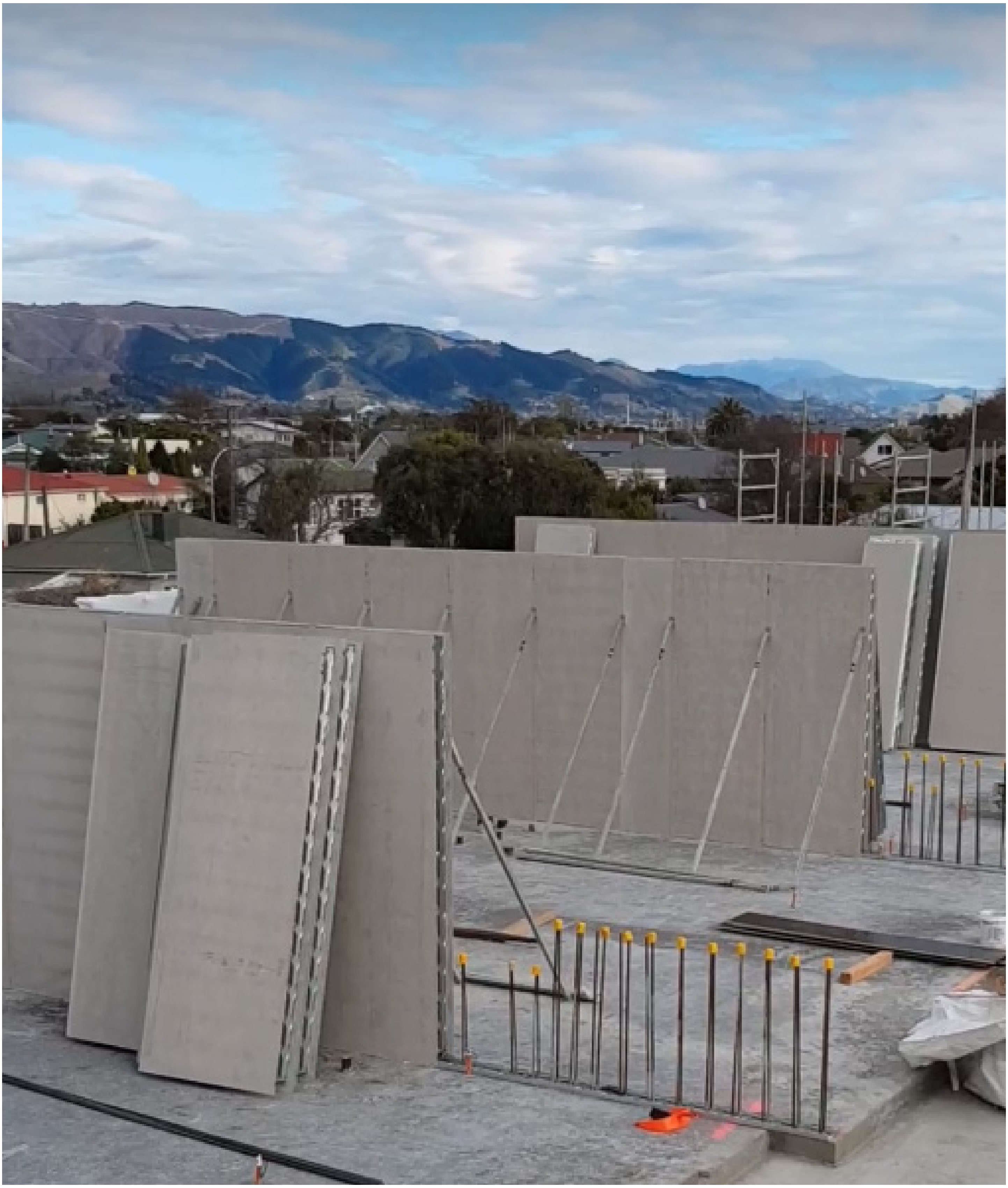
afs logicwall

Holmes



Holmes Worked Example

Limited Ductile AFS200D Wall

November 2022

AFS - Worked Example - Limited Ductile AFS200D Wall

The following work example demonstrates the use of AFS200D LOGICWALL as a substitute for a typical reinforced concrete wall designed to NZS3101:2006. Note that due to the underlying fundamental mechanics and tested performance of AFS200D, this system may be designed in accordance with NZS3101

For low axially-loaded walls, where N*<0.10Ag.fc', and where vertical reinforcing steel is less than 1% Ag, additional transverse reinforcement requirement for lateral restraint, confinement and anti-buckling (Clauses 11.4.5.4.2, 11.4.5.3, 11.4.5.4, 11.4.5.5) is not required. Refer to the test report In & Out-of-plane Shear and Bending Testing of the AFS LOGICWALL System available on the AFS website for more information.

For axial loads in excess of 0.10Ag.fc', AFS200D may still be used, however additional insitu boundary regions may be required at wall ends. Talk to your LOGICWALL supplier for more details on formwork to be used for this purpose.

Talk to your LOGICWALL supplier, or refer to the LOGICWALL Master Specification for the appropriate concrete mix to be used with LOGICWALL products.

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Scope:
This document provides an example shear wall design using AFS Logicwall permanent formwork system. The design is for a 5-storey apartment building in a New Zealand region with moderate seismicity on non-liquefiable soils. The design is for the shear wall only. It assumes the wall has a low axial load (N*<0.10Ag.fc'). The wall is designed be limited ductile (μ = 2).

It has been demonstrated by testing that low-axially loaded AFS200D walls (N*<0.10Ag.fc') without stirrups or transverse ties, can achieve ductile performance equivalent to a reinforcing concrete wall detailed in accordance with NZS3101:2006 compliant detailing.

The Building

Scope:
-5 storey apartment building
- Regular contruction - walls spaced at regular centres along inter-tenancy walls
- logicwall shear wall - 200 thk (AFS200D)
- Chch (medium seismicity)
- TT floors
- Lightweight roof

Floor part-plan:

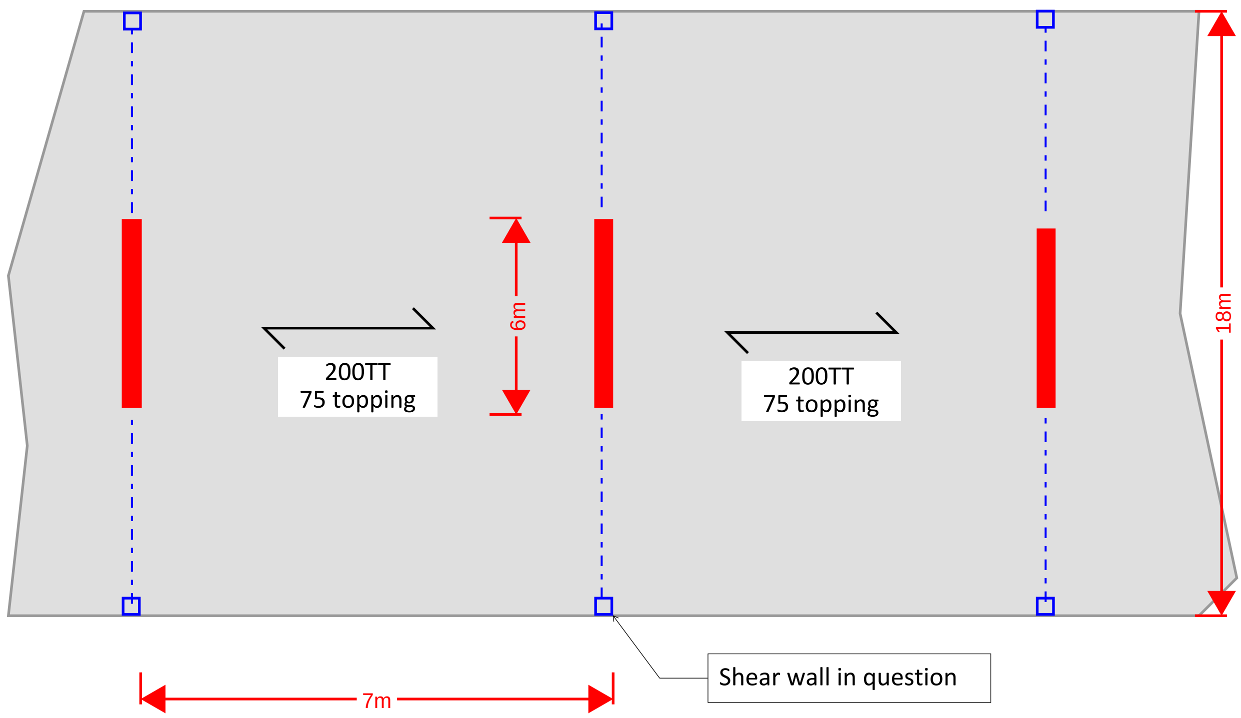

Figure 1 - Typical floor part plan

Notes:
- Concurrency and eccentricity ignored for the purposes of this design.

Loads

Gravity Loads

Calculate building weight and imposed loads in accordance with the principles of NZS1170.1.

Dead Load

Roof	=	1.0 kPa
200TT + 75	=	3.8 kPa
200 thk GF slab	=	5.0 kPa

Super Dead Load

Roof	=	0.5 kPa
Typ. floor	=	1.0 kPa
200 thk GF slab	=	1.0 kPa

Live Load

Roof	=	0.25 kPa
Typ. floor	=	2.0 kPa
200 thk GF slab	=	2.0 kPa

Calculate live load reduction factor

ψQA	=	0.3 + 3/sqrt(18m*7m * 4 floors)
	=	0.5

Gravity Load Combinations:

Roof

[G, Q]	2.50	kPa
[1.2G, 1.5Q]	3.30	kPa
[G, 0.3phiAQ]	1.80	kPa

Typ. floor

[G, phiQ]	6.20	kPa
[1.2G, 1.5Q]	8.76	kPa
[G, 0.3phiAQ]	5.10	kPa

Calculate Seismic Forces

Calculate seismic weight and seismic coefficient in accordance with NZS1170.5

-Chch
- IL2
- Soil Class C
- Estimate T = 0.4s
- Estimated ductility, μ = 2

Seismic Weight

- Trib area (per wall) = 18 m* 7m = 126 m2

Seismic Weights					
Level	**UDL (Swt)**	**Trib**	**Height**	**Wall Thickness**	**Total SWt (incl wall)**
Rf	1.8 **kPa**	126 m2	3.1 m	200 mm	273 **kN**
L04	5.1 **kPa**	126 m1	3.1 m	200 mm	736 **kN**
L03	5.1 **kPa**	126 m2	3.1 m	200 mm	736 **kN**
L02	5.1 **kPa**	126 m2	3.1 m	200 mm	736 **kN**
L01	6.6 **kPa**	126 m2	4 m	200 mm	938 **kN**

Table 1 - Seismic weight per wall

Calculate Seismic Coefficient

Calculate seismic coefficients for elastic and limited ductile (μ = 2) response.

Elastic Loading		**Limited Ductile Loading, μ=2.0**	
Zone Factor	0.3	Zone Factor	0.3
Spectral shape factor	2.36	Spectral shape factor	2.36
Near fault factor	1	Near fault factor	1
Elastic spectra coefficient, C	0.71	Elastic spectra coefficient, C	0.71
Return period factor, R	1	Return period factor, R	1
Structural perfomance factor, Sp	1	Structural perfomance factor, Sp	0.7
Ductility factor, kmu	1	Ductility factor, kmu	1.57
Design Coefficient, Cd	**0.71**	**Design Coefficient, Cd**	**0.32**

Distribute Forces:

Distribute seismic lateral force up the height of the building using the equivalent static method as outlined in NZS1170.5

- Use Equivalent Static Method to NZS 1170

$$F_i = F_t + 0.92V \frac{W_i h_i}{\sum_{i=1}^{n} (W_i h_i)} \quad \ldots 6.2(2)$$

Level	Name	Hi (m)	Wi (kN)	Fi (kN)	Vi (kN)	Mi (kNm)
5	Rf	3.100	269	228	228	706
4	L04	3.100	728	312	540	2381
3	L03	3.100	728	240	780	4798
2	L02	3.100	728	167	947	7733
1	L01	4.000	929	120	1067	11999

Table 2 - Equivalent Static Loading

Check assumed period:

- Use Rayleigh Method

Level	Wi (kN)	Fi (kN)	Di (m)
Rf	269	228	0.026
L04	728	312	0.02
L03	728	240	0.013
L02	728	167	0.007
L01	929	120	0.003
		Period, T (s) =	0.39

Table 3 - Fundamental Period Verification - Rayleigh's Method

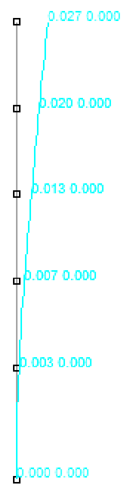

Figure 2 - Storey Drift

<u>T = 0.39s < 0.4s. Therefore for EQS method - OK!</u>

Check if P-delta to be considered:

NB: Largest translational period is < 0.4s, therefore P-delta need not be considered

Check moment capacity:

Calculate moment demand on the wall and determine reinforcing requirements

Assume:
- Hinging Level is above ground

Critical moment at Grd:

M* = 11,999 kNm

Say, XD16 at 292 crs e.f.
fc' = 30 MPa

ρ = 0.07% < 1% (OK)

Calculate axial load on wall

Beams frame into each end of wall, therefore floor load on wall = 6.0m + (18m - 6.0m)/2 = 12 m

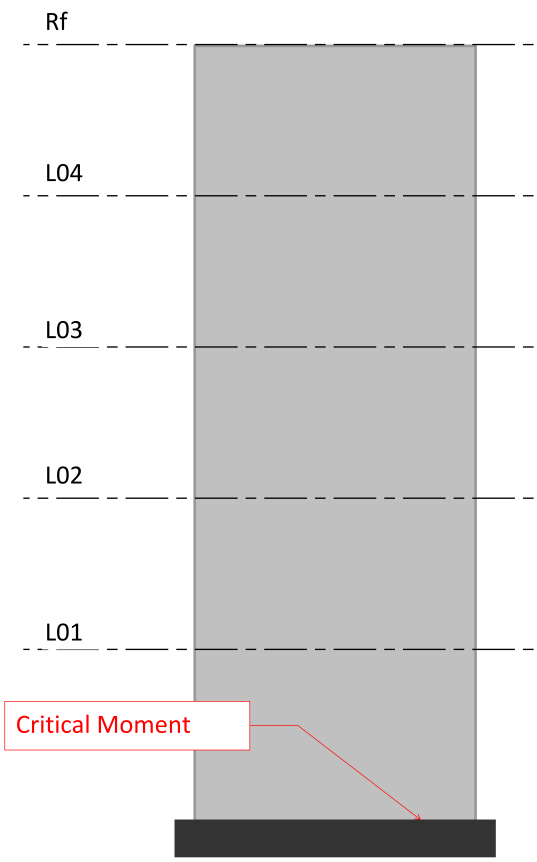

Figure 3 - Wall Elevation

Seismic Weights

Level	UDL (Swt)	Trib width	Trib length	Floor load	Axial per base of floor	Aggregate load
Rf	1.8 kPa	7 m	12 m	151 kN	244	244 kN
L04	5.1 kPa	7 m	12 m	428 kN	521	766 kN
L03	5.1 kPa	7 m	12 m	428 kN	521	1287 kN
L02	5.1 kPa	7 m	12 m	428 kN	521	1808 kN
L01	6.6 kPa	7 m	12 m	554 kN	674	2483 kN

Table 4 - Wall axial load

N* = 2483 kN (0.07Ag fc')

Therefore:

ϕMn = 13,331 kNm > M* = 11,999 kNm

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Calculate Overstrength Moments in Walls Above:

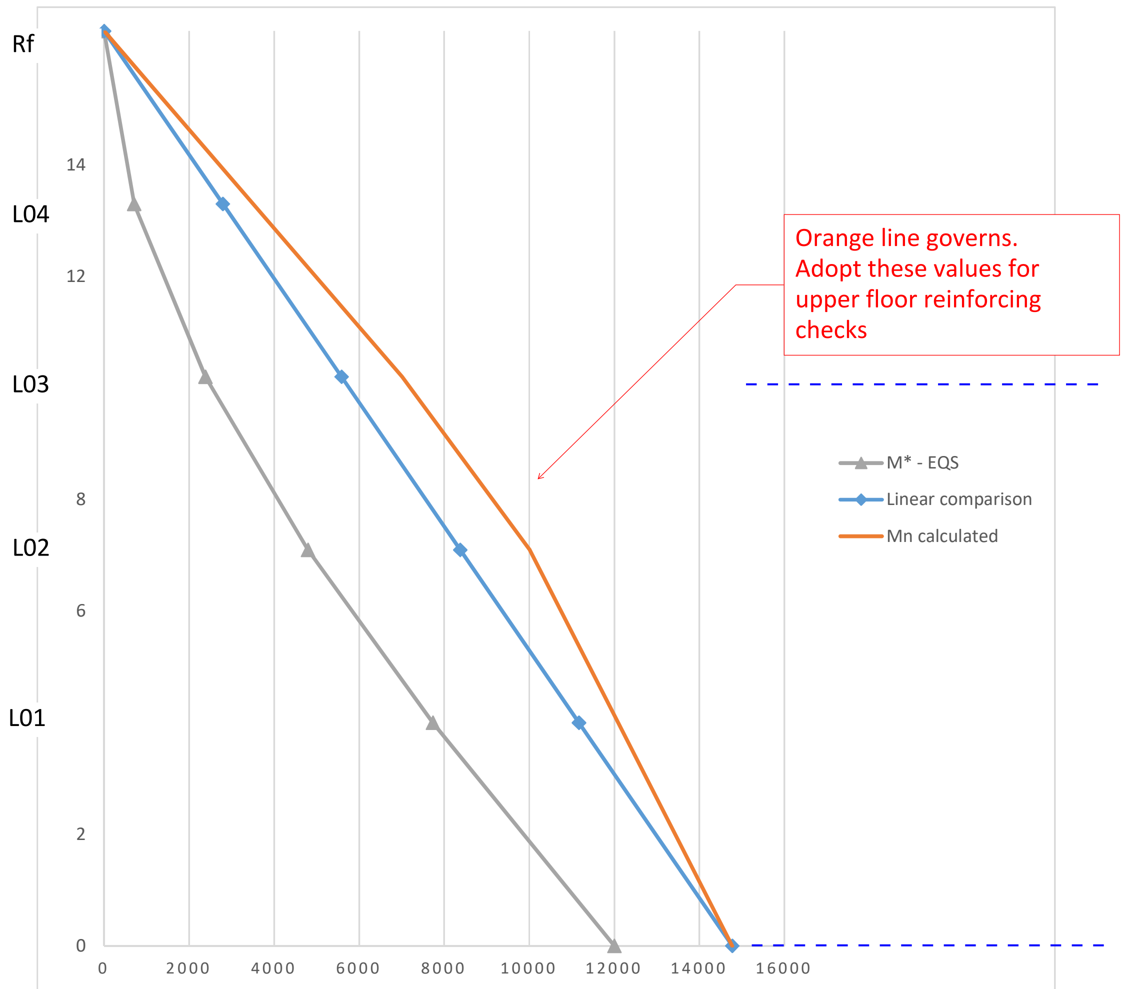
Compare two methods to NZS3101:2006 - CD4.2
1. Linear method
2. Calculated method

Method 2 is deemed critical

Governing case

Level	M* - EQS	phiMn	Linear comparison	Midheight M*	Equation CD-7	Mn calculated
	kNm	kNm	kNm	kNm	kNm	kNm
Rf	0		0			0
L04	706		2520			3505
L03	2381		5040	3940	9271	7010
L02	4798		7560			9816
L01	7733		10080			11351
Grd	11999	13331	13331			13331

Table 5 - Comparison of scaling methods



XD16 at 292 e.f.
ϕMn = 13,331 kNm

Figure 4 - Comparison of Scaling Methods

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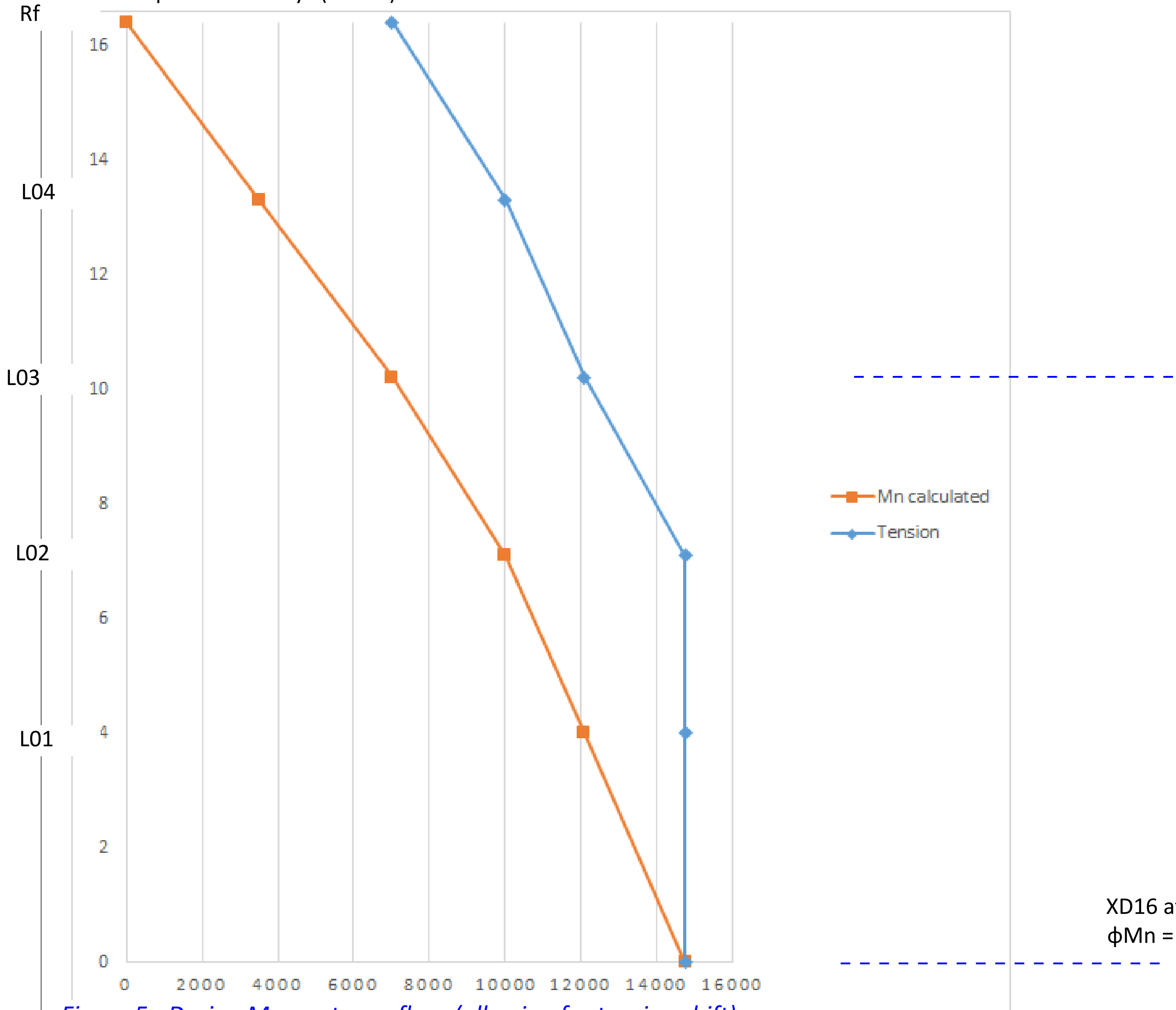
Calculate design moments allowing for tension shift:

Redistribute moments up the height of the wall to allow for tension shift.

- Shift moments up the scale building by Lw = 5.5m
- Round up to two storeys (=6.2m)



XD16 at 200 e.f.
ϕMn = 13,331 kNm

Figure 5 - Design Moments per floor (allowing for tension shift)

N*	Tension Shift M*
0 kN	7010 kNm
241 kN	9816 kNm
754 kN	11351 kNm
1266 kN	13331 kNm
1778 kN	13331 kNm
2441 kN	13331 kNm

Try reducing reinforcing at L04

Try XD12 at 292 e.f. above L04:

ϕMn = 7,368 kNm > M* = 9816 kNm

NG! Therefore, Adopt XD16 at 292 e.f full height of wall

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Check ductility capacity

Confirm ductility demand on the wall and the appropriate level of detailing in accordance with NZS3101:2006.

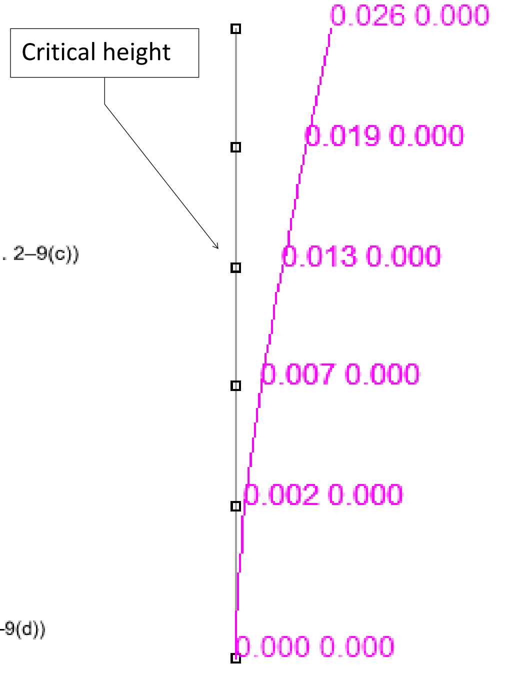
- Confirm assumed ductility, μ = 2.0 is OK
- Confirm Limited Ductile detailing required

Assume elastic displacement equivalent to displacement at M* = ϕMn

Scale elastic actions by ϕMn / M*(m=1.0) = 14107 kNm / 27235 kNm = 0.518

Therefore, elastic displacement = 13mm * 0.518 = 7mm

Inelastic displacement = 13-7 = 6mm



Building Deflection Profile

Calculate plastic hinge length

$$\ell_p = 0.15 \frac{M_e}{V_e} \le 0.5 L_w \quad \text{.......... (Eq. 2–9(c))}$$

lp = 0.15 * 11999 / 1067 = 1.55 m

Therefore,
Ɵp = 0.006 / 10.9m = 0.00055

ϕp = 0.00055 / 1550mm = 0.000000036

Calculate elastic curvature

$$\phi_y = \frac{2 f_y}{E_s h} \quad \text{.......... (Eq.2–9(d))}$$

ϕy = 2 * 425 / (200000*6000) = 0.000000071

Therefore,
Kd = (7.1e-7+3.6e-7)/(7.1e-7) = 1.5

Table 2.4 – K_d factor for limiting curvatures in flexural plastic regions in beams, columns and to walls where $h_w/L_w \geq 1.0$

Classification of plastic region	K_d for detailing type		
	Nominally ductile	**Limited ductile**	**Ductile**
Beams and columns [(1)]	Group (i) 3 Group (ii), see 2.6.1.3.4 (c)(ii) n/a	11	19
Walls, doubly reinforced with confined boundary elements	n/a	9	16
Walls, doubly reinforced	4	6	14
Walls, singly reinforced	0.8	n/a	n/a

NOTES –
(1) See 2.6.1.3.4 (c) (i) and (ii).
(2) A wall may be assumed to have confined boundary elements where the dimensional requirements of 11.4.3 and 11.4.5 and 10.4.7.5.1 for the compression force acting on the each boundary element.
(3) Group (i) and (ii) are described in C2.6.1.3.4

Kd_actual = 1.5 < Kd_limited_ductile = 9

Therefore M=2 and limited ductile detailing is appropriate for this wall.

Calculate Design Shear Force Envelope

$$V_o^* = \omega_v \phi_o V_E \quad \text{(Eq. CD–8)}$$

where ω_v is the dynamic shear magnification factor, which is given by:

$$\omega_v = 0.9 + \frac{n_t}{10} \quad \text{(Eq. CD–9)}$$

NB: For buildings above 6 storeys, ωv is calculated differently (refer NZS3101:2006).

φ_o = flexural overstrength at plastic hinge / bending moment at plastic hinge

ϕo = 20,414 kNm / 14,107 kNm = 1.45

ωv = 0.9 + 5/10 = 1.4

Therefore, factor V_E* by 1.45 * 1.4 = 2.03

NB: NZS3101 only requires that shear excedd nominally ductile actions, μ = 1.25, Sp = 0.925

(1.57 / 0.7) / (1.14 / 0.925) = 1.81 Therefore, walls need not be factored by more than 1.81

Design Wall Shear Reinforcing - Typical

Design shear reinforcing in accordance with NZS3101:2006 11.3.11

Calculate concrete shear contribution:

$$V_c = 0.17\sqrt{f_c'} A_{cv} \quad \text{(Eq. 11–12)}$$

(NB: Assumes wall concrete in net compression)

fc' = 30 MPa

Vc = 819 kN

Calculate required steel shear contribution:

$$V_s = \frac{V^*}{\phi} - V_c \quad \text{(Eq. 11–16)}$$

$$V_s = A_v f_{yt} \frac{d}{s_2} \quad \text{(Eq. 11–18)}$$

Therefore,

A_s = (V*/ϕ - Vc) * s_2 / (d x f_{yt})

Noting that As_min =

$$A_v = \frac{0.7 t_w s_2}{f_{yt}} \quad \text{(Eq. 11–19)}$$

Assuming horizontal bars e.f. at 200 crs

As_min = 56 mm2

Wall shear scaling checks

ϕo * ωv	2.058	
ϕm=1.25	1.81	

Governs

VE (kN)		V*0 kN		V*m=1.25, Sp=0.9 kN	
228	kN	469	kN	412	kN
540	kN	1112	kN	978	kN
780	kN	1605	kN	1412	kN
947	kN	1948	kN	1714	kN
1067	kN	2195	kN	1931	kN

Table 6 - Shear scaling per floor

Check reinforcing requirements

Name: AFS
Job no: xx
Length Wall 6000 mm — As_min 56 mm2
Thickness 200 mm
TABLE: Pier Forces

Story	P	V2	Vc	Vs req	As-req	Required bars area (assumes 1 bar each face)	
	kN	kN	kN	kN	mm2		
L04	241	412	894	0	0	0 mm2	XD12 e.f. OK
L03	754	978	894	410	1025	17 mm2	XD12 e.f. OK
L02	1266	1412	894	988	2470	41 mm2	XD12 e.f. OK
L01	1778	1714	894	1391	3477	58 mm2	XD12 e.f. OK
Grd	2441	1931	894	1680	4201	70 mm2	XD12 e.f. OK

Adopt XD12 at 200 crs OK

Table 7 - Shear reinforcing per floor

Design Wall Shear Reinforcing - Plastic Hinge **NZS3101:2006 - 11.4.6**

Calculate level of shear reinforcing required in the plastic hinge zone

Calculate concrete shear strength *11.4.6.1*

$$V_c = \left(0.27\lambda\sqrt{f_c'} + \frac{N^*}{4A_g}\right)t_w d \geq 0.0 \quad \text{(Eq. 11–28)}$$

λ = 0.5 for limited ductile plastic regions defined by Table 2.4

Vc = 0.27 * 0.5 * sqrt(30) + 2441kN/(4*200*6000) * 200 * 6000 =

0.74 MPa * 0.501 * 200 * 6000 = 449 kN

Story	P	V2	Vc	Vs req	As-req	Required bars area (assumes 1 bar each face)	
	kN	kN	kN	kN	mm2		
Grd	2441	1931	449	2126	5314	89 mm2	XD12 e.f. OK

Adopt XD12 at 200 crs in PHZ - OK

Check maximum shear force

$$\frac{V^*}{\phi} \leq \left(\frac{\phi_{ow}}{\alpha_q} + 0.15\right)\sqrt{f_c'}\,A_{cv} \leq v_{max}A_{cv} \quad \text{(Eq. 11–29)}$$

Max shear force in accordance with 11.4.6.3		
V*/ϕ	<=	Max design shear
2575 kN	<=	3330 kN

Additional Detailing Requirements for Members Designed for Ductility: **NZS3101:2006 - 11.4.2**

Determine Ductile Detailing Length *NZS3101:2006 - 11.4.2*

The ductile detailing length measured from the critical section for in-plane actions shall be taken as the larger of:

(a) $\frac{0.25M_e}{V_e} \leq 2L_w$; or

(b) 1.5 L_w.

where $\frac{M_e}{V_e}$ at the critical section is the moment to shear force ratio for seismic actions found from an

0.25 * Me/Ve = 2.8

Therefore, adopt 1.5 Lw = 9 m

Check dimensional limitations *NZS3101:2006 - 11.4.2*

$$t_m = \frac{\alpha_r k_m \beta (A_r + 2) L_w}{1700\sqrt{\xi}} \quad \text{(Eq. 11–20)}$$

tm = 130mm
tw = 200mm - OK

Check dimensional limitations *NZS3101:2006 - 11.4.4.2*

ρe_min = sqrt(30)/(2*500) = 0.55%
ρe_actual = = 0.70% OK

Transverse Reinforcing

For vertical reinforcing quantities less than 1%, no additional confinement steel is required when using AFS200D logicwall system. The formwork system provides sufficient confinement

THEREFORE, ADOPT AFS200D,
XD16 AT 392 CRS E.F VERTICAL
XD12 AT 200 CRS E.F. HORIZ.

Note that laps are to be detailed as non-contact laps in accordance with NZS3101:2006 using the equation:

$$L_{ds} \geq L_d + 1.5\, s_L.$$

Where S_L = stud spacing = 146mm.

eg

L_{ds} = (0.5 * 500 * 16)/sqrt(30MPa) + 1.5 * 146 mm = 950 mm

*** END OF CALCS ***