the assessed yield mechanism as given in Figure 4E.8.

7. Determine whether the structure is torsionally eccentric.

(a) If it is, then determine the strength eccentricity. With reference to Figure 4E.9.

$$\bar{y} = \sum V_{Bk}\, y_k \Big/ \sum V_{Bk}$$
$$e = \bar{y} - y_{mass\ centre}$$

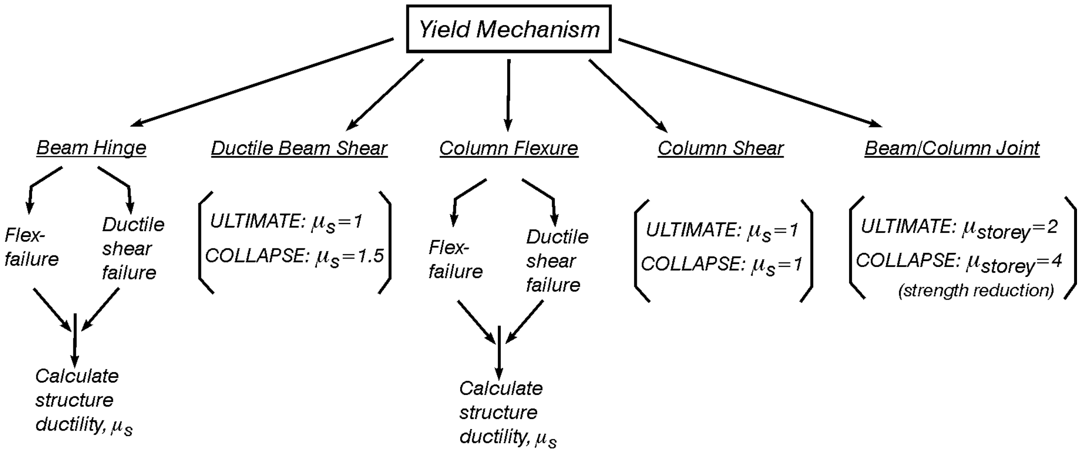


Figure 4E.8 Frame ultimate displacement capacity

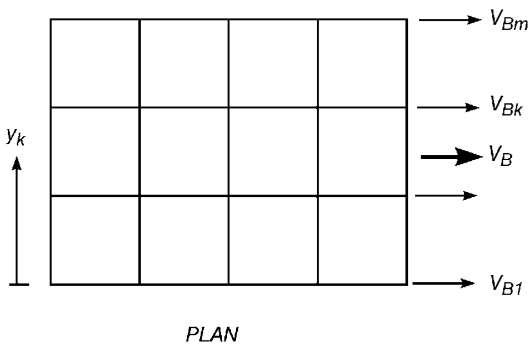


Figure 4E.9 Strength eccentricity

8. Determine which frame is subjected to the critical ultimate displacement.

9. Taking twist into account, determine the ultimate displacement, μ_u, at the centre of mass (or the displacement, μ_c, at collapse).

10. The structure displacement capacity is then given by

$$\mu_{sc} = \Delta_u / \Delta_y$$

11. Determine the elastic stiffness, K_e, where

$$K_e \;=\; V_B / \Delta_y$$

12. Determine the effective mass, M_e, from

$$M_e = \frac{\sum m_i h_i}{h_e}$$

Also check the situation where the effective mass in the first mode is less than 100%.

13. Determine the elastic period, T, as

$$T = 2\pi\sqrt{\frac{M_e}{K_e}}$$

14. The ductility demend, μ_{SD}, can be determined from (V_B / M_e)and the spectrum.

15. The (*%NBS*) is given by:

$$(\%NBS) = \frac{\mu_{SC}}{\mu_{SD}}$$

4E.10.4 Displacement-based Assessment of Demand

1. Determine the overturning moment for each frame of the structure as for the force-based assessment (FBA).

2. Determine the overturning moment for the structure as for the FBA

3. Determine the ultimate displacement profile for each frame.

4. Determine the effective height as:

$$h_{eff} = \frac{\sum m_i \Delta_i h_i}{\sum m_i \Delta_i}$$

5. Determine the base shear capacity, V_B, as for FBA.

6. Determine the yield displacement, Δ_y, as for FBA.

7. The structure ultimate displacement capacity, Δ_{UC}, can be determined as in steps 7-10 for FBA.

8. The effective mass is determined by

$$M = \frac{\sum m_i \Delta_i}{\Delta_{UC}}$$

Check the situation where the effective mass is less than 100% in the first mode.

9. The effective stiffness is:

$$K_e = V_B / \Delta_{UC}$$

10. The effective damping, ξ_{eff}, needs to be determined for the particular μ_{SC} (= μ_u / μ_y) using Equation 6(3).

11. Calculate the effective period as in step 13 of the FBA.

12. Calculate the displacement demand, μ_{UD}, from the displacement spectrum and the effective damping.

13. Calculate the (*%NBS*) as

$$(\%NBS) = \frac{\mu_{UC}}{\mu_{UD}}$$

4E.11 Lateral Pushover Analysis

If the Nonlinear Static Procedure (NSP) is selected for seismic analysis of the building, a mathematical model directly incorporating the nonlinear load-deformation characteristics of individual components and elements of the building shall be subjected to monotonically increasing lateral loads representing inertia forces in an earthquake until a target displacement is exceeded. Mathematical modeling and analysis procedures should comply with the requirements of Section 4E.11.1

The target displacement is intended to represent the maximum displacement likely to be experienced during the design earthquake. Because the mathematical model accounts directly for effects of material inelastic response, the calculated internal forces will be reasonable approximations of those expected during the design earthquake. A method for determining suitable target displacements is described in Section 3.3.3.3 of FEMA 356 (2000).

4E.11.1 Modelling and Analysis Considerations

The selection of a control node, the selection of lateral load patterns, the determination of the fundamental period, and analysis procedures should comply with the requirements of this section.

The relation between base shear force and lateral displacement of the control node should be established for control node displacements ranging between zero and 150% of the target displacement, Δ_t.

The component gravity loads should be included in the mathematical model for combination with lateral loads as specified in AS/NZS 1170.0. The lateral loads should be applied in both the positive and negative directions, and the maximum seismic effects should be used for design.

The analysis model is discretised to represent the load-deformation response of each component along its length to identify locations of inelastic action. All primary and secondary lateral-force-resisting elements should be included in the model.

The force-displacement behavior of all components can be explicitly included in the model using full backbone curves that include strength degradation and residual strength, if any.

Alternatively, a simplified analysis can be used. In such an analysis, only primary lateral force resisting elements are modeled, the force-displacement characteristics of such elements are bilinear, and the degrading portion of the backbone curve is not explicitly modeled. Elements not meeting the acceptance criteria for primary components are designated as secondary, and removed from the mathematical model.

When using the simplified analysis, care should be taken to make sure that removal of degraded elements from the model does not result changes in the regularity of the structure that would significantly alter the dynamic response. In pushing with a static load pattern, the simplified analysis does not capture changes in the dynamic characteristics of the structure as yielding and degradation take place.

In order to explicitly evaluate deformation demands on secondary elements that are to be excluded from the model, one might consider including them in the model, but with negligible stiffness, to obtain deformations demands without significantly affecting the overall response.

4E.11.2 Control Node Displacement

The control node should be located at the center of mass at the roof of a building. For buildings with a penthouse, the floor of the penthouse should be regarded as the level of the control node. The displacement of the control node in the mathematical model should be determined for the specified lateral loads.

4E.11.3 Lateral Load Distribution

Lateral loads are applied to the mathematical model in proportion to the distribution of inertia forces in the plane of each floor diaphragm. For all analyses, at least two vertical distributions of lateral load should be applied. One pattern shall be selected from each of the following two groups:

1. A modal pattern selected from one of the following:

    a) A vertical distribution proportional to the values of C_{vx} given in eqn 4E(10). Use of this distribution should be used only when more than 75% of the total mass participates in the fundamental mode in the direction under consideration, and the uniform distribution is also used.
    b) A vertical distribution proportional to the shape of the fundamental mode in the direction under consideration. Use of this distribution should be used only when more than 75% of the total mass participates in this mode.
    c) A vertical distribution proportional to the story shear distribution calculated by combining modal responses from a response spectrum analysis of the building, including sufficient modes to capture at least 90% of the total building mass, and using the appropriate ground motion spectrum. This distribution should be used when the period of the fundamental mode exceeds 1.0 second.

2. A second pattern selected from one of the following:

    a) A uniform distribution consisting of lateral forces at each level proportional to the total mass at each level.
    b) An adaptive load distribution that changes as the structure is displaced. The adaptive load distribution should be modified from the original load distribution using a procedure that considers the properties of the yielded structure.

The distribution of lateral inertial forces determines relative magnitudes of shears, moments, and deformations within the structure. The distribution of these forces will vary continuously during earthquake response as portions of the structure yield and stiffness characteristics change. The extremes of this distribution will depend on the severity of the earthquake shaking and the degree of nonlinear response of the structure. Use of more than one lateral load pattern is intended to bound the range of design actions that may occur during actual dynamic response.

In lieu of using the uniform distribution to bound the solution, changes in the distribution of lateral inertial forces can be investigated using adaptive load patterns that change as the structure is displaced to larger amplitudes. Procedures for developing adaptive load patterns include the use of story forces proportional to the deflected shape of the structure (Fajfar and Fischinger), the use of load patterns based on mode shapes derived from secant stiffnesses at each load step (Eberhard and Sozen), and the use of load patterns proportional to the story shear resistance at each step (Bracci et al.). Use of an adaptive load pattern will require more analysis effort, but may yield results that are more consistent with the characteristics of the building under consideration.

4E.12 Inelastic Time History Analysis

Where an inelastic time history analysis carried out for the seismic analysis of the building, a mathematical model directly incorporating the nonlinear load-deformation characteristics of individual components and elements of the building should be subjected to earthquake shaking represented by ground motion time histories in accordance with Clause 6.4 of NZS 1170.5:2004 to obtain forces and displacements.

The calculated response can be highly sensitive to characteristics of individual ground motions; therefore, the analysis should be carried out with more than one ground motion record. Because the numerical model accounts directly for effects of material inelastic response, the calculated internal forces will be reasonable approximations of those expected during the design earthquake.

Appendix 8A: Bolted and Riveted Joint Moment-Rotation Determination

8A.1 Clip angle type connections

A comprehensive procedure for evaluating the nominal moment capacity and rotation available from riveted or early bolted steel connections is given in (Roeder et al 1996). This procedure is applicable for beam to column connections formed with either tee-stub or clip angle connections between beam flange and column flange, as shown in Fig. 1 of (Roeder et al. 1996).

The procedure includes a method for calculating the effective yield moment for a riveted connection, along with expressions for the rotational capacity at maximum strength of the connection, (ie. the rotation limit above which the moment capacity falls significantly below that given by calculated nominal yield moment. Both yield moment and degradation threshold are a function of the expected mode of failure of the connection to the beam flanges. Roeder et al (1996). require three modes of failure to be checked for the critical case, ie.:

(1) Tensile failure of the stem or outstanding leg (OSL) of the angle or tee section connection onto the supported beam flange.
(2) Shear yielding/failure of the connectors, and
(3) Flexural yielding of the leg(s) of the angle or tee-stem connecting onto the supporting column flange.

The failure mode giving the least capacity of these three becomes the failure mode for the connection, in terms of this evaluation.

Most older riveted or bolted beam to column joints in New Zealand have used clip angles, as shown in Fig. 8A.1. A simplified procedure for calculating the yield moment and the moment-rotation characteristics is given below.

This procedure is based around the critical failure mode being that associated with flexural yielding of the legs of the angle or tee-section connecting onto the column. The first two failure modes need to also be assessed and only when the third failure mode is shown to govern can the procedure given in this simplified section be used.

If either tensile failure or shear yielding /failure of the connectors governs, then use the procedure in Section 8A.2.

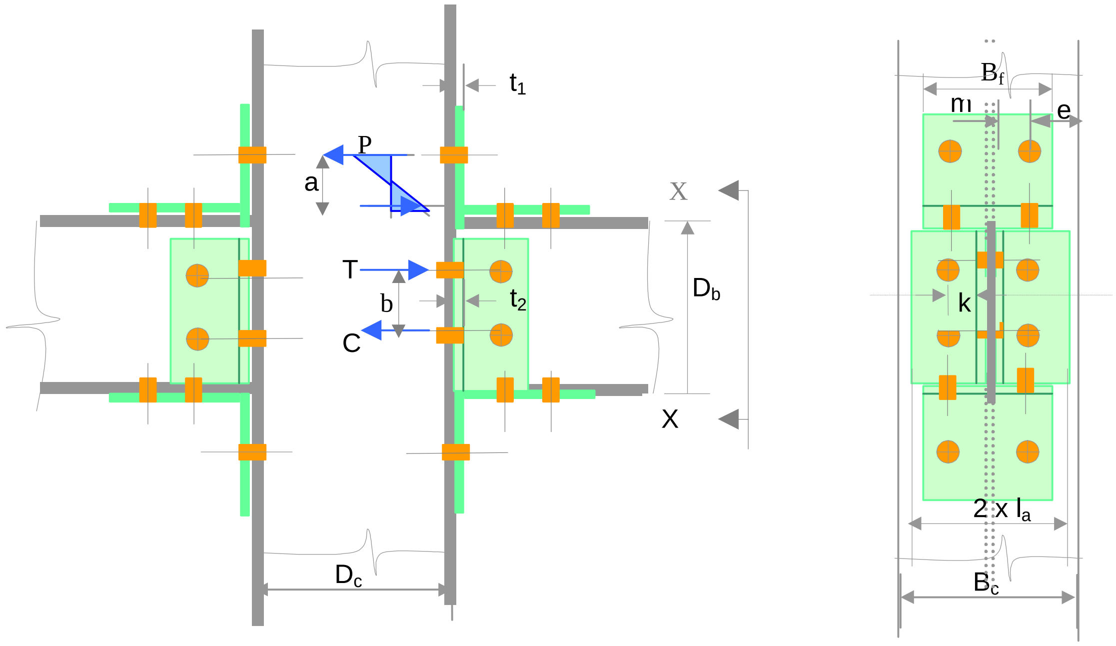


Figure 8A.1: Joint detail

Bending moment capacity of flange cleat angle:

$$M_f = \frac{B_f t_1^2}{4} f_{ya} \qquad \text{...8A(1)}$$

where B_f is minimum of (beam flange width; angle length), t_1 is thickness of flange cleat angle leg, and f_{ya} is design yield strength of the angle section.

From eqn 8A(1), tensile force in the flange cleat bolts/rivets:

$$P = \frac{2M_f}{a} \qquad \text{...8A(2)}$$

where a is the distance between bolt centreline to the flange cleat angle leg.

Bending moment capacity of web cleat angle:

$$M_w = \frac{2l_a t_2^2}{4} f_{ya} \qquad \text{...8A(3)}$$

where l_a is the length of web cleat angle face and t_2 is thickness of web cleat angle leg.

From eqn 8A(3), tensile force in the web cleat bolts/rivets:

$$T = \frac{2M_w}{k} \qquad \text{...8A(4)}$$

where k is the distance between bolt centreline to the web cleat angle leg.

Tension strength of the column flange:

$$T_c = (4m + 1.25e) t_c f_{yc} \qquad \text{...8A(5)}$$

where m is the distance from centre of bolt hole to radius root at web, e is distance from rivet centre to flange edge, and t_c is thickness of the column flange and f_{yc} is the yield stress of the column flange.

Yield moment capacity of the joint is:

$$M_y = PD_b + Qb \qquad \ldots 8A(6)$$

where Q is either T from eqn 8A(4) or T_c from eqn 8A(5), whichever is less, and b is the distance between the centroid of tension and compression forces in the web cleat.

8A.1.1 Moment – rotation behaviour

Figure 8A.2 shows the proposed moment–rotation behaviour of riveted clip angle/T-stub connection based on Roeder et al experimental studies on seismic resistance on older steel structures at the University of Washington and University of Minnesota (Roeder et al 1994).

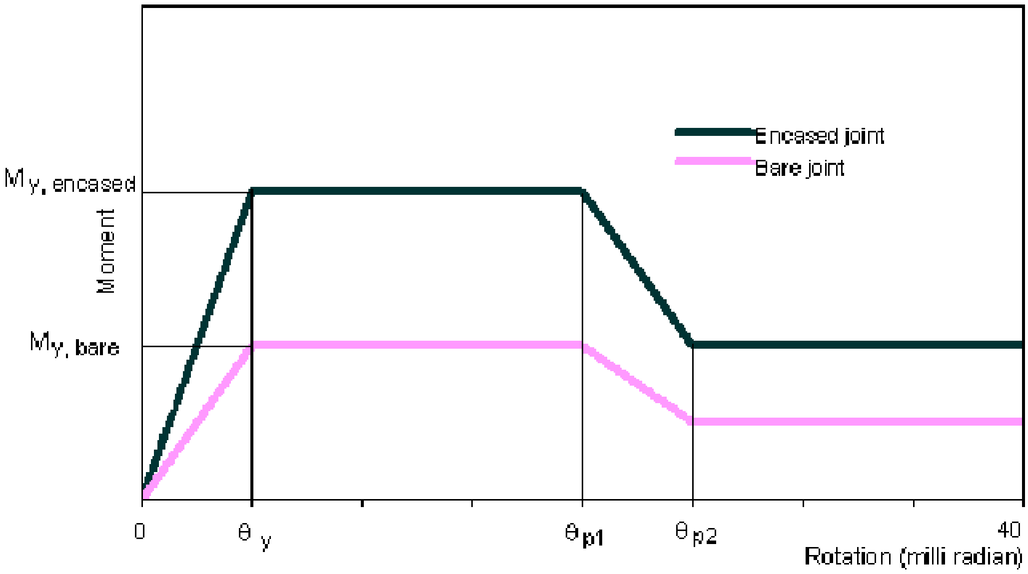


Figure 8A.2: Moment–rotation curve

In Fig. 8A.2

θ_y = 5 milliradians, for a clip angle type connection

θ_{p1} = $\dfrac{12.5}{d_b}$ milliradians …8A(7)

d_b = depth of beam, in metres

θ_{p2} = $(\theta_{p1} + 5)$ milliradians …8A(8)

$M_{y,bare}$ = as given by eqn 8A(6) for a bare steel connection

$M_{y,encased}$ = 2 $M_{y,bare}$ for a clip angle type connection …8A(9)

When the joint is rotated from θ_{p1} to θ_{p2} , the moment reduces by a factor of 0.5 and then remains constant up to θ = 40 milliradians, after which zero moment capacity is assumed.

In regard to the above:

- θ_y = 5 milliradians is an appropriate rotation at first yield for this pre-1975 building connection
- eqn 8A(7) is from (Roeder et al. 1996), for connections with flexural yield of connecting elements
- the experimental tests undertaken show that the degradation in moment capacity occurs over a rotation of approx. 5 milliradians, hence this is the difference used between θ_{p1} and θ_{p2}.
- the enhancement factor for $M_{y,encased}$ compared with $M_{y,bare}$ is that recommended by (Roeder et al. 1996) for this, the most flexible form of semi-rigid connection.

8A.1.2 Joint deterioration

The joints tested by Roeder, both concrete encased and bare joints generally experienced degradation at rotation 20–25 milliradians. It was also observed that the concrete encased composite joint had a better performance over the bare joints. The concrete encasement prevented any local deformation of the joint until the concrete crushed when the joint capacity deteriorates to that of a bare joint. The enhancement provided by the composite action of concrete encasement and floor slabs to connection capacity was found to be substantial and in the range of 30–100% increase to that of bare joint moment capacity. The higher increase of capacity was noted in the weaker joints such as clip angles.

In bare joints without concrete encasement the joint capacity deteriorated significantly when the clip angle to the beam flange failed but the capacity did not drop to zero because of the resistance provided by the web cleat angle connection.

8A.1.3 Background to Roeder's experiments

Roeder et al (1994, 1996) focused their experimental work on issues that were not addressed previously by researchers in determining the seismic resistance of older steel building. Some of the key objectives of their work were:

- ▶ to study the cyclic behaviour of these older steel structures considering the change in stiffness at large inelastic deformation. The past research work were primarily under monotonic loading.
- ▶ to study the effect of concrete encasement provided for fire resistance on connection stiffness, strength, and ductility
- ▶ to understand the effect of rivets on seismic behaviour of joints
- ▶ to develop a model to establish the strength, stiffness, and ductility of these older steel structures based on their experiments.

The research work was a joint effort between the University of Washington, the University of Minnesota, and Preece/Goudie & Associates. As part of the testing programme they tested 23 large-scale specimens including bare steel and encased joints with clip angle, T-stub, and stiffened seat connections.
The main findings of the research were:

1 The hysteretic behaviour of the connections was relatively poor but the connections often were able to sustain large deformations. They behaved as partially restrained connections. Clip angle connections were generally weaker and more flexible than the other connections.

2 Concrete encasement significantly increased the strength and stiffness of weaker and more flexible joints such as clip angle connections and modestly increased for stiffer and stronger connections. See Figure 8A.3 taken from Roeder (1994).

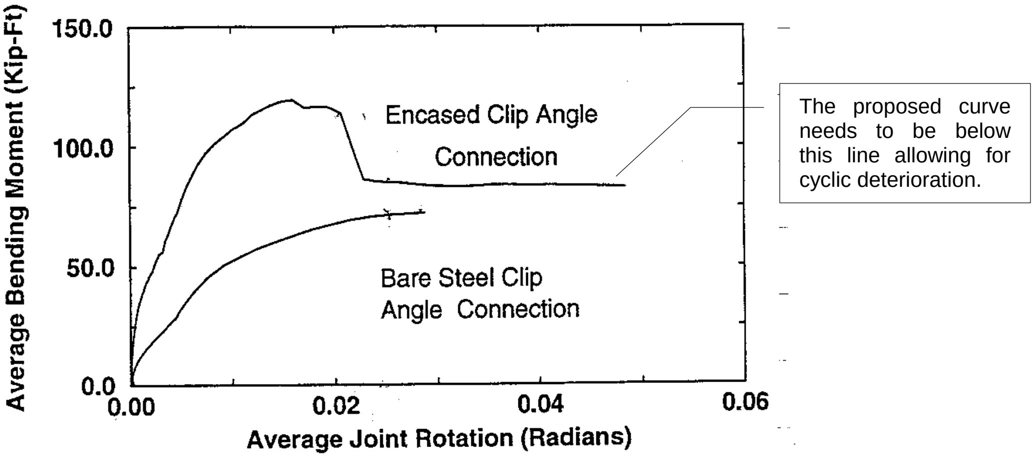


Figure 8A.3: Comparison of bare steel and encased moment–rotation behaviour

3 The tests showed that mode of failure for the cyclic loading was very similar to the monotonic loading. Both monotonic and cyclic load tests deteriorate or fail at very similar deformations as shown in Figure 8A.4. The monotonic tests typically provided an upper bound envelope for the cyclic tests. T-stub and clip angle connections for both bare steel and encased connections displayed this behaviour.

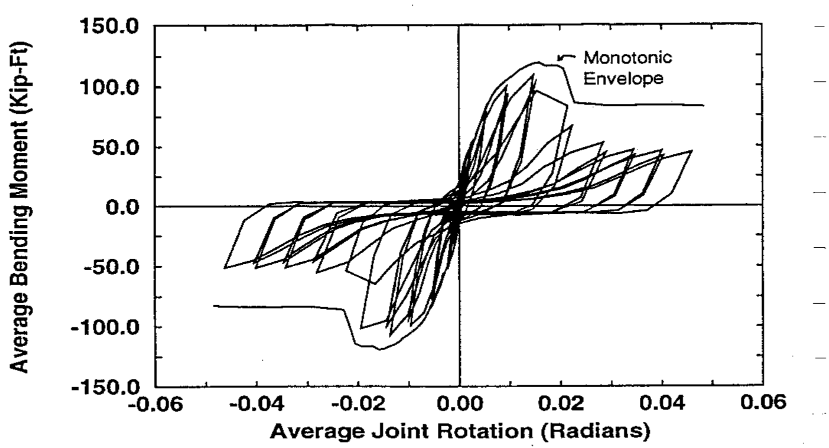


Figure 8A.4: Comparison of monotonic and cyclic moment–rotation behaviour

4 All connectors failed at almost the identical deformation for both bare steel and encased connection. However, the initial failure of these connectors did not result in a complete loss of the resistance of the connection. See the moment rotation behaviour in Figure 8A.2. Considerable resistance was provided by the web angles and composite action provided by the concrete encasement even after the initial failure.

The above experimental studies were on riveted connections. It should be noted that bolted connections would be stiffer and have more rotational capacity than the comparable riveted connections. However, the limits on the overall system inelastic displacement would be such that the bolted connections cannot attain its full capacity. For example, when the connection is the *weakest element, then the connection rotation will be around 30 milliradians maximum for a frame displacement of 2.5% of the interstorey height. Thus the 40 milliradians limit on rotation is a practical upper limit for the system as a whole, even if the individual joint is capable of greater rotations while maintaining a dependable level of moment capacity.*

8A.2 Other bolted and riveted connections

For bolted and riveted connections in general - especially other than clip angle connections of the form shown by Fig. 8A.1 – use the procedure from (Roeder et al., 1996) to determine the moment capacity M_y. (This is termed M_u in that paper). This involves using the seven step procedure on pages 370 and 371 of that paper.

The moment-rotation curve is then constructed in a similar manner to Fig. 8A.2, using the following key values for rotation and moment:

θ_y = 5 milliradians for clip angle connections
= 3 milliradians for tee stub connections

$\theta_{p1} = \frac{3.75}{d_b}$ milliradians, for failure mode being tensile yielding of the stem of the tee stub or clip angle connected to the beam flange …8A(10)

$\theta_{p1} = \frac{7.5}{d_b}$ milliradians, for failure mode being shear yielding of the connectors …8A(11)

θ_{p1} = as given by eqn 8A(7), for failure mode being flexural yielding of the connecting elements

$\theta_{p2} = (\theta_{p1} + 5)$ milliradians …8A(12)

$M_{y,\,bare}$ = as given by (Roeder et al., 1996)

$M_{y,\,encased} = C_1 M_{y,\,bare}$ …8A(13)

C_1 = 1.3 for a tee-stub type connection
= 2.0 for a clip angle type connection

Appendix 8B: Simplified Pushover Analysis for Use in the Evaluation

The analysis must have the capability to take into account the P-Δ action by large displacement analysis and the modelling of joint elastic springs in the system.

1 Take the force vector from Section 4.9.7(d) and assign a unit Load Factor (LF) to it.

2 Increase the LF until past the yield moment (M_{yield}) in approximately one-quarter of the joints on any level.

3 Reduce the joint elastic stiffness on that level to the first inelastic value, that is, as shown on Figure 8B.1 and reapply loads using LF_{max} from Step 2.

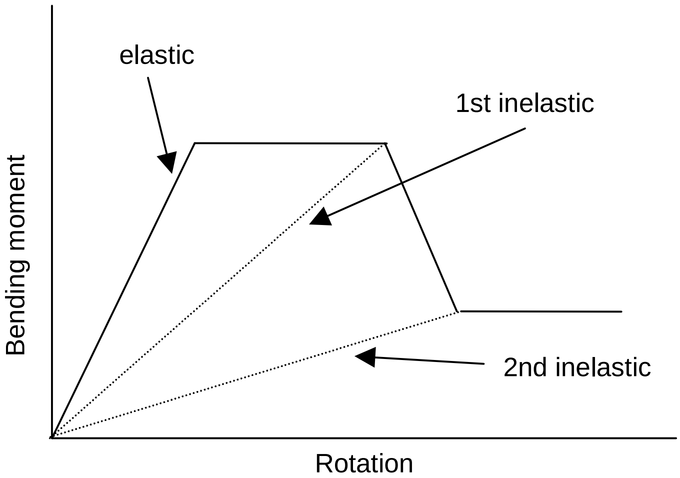


Figure 8B.1: Moment–rotation curve for riveted clip-angle/T – stub connection

4 Check all levels to see if M_{yield} is exceeded in approximately one-quarter of the joints. If so, reduce the joint elastic stiffness in all joints on that level and reanalyse. Keep the top one-third (or three) joints elastic throughout to model the concentration of demand in lower levels.

5 Check the rotation in the joints at the lower levels. If > 20 milliradians, then reduce the joint stiffness to the 2nd inelastic level and reanalyse. Reduce LF if necessary to keep within the deflection limits if these limits are exceeded when the joint stiffness on a given layer is reduced to the second inelastic level.

6 When the deflection limit is attained, check if LF ≥ 0.8 LF_{max}.

Appendix 10A: Derivation of Instability Deflection and Fundamental Period for Masonry Buildings

10A.1 General considerations and approximations

It should be appreciated that there are many variations that need to be taken into account in considering a general formulation for unreinforced masonry walls that might fail out-of-plane. Among these considerations are the following.

- Walls will not in general be of constant thickness in a building, or even within a storey.
- Walls will have embellishments, appendages and ornamentation that may lead to eccentricity of masses with respect to supports.
- Walls may have openings for windows or doors.
- Support conditions will vary.
- Existing building may be rather flexible, leading to possibly large inter-storey displacements that may adversely affect the performance of face-loaded walls.

To simplify the analysis while taking into account important factors, the following are the approximations that are employed.

1 Deformations due to distortions (straining) in the wall are ignored. Deflections are assumed to be entirely due to rigid body motion.

This is equivalent to saying that the change in potential energy due to a disturbance of the wall from its initial position is due mostly to the movement of the masses of the elements comprising the wall and the movements of the masses tributary to the wall. Strain energy contributes less to the change in potential energy.

2 It is assumed that potential rocking occurs at the support lines (at roof or floor levels, for example) and, for walls that are supported at the top and bottom of a storey, at the mid-height. The mid-height rocking position divides the wall into two parts of equal height, a bottom part (subscript b) and a top part (subscript t). The masses of each part are not necessarily equal.

It is implicit within this assumption and that in (1) above, that the two parts of the wall remain undistorted when the wall deflects. For walls constructed of softer mortars or for walls where there is little vertical prestress from storeys above, this is not actually what occurs—the wall takes up a curved shape, more particularly in the upper part. Nevertheless, the errors that occur from the use of the stated assumptions have been found to be small and acceptably accurate results are still obtained.

3 The thickness is assumed to be small relative to the height of the wall, and the slope, A, of both halves of the wall is assumed to be small, in the sense that $\cos(A) \approx 1$ and $\sin(A) \approx A$.

The approximations for slope are likely to be sufficiently accurate for reasonably thin walls. For thick walls where the height to thickness ratio is smaller, the formulations that are developed in this appendix are likely to provide less accurate results. However, for walls of this kind force-based approaches provide an alternative.

4 Inter-storey slopes due to deflection of the building are assumed to be small.

Approximate corrections for this effect are noted in the method.

5 In dynamic analyses, the moment of inertia is assumed constant and equal to that applying when the wall is in its undisturbed position, whatever the axes of rotation.

It should be appreciated that the moment of inertia is dependent on the axes of rotation. During excitation the axes continually change position. The approximation assumes that the inertia is constant. Within the context of other approximations employed, this is reasonable.

6 Damping is assumed at the default value in NZS 1170.5:2004 (or NZS 4203:1992), which is 5% of critical.

For the aspect ratio of walls of interest, additional effective damping due to loss of energy on impact is small. Furthermore it has been found that the surfaces at rocking (or hinge) lines tend to fold onto each other rather than experience the full impact that is theoretically possible, reducing the amount of equivalent damping that might be expected. However, for in-plane analysis of buildings constructed largely of unreinforced masonry, adoption of a damping ratio that is significantly greater than 5% is appropriate.

7 It is assumed that all walls in storeys above and below the wall under study move "in phase" with the subject wall.

This is found to be the case in analytical studies. One reason for this is that the effective stiffness of a wall as it moves close to its limit deflection (as measured by its period, for example) becomes very low, affecting its resistance to further deflection caused by accelerations transmitted to the walls through the supports. This assumption means that upper walls, for example, will tend to restrain the subject wall by exerting restraining moments.

10A.2 Case 1: One-way vertically spanning face-loaded walls

10A.2.1 General formulation

Figures 10A.1 and 10A.2 show the configuration of a wall panel within a storey at two stages of deflection. The wall is intended to be quite general. Simplifications to the general solutions for walls that are simpler (e.g. of uniform thickness) are made in a later section.

Figure 10A.1 shows the configuration at incipient rocking. Figure 10A.2 shows the configuration after significant rocking has occurred, with the wall having rotated through an angle A and with mid-height deflection Δ, where $\Delta = Ah/2$.

In Figure 10A.1 the dimensions e_b and e_t relate to the mass centroids of the upper and lower parts of the panel. e_p relates to the position of the line of action of weights from upper storeys (walls, floors and roofs) relative to the centroid of the upper part of the panel. The arrows on the associated dimensioning lines indicate the positive direction of these dimensions for the assumed direction of motion (angle A at the bottom of the wall is positive in the anti-clockwise sense). Under some circumstances the signs of the eccentricities may be negative, for example for e_p when an upper storey wall is much thinner than the upper storey wall represented here, particularly where the thickness steps on one face.

In the figures the instantaneous centres of rotation (marked ICR) are shown. These are useful in deriving virtual work expressions.

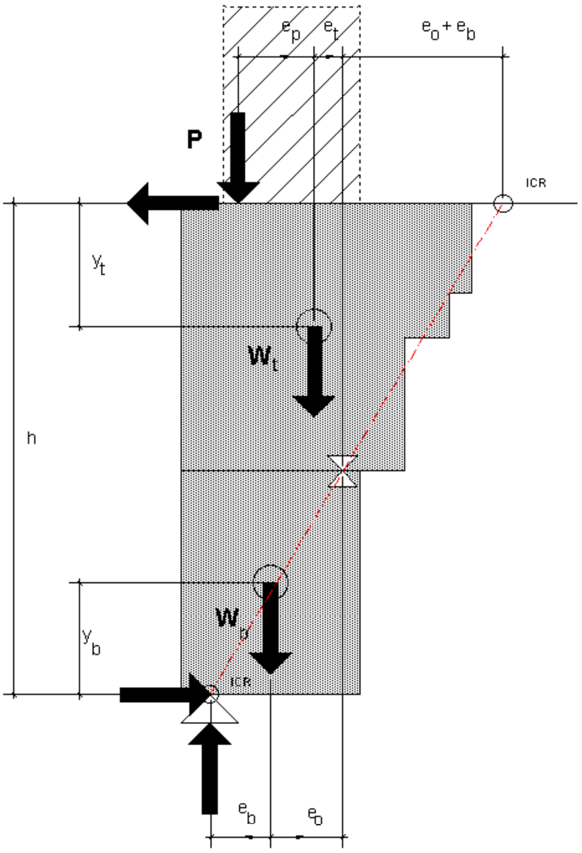


Figure 10A.1: Configuration at incipient rocking

10A.2.2 Limiting deflection for static instability

With reference to Figure 10A.2, and using virtual work, the equation of equilibrium can be directly written. For static conditions this is given by:

$$W_b(e_b - Ay_b) + W_t\left(e_o + e_b + e_t - A\left(\frac{h}{2} + y_t\right)\right) + P(e_o + e_b + e_t + e_p - Ah) = 0 \qquad \ldots 10A(1)$$

Writing:

$$a = W_b y_b + W_t\left(\frac{h}{2} + y_t\right) + Ph \qquad \ldots 10A(2)$$

and

$$b = W_b e_b + W_t(e_o + e_b + e_t) + P(e_o + e_b + e_t + e_p) \qquad \ldots 10A(3)$$

and collecting terms in *A*, the equation of equilibrium is rewritten as:

$$-aA + b = 0 \qquad \ldots 10A(4)$$

from which:

$$A = \frac{b}{a} \qquad \ldots 10A(5)$$

when the wall becomes unstable.

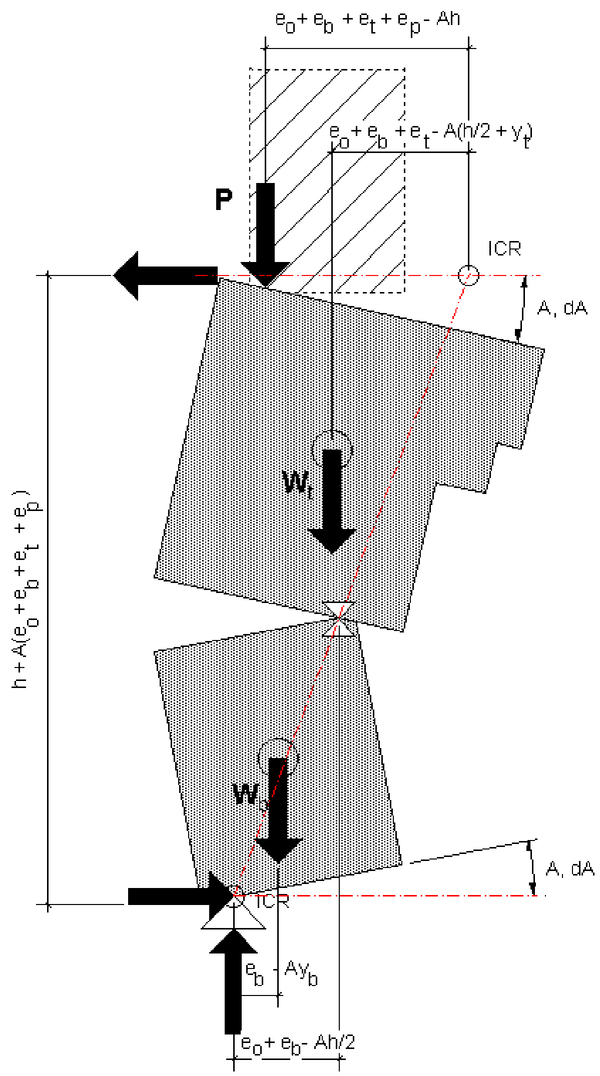


Figure 10A.2: Configuration when rotations have become significant

The critical value of the deflection at mid-height of the panel, at which the panel will be unstable, is therefore:

$$\Delta_i = A\frac{h}{2} = \frac{bh}{2a} \qquad \text{...10A(6)}$$

It is assumed that Δ_m, a fraction of this deflection, is the maximum useful deflection. Experimental and analytic studies indicate that this fraction might be assumed to be about 0.6. At larger displacements that $0.6\Delta_i$, analysis reveals an undue sensitivity to earthquake spectral content and a wide scatter in results. Some compensation is made for taking this fraction as less than unity when the final assessment for the likely performance of the wall is made.

10A.2.3 Equation of motion for free vibration

When conditions are not static the virtual work expression on the left-hand side in the equation above is unchanged, but the zero on the right-hand side of the equation is replaced by the mass times acceleration, in accordance with Newton's law. Thus we have:

$$-aA+b=-J\ddot{A} \qquad \text{…10A(7)}$$

where the usual notation for acceleration using a double dot to denote the second derivative with respect to time is used, in this case indicating angular acceleration, and J is the rotational inertia.

The rotational inertia can be written directly from the figures, noting that the centroids undergo accelerations vertically and horizontally as well as rotationally, and noting that these accelerations relate to the angular acceleration in the same way as the displacements relate to the angular displacement. While the rotational inertia is dependent on the displacements, the effects of this variation are ignored. Accordingly the rotational inertia is taken as that when no displacement has occurred. This then gives the following expression for the rotational inertia.

$$J = J_{bo} + J_{to} + \frac{1}{g}\left\{W_b\left[e_b{}^2 + y_b{}^2\right] + W_t\left[(e_o + e_b + e_t)^2 + y_t{}^2\right] + P\left[(e_o + e_b + e_t + e_p)^2\right]\right\} + J_{anc} \qquad \text{…10A(8)}$$

where J_{bo} and J_{to} are respectively the moments of inertia of the bottom and top parts about their centroids, and J_{anc} is the inertia of any ancillary masses, such as veneers, that are not integral with the wall but that contribute to its inertia.

Note that in this equation the expressions in square brackets are the squares of the radii from the instantaneous centres of rotation to the mass centroids, where the locations of the instantaneous centres of rotation are those when there is no displacement. Some CAD programs have functions that will assist in determining the inertia about an arbitrary point (or locus), such as about the ICR shown in Figure 10A.2.

Collecting terms and normalising the equation so that the coefficient of the acceleration term is unity, we have the following differential equation of free vibration.

$$\ddot{A} - \frac{a}{J}A = -\frac{b}{J} \qquad \text{…10A(9)}$$

10A.2.4 Period of free vibration

The solution of the equation for free vibration derived in the previous section is:

$$A = C_1 \sinh\left(\sqrt{\frac{a}{J}}\tau\right) + C_2 \cosh\left(\sqrt{\frac{a}{J}}\tau\right) + \frac{b}{a} \qquad \text{…10A(10)}$$

The time, τ, is taken as zero when the wall has its maximum rotation, A $(=\Delta/2h)$. Using this condition and the condition that the rotational velocity is zero when the time $\tau = 0$, the solution becomes:

$$A = \left(\frac{2\Delta}{h} - \frac{b}{a}\right)\cosh\left(\sqrt{\frac{a}{J}}\tau\right) + \frac{b}{a} \qquad \text{…10A(11)}$$

For the period of the “part”, T_p, we take it as four times the duration for the wall to move from its position at maximum deflection to the vertical. Then the period is given by:

$$T_p = 4\sqrt{\frac{J}{a}}\cosh^{-1}\left(\frac{\frac{b}{a}}{\frac{b}{a}-\frac{2\Delta}{h}}\right) \qquad \ldots 10A(12)$$

However, this can be further simplified by substituting the term for Δ_i found from the static analysis and putting the maximum value of Δ as Δ_m to give:

$$T_p = 4\sqrt{\frac{J}{a}}\cosh^{-1}\left(\frac{1}{1-\frac{\Delta_m}{\Delta_i}}\right) \qquad \ldots 10A(13)$$

If we accept that the deflection ratio of interest is 0.6, then this becomes:

$$T_p = 6.27\sqrt{\frac{J}{a}} \qquad \ldots 10A(14)$$

10A.2.5 Maximum acceleration

The acceleration required to start rocking of the wall occurs when the wall is in its initial (undisturbed) state. This can be determined from the virtual work equations by assuming that $A=0$. Accordingly:

$$\ddot{A}_{\max} = \frac{b}{J} \qquad \ldots 10A(15)$$

However, a more cautious appraisal assumes that the acceleration is influenced primarily by the instantaneous acceleration of the supports, transmitted to the wall masses, without relief by wall rocking. Accordingly:

$$C_m = \frac{b}{(W_b y_b + W_t y_t)} \qquad \ldots 10A(16)$$

where C_m is the acceleration *coefficient* to just initiate rocking.

10A.2.6 Adjustments required when inter-storey displacement is large

When inter-storey displacement is large, as measured by the slope ψ (equal to the inter-storey displacement divided by the storey height), the following adjustment can be made.

The parameter b is reduced by δb in the determination of the static displacement, where:

$$\delta b = (W_b y_b + W_t y_t)\psi \qquad \ldots 10A(17)$$

Otherwise there is no undue complication. A typical limit on ψ is 0.025.

10A.2.7 Participation Factor

The participation factor can be determined in the usual way by normalising the original form of the differential equation for free vibration, modified by adding the ground acceleration term. For the original form of the equation, the ground acceleration term is added to the RHS. Written in terms of a unit rotation, this term is ($W_b y_b$ + $W_t y_t$) times the ground acceleration. The equation is normalised by dividing through by J, and then multiplied by h/2 to convert it to one involving displacement instead of rotation. The participation factor is then the coefficient of the ground acceleration. That is

$$\gamma = \frac{(W_b y_b + W_t y_t)h}{2Jg} \qquad \ldots 10A(18)$$

10A.2.8 Simplifications for regular walls

Simplifications can be made where the thickness of a wall within a storey is constant, there are no openings and there are no ancillary masses. Further approximations can then be applied:

- The weight of each part (top and bottom) is half the total weight, *W*.
- $y_b = y_t = h/4$
- The moment of inertia of the whole wall is further approximated by assuming that all *e* are very small relative to the height (or, for the same result, ignoring the shift of the ICR from the mid-line of the wall), giving $J = Wh^2/12g$. Alternatively, the simplified expressions for J that are given in Table 10A.1 can be used.

10A.2.9 Approximate displacements for static instability

The following table gives values for *a* and *b* and the resulting mid-height deflection to cause static instability when e_b and/or e_p are either zero or half of the effective thickness of the wall, *t*. In the table e_o and e_t are both assumed to be equal to half the effective wall thickness. While these values of the eccentricities are reasonably common, they are not the only values that will occur in practice.

The effective thickness may be assumed given by the expression:

$$t = \left(0.975 - 0.025\frac{P}{W}\right)t_{nom} \qquad \ldots 10A(19)$$

where t_{nom} is the nominal thickness of the wall.

Experiments show that this is a reasonable approximation, even for walls with soft mortar. Where there is soft mortar, greater damping occurs that reduces response, which compensates for errors in the expression for the effective thickness.

10A.2.10 Approximate expression for period of vibration

Noting that:

$$a = \left(\frac{W}{2} + P\right)h \qquad \ldots 10A(20)$$

and using the approximation for *J* relevant to a wall with large aspect ratio, the expression for the period is given by:

$$T_p = 6.27\sqrt{\frac{2Wh}{12g(W+2P)}} \qquad \text{...10A(21)}$$

where it is to be noted that the period is independent of the restraint conditions at the top and bottom of the wall (i.e. independent of both e_b and e_p).

If the height is expressed in metres, then this expression further simplifies to:

$$T_p = \sqrt{\frac{0.67h}{(1+2P/W)}} \qquad \text{...10A(22)}$$

a value confirmed from experimental results. It should be appreciated that periods may be rather long. For example, if a storey height is 3.6 m and there is no surcharge (i.e. $P=0$), then the period is about 1.55 seconds for an initial displacement that is 60% of the displacement that would cause static instability (typically in the order of the wall thickness – see Table 10A.1).

This approximation errs on the low side, which leads to an under-estimate of displacement demand and therefore to slightly incautious results. The fuller formulation is therefore preferred.

10A.2.11 Participation Factor

Suitable approximations can be made for the participation factor. It could be taken at the maximum value of 1.5. Alternatively, the numerator can be simplified as provided in the following expression, and the simplified value of J shown in Table 10A.1 can be used.

10A.2.12 Maximum acceleration

By making the same simplifications as above, the maximum acceleration is given by:

$$\ddot{A}_{\max} = \frac{b}{J} = \frac{12bg}{Wh^2} \qquad \text{...10A(23)}$$

Or, more cautiously, the *acceleration coefficient*, C_m, is given for the common cases regularly encountered in Table 10A.1.

10A.2.13 Adjustments required when inter-storey displacement is large

Using the common limit on ψ of 0.025, and substituting for $W_b = W_t = W/2$ and $y_b = y_t = h/4$, δb is found to be $Wh/160$. Taking h/t = 25, then, in the absence of any surcharge, the percentage reduction in the instability deflection is as follows for each case shown in Table 10A.1: 31% for Cases 0 and 2; and 16% for Cases 1 and 3. These are not insignificant, and these affects should be assessed especially in buildings with flexible principal framing such as steel moment-resisting frames.

Table 10A.1: Static instability defection for uniform walls, various boundary conditions

Case number	0	1	2	3
e_p	0	0	$t/2$	$t/2$
e_b	0	$t/2$	0	$t/2$
b	$(W/2+P)t$	$(W+3P/2)t$	$(W/2+3P/2)t$	$(W+2P)t$
a	$(W/2+P)h$	$(W/2+P)h$	$(W/2+P)h$	$(W/2+P)h$
$\Delta_i = bh/(2a)$	$t/2$	$\frac{(2W+3P)t}{(2W+4P)}$	$\frac{(W+3P)t}{(2W+4P)}$	t
J	$\{(W/12)[h^2+7t^2]+Pt^2\}/g$	$\{(W/12)[h^2+16t^2]+9Pt^2/4\}/g$	$\{(W/12)[h^2+7t^2]+9Pt^2/4\}/g$	$\{(W/12)[h^2+16t^2]+4Pt^2\}/g$
C_m	$(2+4P/W)t/h$	$(4+6P/W)t/h$	$(2+6P/W)t/h$	$4(1+2P/W)t/h$

10A.3 Case 2: Vertical cantilevers

10A.3.1 General formulation

Figure 10A.2 shows a general arrangement of a cantilever. The wall that is illustrated has an overburden load at the top, but this load will commonly be zero, as in a parapet. Where a load does exist it is important to realise that the mass associated with that load can move horizontally, so that the inertia of the wall is affected by the overburden to a greater extent than for the walls that are supported horizontally at the top. If the top load is supported onto the wall in such a way that its point of application can change, as when it is through a continuous beam or slab that cross the wall, then the formulation for the analysis of the wall will differ from that noted here.

Sometimes several walls will be linked, as when a series of face-loaded walls provide the lateral resistance to a single-storey building. This case can be solved by methods derived from the general formulation, but express formulations for it are not provided here. Refer to examples for particular applications.

For the single wall illustrated, it is assumed that P is applied to the centre of the wall at the top and that point of application remains constant. It is straightforward to obtain the following parameters:

$$a = Ph + Wy_b \qquad \text{...10A(24)}$$

$$b = (P + W)e_b \qquad \text{...10A(25)}$$

$$J = \frac{W}{12g}\left(h^2 + t_{nom}^2\right) + \frac{W}{g}\left[\left(y_b^2 + e_b^2\right)\right] + \frac{P}{g}\left[\left(h^2 + e_b^2\right)\right] \qquad \text{...10A(26)}$$

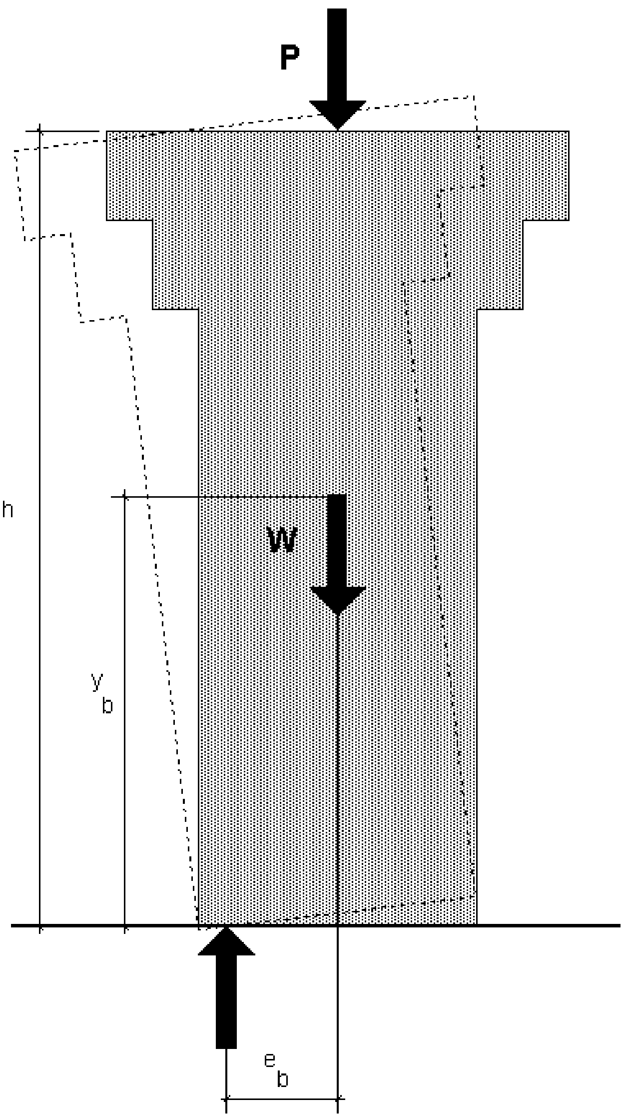


Figure 10A.3: Single cantilever

10A.3.2 Limiting deflection for static instability

When the wall just becomes unstable, the relationship for *A* remains the same as before, but the deflection is *Ah*. Thus, the limiting deflection is given by:

$$\Delta_i = Ah = \frac{bh}{a} = \frac{(P+W)he_b}{Ph+Wy_b} \qquad \ldots 10A(27)$$

For the case where $P=0$ and $y_b=h/2$ this reduces to $\Delta_i = 2e_b = t$.

10A.3.3 Period of vibration

The general expression for period remains valid. Where $P=0$, $e_b=t/2$, $y_b=h/2$, approximating $t=t_{nom}$ and expressing h in metres, the period of vibration is given by:

$$T_p = \sqrt{2.67\left[1+\left(\frac{t}{h}\right)^2\right]} \qquad \ldots 10A(28)$$

10A.3.4 Participation Factor

The expression for the participation factor remains unaffected. That is, $\gamma = Wh^2/2J$. This may be simplified for uniform walls with P=0 (no added load at the top) by inserting the specific expression for J. This gives

$$\gamma = \frac{3}{2\left(1+\left(\frac{t}{h}\right)^2\right)} \qquad \ldots 10A(29)$$

10A.3.5 Maximum acceleration

Using the same simplifications as above:

$$C = \frac{t}{h} \qquad \ldots 10A(30)$$

Appendix 10B: Tests for Assessing the Strength of Masonry and Connectors

Source: 1995 Red Book.

10B.1 Notation

ϕ Strength reduction factor.
v_a Maximum in-plane shear stress at the ultimate limit state.

10B.2 Existing materials

Strength assessments of existing masonry may be made from the results of tests. If testing is undertaken, the results of *all* tests should be recorded and reported.

For unreinforced masonry walls to be considered as structural members providing vertical support to roofs and floors or for resisting lateral loads the following conditions should be satisfied (see Figure 10B.1:

- The bonding of such walls should be such that each face of the wall surface is comprised of headers comprising not less than 4% of the wall surface and extending not less than 90 mm into each wythe.
- The distance between adjacent full-length headers should not exceed 600 mm either vertically or horizontally.
- In walls in which a single header does not extend through the wall, bonders from opposite sides should be covered with another bonder course overlapping the bonder below by at least 90 mm. If the masonry does not comply it should be removed, strengthened, or treated as a veneer or two separate skins.

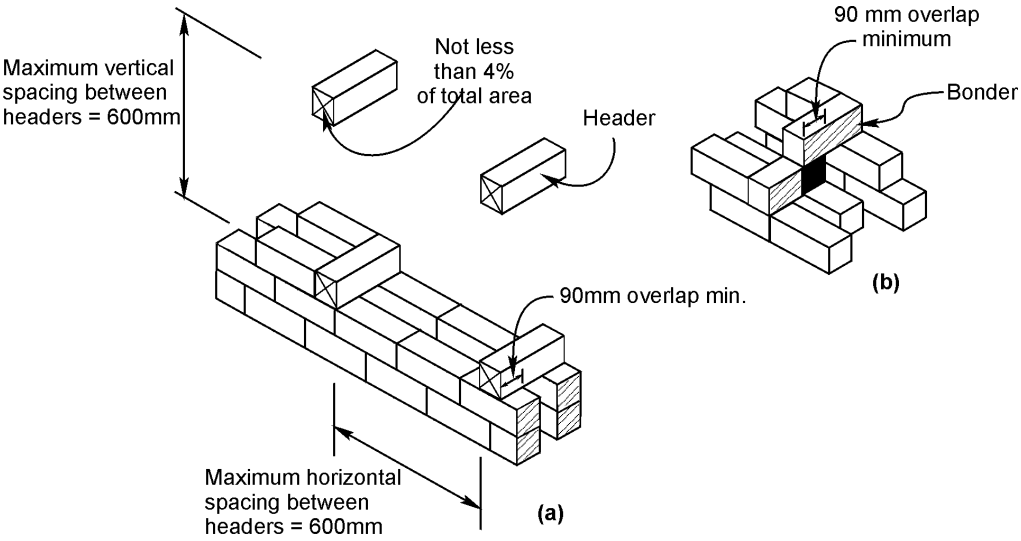


Figure 10 B.1: Bonding requirements for unreinforced masonry walls

10B.3 Tests for Masonry Strengths

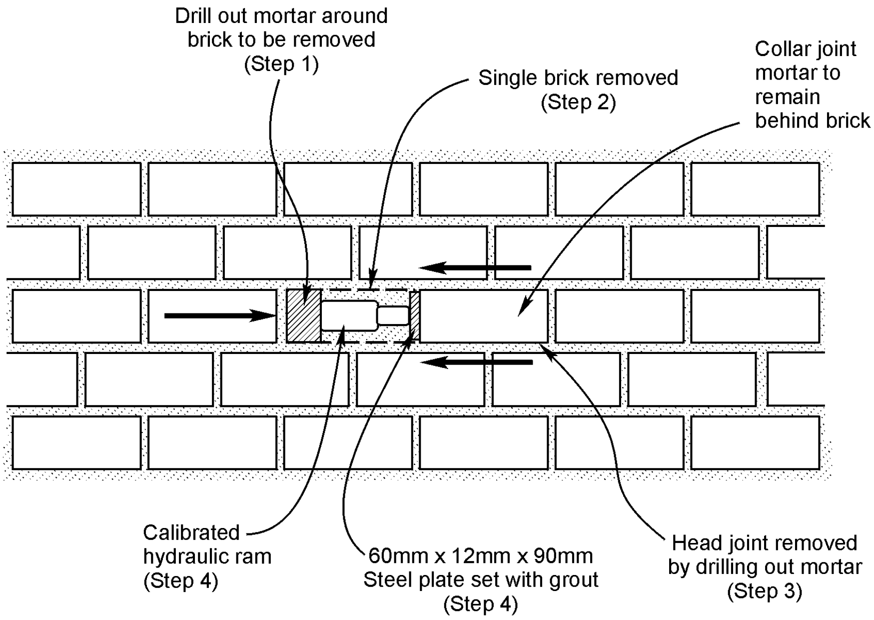
The designer may choose to conduct tests on existing masonry to establish design values. The test procedures described in this section are considered to be acceptable.

10B.3.1 In-place mortar shear test

Note: This test is thought to give unreliable results where the mortar strength has low cohesion. This is because in the process of frictional sliding the expansion of the mortar normal to the sliding plane is prevented, and this gives rise to confining pressures that will not necessarily arise during earthquake response. Core tests or tests on doublets or triplets are therefore generally preferred.

Preparation of sample

The bed joints of the outer wythe of the masonry shall be tested in shear by laterally displacing a single brick relative to the adjacent bricks in the same wythe. The head joint opposite the loaded end of the test brick shall be carefully excavated and cleared. The brick adjacent to the loaded end of the test brick shall be carefully removed by sawing or drilling and excavating to provide space for a hydraulic ram and steel loading blocks (see Figure 10B.2).



Procedure:

1. Existing mortar drilled out with 8mm masonry drill by 100mm long
2. Remove brick
3. Drill out head joint mortar by 100mm deep
4. Install jack and test
5. Test shear strength of mortar (kPa) = $\dfrac{\text{P (Load in Newtons) x 1000}}{\text{2 x Flat area of brick (mm}^2\text{)}}$

Figure 10B.2: In-place mortar shear tests

Application of load and determination of results

Steel blocks, the size of the end of the brick, shall be used on each end of the ram to distribute the load to the brick. The blocks shall not contact the mortar joints. The load shall be applied horizontally, in the plane of the wythe, until either a crack can be seen or a slip occurs. The strength of the mortar shall be calculated by dividing the load at the first cracking or movement of the test brick by the nominal gross area of the sum of the two bed joints.

Test frequency

Test positions shall be distributed such that the conditions are representative of those of the entire structure expected to be utilised for seismic resistance. The minimum number of tests shall be as follows:

a) At each of the first and top storeys, not less than two tests per wall or line of wall elements providing a common line of resistance to lateral forces

b) At all other storeys, not less than one test per wall or line of wall elements providing a common line of resistance to lateral forces.

c) In any case, not less than one test per 500 sq m of wall surface nor less than a total of eight tests.

Determination of design values from tests

The relationship between the test results and the maximum ultimate limit state design shear stress, v_a, is given in Table 10B.1.

10B.3.2 Bed joint shear test

Note: This test will only provide the total shear strength (cohesion and friction). However, the effects of friction are unlikely to be large where the test is undertaken on reasonably competent mortar, so the shear strength recorded might be assigned entirely to cohesion. Alternatively, a representative value of μ may be assumed to enable evaluation of the true cohesion.

Preparation of sample

A core of typically 200 mm diameter shall be taken through the wall, centred on a horizontal mortar joint (see Figure 10B.2).

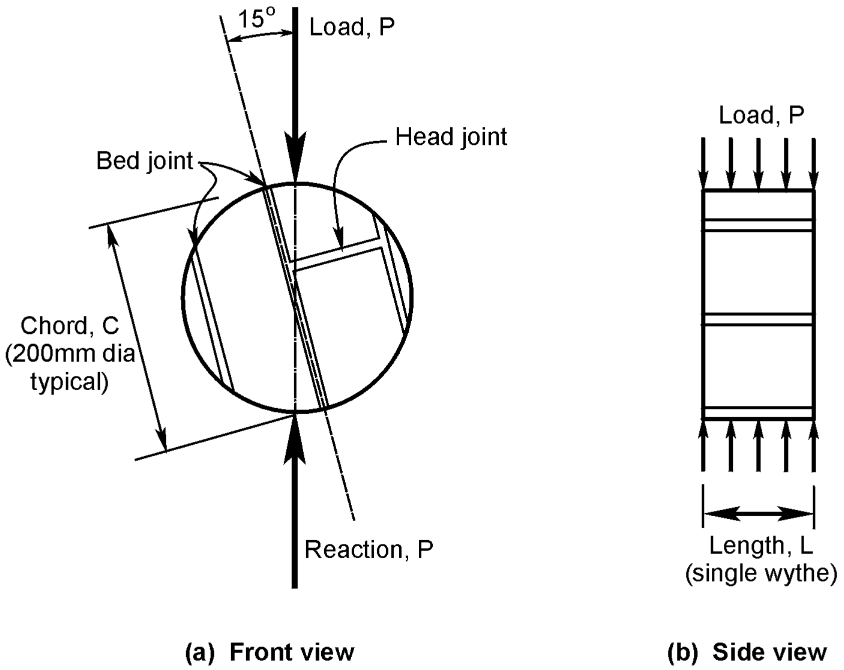


Figure10B.2: Bed joint shear test arrangement

Application of load and determination of results

The core shall be placed between the platens of a compression testing machine with the plane of the horizontal mortar joint aligned at 15° to the vertical. The strength of the mortar shall be calculated by dividing the load at failure by the nominal gross area of the mortar joint.

Test frequencies

Test frequencies shall be as for the in-place mortar shear test.

Determination of design values from tests

The relationship between the test results and the ultimate limit state design shear stress, v_a, is given in Table 4.11B.1.

Table 10B.1: Determination of design values from in-place mortar shear tests and bed joint shear tests

In-place mortar shear	Bed joint shear	Ultimate limit state in-plane shear stress v_a (kPa)
80% of test results not less than (kPa) χ + axial stress	Average test results of cores (kPa) 0.7 χ	 χ (maximum 1000 kPa) (refer note 2)

Notes:

1 These values may only be used when the wall response is not dominated by flexural action (i.e. significant flexural cracking not expected)

2 Shear stress may be increased by the addition of 30% of the dead weight stress of the wall above.

Example of application of Table 4.11B.1: if 80% of in-place mortar shear test results were not less than 400 kPa and the axial stress was 100 kPa, then the ultimate limit state in-plane shear stress would be (400–100) + 0.3(100) = 330 kPa.

If bed joint shear tests were carried out on samples taken from the same location and the average result was 230 kPa, then the ultimate limit state in-plane shear stress would be (210/0.7) + 0.3(100) = 330 kPa.

10B.3.3 Tests on Doublets and Triplets

Testing of doublets and triplets are possibly the best and most reliable means of determining strength parameters of masonry. An advantage of the methods is that clamping forces can be independently varied, so that separate values of friction and cohesion parameters can be obtained.

Figure 10B.3 shows a schematic of a test set-up for doublets. Further information on the testing procedures and details of suitable test rigs are given in Hansen (1999).

Testing on triplets require less sophistication.

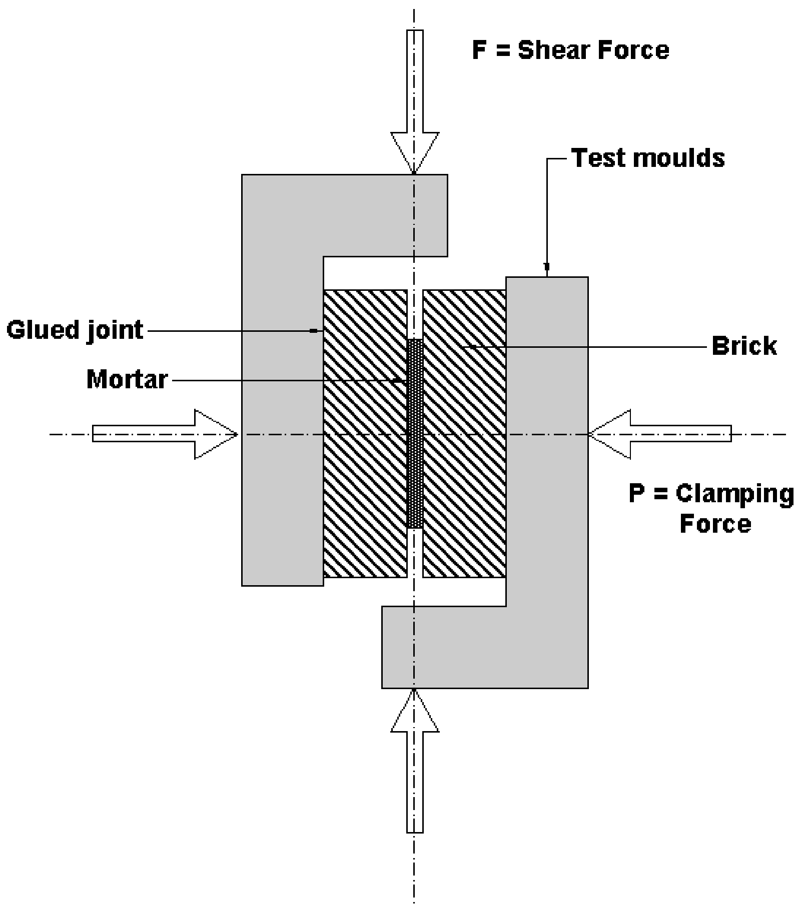


Figure 10B.3: Schematic of an arrangement for testing doublets

10B.4 Tests on Connectors

10B.4.1 Default Strength for Bolts

The following Table 10B.2 lists design strengths that may be adopted for bolts connecting components to masonry. Larger values may be adopted if justified by tests conducted in accordance with b).

Table 10B.2: Default connector strengths

Item	Type	Comment	Strength	ϕ
1	**Shear Connectors**	Bolts should be centred in an oversized hole with non-shrink grout or epoxy resin grout around the circumference.		0.7
	Shear bolts and shear dowels embedded at least 200 mm into unreinforced masonry walls.		M12 bolt: 6 kN M16 bolt: 9 kN M20 bolt: 14 kN	
2	Tension Connectors	The designer should also ensure that the connection to other components is adequate. 25% of all new anchors should be tested to the following torques: —M12: 54 Nm —M16: 68 Nm —M20: 100 Nm		0.7
	Tension bolts extending entirely through the masonry, and secured with a bearing plate at least 138 x 138 or 155 diameter.		29 kN (all sizes)	
	Tension bolts and reinforcing bars grouted (cementitious or epoxy resin) 50 mm less than the thickness of the masonry	Bolts grouted with epoxy may lose strength and fail abruptly id wall cracking occurs at the bolt. The designer should ensure that failure cones from adjacent bolts do not overlap.	11 kN (all sizes)	

10B.4.2 Tension strength of anchors

This section outlines procedures for preliminary testing where the designer may wish to conduct tests on new anchors to derive greater design values than suggested in Table 10B.2.

Application of load and determination of results

The masonry wall should support the test apparatus. The distance between the anchor and the test apparatus support should not be less than the wall thickness. The tension test load reported should be the load recorded at 3 mm relative movement of the anchor and the adjacent masonry surface. For the testing of existing anchors, a preload of 1.5 kN shall be applied prior to establishing a datum for recording elongation. Anchors should be installed in the same manner and using the same materials as intended to be used in the actual construction.

Test frequency

A minimum of *five* tests for each bolt size and type should be undertaken.

Determination of design values from tests

The ultimate limit state strength of tested existing wall anchors should be taken as the mean of all results less 0.8 times the standard deviation for each bolt size. A strength reduction factor of 0.7 should be used to determine the design strength.

Appendix 11A: Timber Diaphragm Stiffness

The mid span deflection of a horizontal diaphragm Δ_h can be calculated from

$$\Delta_h = \Delta_1 + \Delta_2 + \Delta_3 \qquad ...11A(1)$$

where

Δ_1 = diaphragm flexural deformation considering chords acting as a moment resisting couple (mm)
Δ_2 = diaphragm shear deformation resulting from beam action of the diaphragm (mm)
Δ_3 = deformation due to nail slip for horizontal diaphragm (mm)

For transverse sheathing:

$$\Delta_1 = 0$$
$$\Delta_2 = 0$$
$$\Delta_3 = \frac{Le_n}{2s} \qquad …11A(2)$$

For single diagonal sheathing:

$$\Delta_1 = \frac{5WL^3}{192EAB^2} \qquad ...11A(3)$$
$$\Delta_2 = \frac{WL}{4EBt} \qquad …11A(4)$$
$$\Delta_3 = \frac{(1+a)me_n}{2} \qquad …11A(5)$$

For double diagonal sheathing:

$$\Delta_1 = \frac{5WL^3}{192EAB^2} \qquad ...11A(6)$$
$$\Delta_2 = \frac{WL}{8EBt} \qquad …11A(7)$$
$$\Delta_3 = \frac{(1+a)me_n}{2} \qquad ...11A(8)$$

For panel sheathing:

$$\Delta_1 = \frac{5WL^3}{192EAB^2} \qquad ...11A(9)$$
$$\Delta_2 = \frac{WL}{8GBt} \qquad …11A(10)$$
$$\Delta_3 = \frac{(1+a)me_n}{2} \qquad ...11A(11)$$

where

a = Aspect Ratio of each sheathing panel:
= 0 when relative movement along sheet edges is prevented,
= 1 when square sheathing panels are used,
= 2 when 2.4 x 1.2 m panels are orientated with the 2.4 m length parallel with the diaphragm chords (= 0.5 alternative orientation)
A = Sectional area of one chord (mm^2)
B = Distance between diaphragm chord members (mm)
e_n = Nail slip resulting from the shear force V (mm)

E = Elastic modulus of the chord members (MPa)
G = Shear modulus of the sheathing (MPa)
L = Span of a horizontal diaphragm (mm)
m = Number of sheathing panels along the length of the edge chord
t = Thickness of the sheathing (mm)
W = Lateral load applied to a horizontal diaphragm (N)

Appendix 11B: Timber Diaphragm Strength

11B.1 Square sheathing:

The strength of transversely sheathed diaphragms, i.e. where the sheathing runs perpendicular to the diaphragm span, depends on the resisting moment furnished by nail couples at each stud crossing. If the nail couple, M = F_n.s, then the shear force per metre length, v , that can be resisted is

$$v = \frac{F_n}{l} . \frac{s}{b} \qquad \text{...11B(1)}$$

and the total shear strength is

$$V = \frac{2F_n\, s\, B}{b\, l} . \qquad \text{...11B(2)}$$

If the boards have not shrunk apart, then friction between the board edges could possibly increase the load carrying capacity by the addition of a term, 2Bv', where

v' = 74 N/m for 25 mm sawn boards,
= 148 N/m for 50 mm sawn boards, and
= 222 N/m for tongue and groove boards.

The in-plane stress in the sheathing is given by the expression

$$V = \frac{2F_b\, z\, B}{b\, l} \qquad \text{...11B(3)}$$

where;

z = section modulus of the sheathing board = $\dfrac{b^2\, t}{6}$.

11B.2 Single diagonal sheathing:

As above, the strength of the diaphragm depends on the resisting moment produced by the nail couples at each joint crossing. The total load that can be resisted is;

$$W = \frac{F_n\, N\, B}{b} \qquad \text{...11B(4)}$$

where;

N is the total number of nails.

The in-plane stress in the sheathing is given by the expression,

$$W = F_c B t. \qquad \text{...11B(5)}$$

The chord members need to be checked for combined bending and axial stresses (refer to NZS3603).

11B.3 Double diagonal sheathing:

The total load that can be resisted by the nail couples at each joist crossing is the same as for the single diagonal sheathing and the load resisted by the in-plane stress in the sheathing is;

$$W = 2F_cBt. \qquad \ldots 11B(6)$$

11B.4 Panel sheathing:

The strength values in Table 11.1 should be used in assessing the strength of these elements – unless specific tests are carried out.

Appendix 11C: Timber Shear Wall Stiffness

The horizontal inter storey deflection in one storey of a shearwall Δ_w can be calculated from:

$$\Delta_w = \Delta_4 + \Delta_5 + \Delta_6 + \Delta_7 \qquad \text{...11C(1)}$$

where

Δ_4 = deformation due to support connection relaxation
Δ_5 = wall shear deformation
Δ_6 = deformation due to nail slip
Δ_7 = deformation due to flexure as a cantilever (may be ignored for single storey shear walls).

For transverse sheathing:

$$\Delta_4 = (\delta_c + \delta_t)\frac{H}{B} \qquad \text{...11C(2)}$$

$$\Delta_5 = 0$$

$$\Delta_6 = 2\frac{H}{s}e_n \qquad \text{...11C(3)}$$

$$\Delta_7 = H\theta$$

For single diagonal sheathing:

$$\Delta_4 = (\delta_c + \delta_t)\frac{H}{B} \qquad \text{...11C(4)}$$

$$\Delta_5 = \frac{VH}{GBt} \qquad \text{...11C(5)}$$

$$\Delta_6 = 2\sqrt{2}\,e_n \quad \text{for the case where H} \le \text{B, OR} \qquad \text{...11C(6)}$$

$$= 2\sqrt{2}\,\frac{H}{B}e_n \quad \text{for the case where H} \ge \text{B}$$

$$\Delta_7 = \frac{2VH^3}{3EAB^2} + H\theta \qquad \text{...11C(7)}$$

For double diagonal sheathing:

$$\Delta_4 = (\delta_c + \delta_t)\frac{H}{B} \qquad \text{...11C(8)}$$

$$\Delta_5 = \frac{VH}{GBt} \qquad \text{...11C(9)}$$

$$\Delta_6 = \sqrt{2}\,e_n \quad \text{for the case where H} \le \text{B, OR} \qquad \text{...11C(10)}$$

$$= \sqrt{2}\,\frac{H}{B}e_n \quad \text{for the case where H} \ge \text{B} \qquad \text{...11C(11)}$$

$$\Delta_7 = \frac{2VH^3}{3EAB^2} + H\theta \qquad \text{...11C(12)}$$

For panel sheathing:

$$\Delta_4 = (\delta_c + \delta_t)\frac{H}{B} \qquad \text{...11C(13)}$$

$$\Delta_5 = \frac{VH}{GBt} \qquad \text{...11C(14)}$$

$$\Delta_6 = 2(1 + a)me_n \qquad \text{...11C(14)}$$

$$\Delta_7 = \frac{2VH^3}{3EAB^2} + H\theta \qquad \text{...11C(15)}$$

where;

a	=	Aspect Ratio of each sheathing panel:
	=	0 when relative movement along sheet edges is prevented,
	=	1 when square sheathing panels are used,
	=	2 when 2.4 x 1.2 m panels are orientated with the 2.4 m length parallel with the diaphragm chords (= 0.5 alternative orientation)
A	=	Sectional area of one chord (mm^2)
B	=	Distance between diaphragm or shear wall chord members (mm)
e_n	=	Nail slip resulting from the shear force *V* (mm)
E	=	Elastic modulus of the chord members (MPa)
G	=	Shear modulus of the sheathing (MPa)
H	=	Height of the storey under consideration (mm)
m	=	Number of sheathing panels along the length of the edge chord
t	=	Thickness of the sheathing (mm)
V	=	Shear force in storey under consideration (N)
θ	=	Flexural rotation at base of storey under consideration (radians)
δ_c	=	Vertical downward movement (mm) at the base of the compression end of the wall (this may be due to compression perpendicular to the grain deformation in the bottom plate)
δ_t	=	Vertical upward movement (mm) at the base of the tension end of the wall (this may be due to deformations in a nailed fastener and the members to which it is anchored).

Appendix 11D: Timber Shear Wall Strength

11D.1 Transverse sheathing:

The strength of transversely sheathed shear walls depends on the resisting moment furnished by nail couples at each stud crossing. If the nail couple, $M = F_n$. s, then the shear force per metre length, v , that can be resisted is;

$$v = \frac{F_n}{l} . \frac{s}{b} \qquad \text{...11D(1)}$$

and the total shear strength is;

$$V = \frac{F_n \, s \, B}{b \, l} . \qquad \text{...11D(2)}$$

If the boards have not shrunk apart, then friction between the board edges could possibly increase the load carrying capacity by the addition of a term Bv', where

v' = 74 N/m for 25 mm sawn boards,
= 148 N/m for 50 mm sawn boards, and
= 222 N/m for tongue and groove boards.

The in-plane stress in the sheathing is given by the expression;

$$V = \frac{F_b \, z \, B}{b \, l} \qquad \text{...11D(3)}$$

where;

z = section modulus of the sheathing board = $\frac{b^2 \, t}{6}$.

11D.2 Single diagonal sheathing:

The horizontal shear, V_i, carried by each board is;

$$V_i = \frac{1}{\sqrt{2}} N \, F_n \qquad \text{...11D(4)}$$

giving a total strength of;

$$V = \frac{F_n \, NB}{2b} \qquad \text{...11D(5)}$$

Since the axial force in the sheathing is the same on both sides of any intermediate stiffener, no load is transferred into the stiffeners from the sheathing. However, the perimeter members are subjected to both axial loads and bending and must be designed for the combined stresses (see NZS3603). The bending in the chord members is caused by a UDL of;

$$w = \frac{N \, F_n}{b} . \qquad \text{...11D(6)}$$

The in-plane strength of the sheathing is given by;

$$V = \frac{F_c\, bt}{2}. \qquad \text{…11D(7)}$$

11D.3 Double diagonal sheathing:

Based on the strengths of the nail couples, the strength of the shear wall is given by;

$$V = \frac{F_n\, NB}{2b} \qquad \text{…11D(8)}$$

The in-plane stress in the sheathing boards is given by the expression;

$$V = F_c\, Bt. \qquad \text{…11D(9)}$$

The stress in the chords is given by;

$$V = \frac{F_c\, BA}{H} \qquad \text{…11D(10)}$$

while the stress in the plates is given by;

$$V = F_c\, A_p. \qquad \text{…11D(11)}$$

11D.4 Panel sheathing:

The strength values in Table 11.1 should be used in assessing the strength of these elements – unless specific tests are carried out.