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* Solving quadratic equation for K

$$\therefore K^2 + 2 \cdot \left(3 - 4 \times \frac{1.2^2}{3.6^2}\right) K - 192 \times \frac{1.2^2}{3.6^2} = 0.$$

$$\therefore K^2 + 5.1 K - 21.3 = 0$$

$$ax^2 + \beta x + c = 0.$$

$$x_{1,2} = \frac{-\beta \pm \sqrt{\beta^2 - 4ac}}{2a}$$

$$\Delta = 5.1^2 + 4 \times 1 \times 21.3$$

$$\Delta = 111.2 > 0.$$

$$\therefore K_{1,2} = \frac{-5.1 \pm \sqrt{111.2}}{2 \times 1.0}$$

$\therefore K_{1,2} =$ +ve only $\Rightarrow \underline{K = 2.7}$

FROM (3):

$$M^* = \frac{7.43 \times 1.35^2}{2.7} = \underline{\underline{5 \text{ kNm}}} \text{ /m run}$$

* From previous calculation of face load demand

$F_{ph} = 7.43$ kN/m $\times 3.6 = 26.8$ kN.

$\therefore$ Dowel load $26.8/4 = 6.7$ kN. ✓ OK

Provided the type of fixings.

CALCULATION SHEET

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Flexure Capacity : M_n

$$\phi M_n = \phi (N + A_s f_y)(d - a/2)$$

Where: • $\phi = 1.0$.

• $N = 24 \times 0.15 \times 1.2 \times 1 = 4.32$ kN
thk. high long

• Provided 665 mesh $\Rightarrow A_s = 147\ mm^2$ per m width.

• $d = 150/2 = 75$ mm.

• $a = \dfrac{A_s f_y}{0.85 \times f_c b} = \dfrac{147 \times 300}{0.85 \times 30 \times 1000} = 1.73$.

$\therefore \phi M_n = (4.32 \times 10^3 + 147 \times 300)(75 - 1.73/2)$

$\therefore \phi M_n = 3.59 \simeq 3.6$ kNm/m long.

% NBS $= \dfrac{3.6}{5} = 72\%$. ✓ ok ($f_y = 300$ MPa)

$> 67\%$.

Alternatively:

$a = \dfrac{147 \times 485}{0.85 \times 30 \times 1000} = 2.79$ ($f_y = 485$ MPa)

$\therefore \phi M_n = (4.32 \times 10^3 + 147 \times 485)(75 - 2.79/2)$

$\therefore \phi M_n = 5.56$ kNm / m long

$\therefore$ % NBS $= \dfrac{5.56}{5} \simeq 100\%$. ✓ ok

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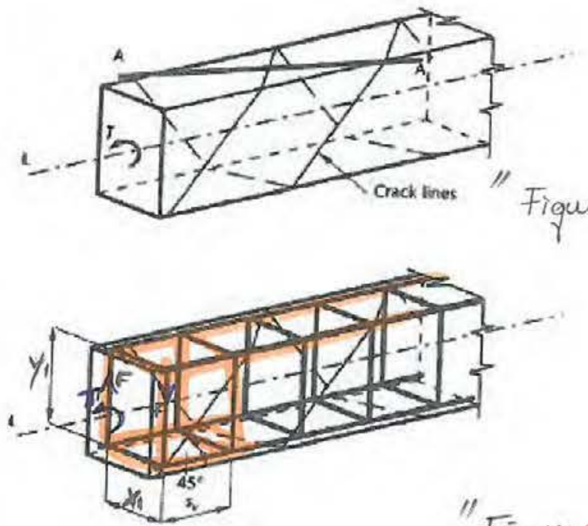
Beam subject to Torsional moments



"Figure 1"

"Figure 2"

Torsional moments produce shear stresses which result in principal tensile stresses inclined approximately 45° to the longitudinal axis of the member. Diagonal cracking occurs when these tensile stresses exceed the tensile strength of the concrete. The cracks will form a spiral around the member, as per Figure 1.

Reinforcement in the form of closed links and longitudinal bars will carry the forces from increasing torsional moment by a truss action with reinforcement as tension members and concrete as compressive struts between links. Failure will eventually occur by reinforcement yielding, coupled with crushing of the concrete along line AA as cracks open up on other faces.

CALCULATION SHEET

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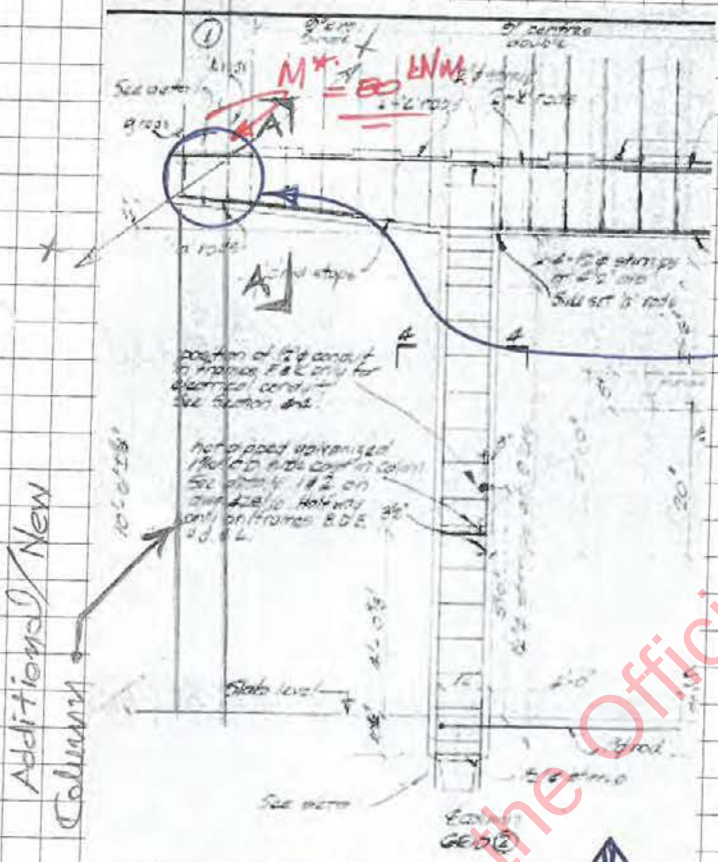
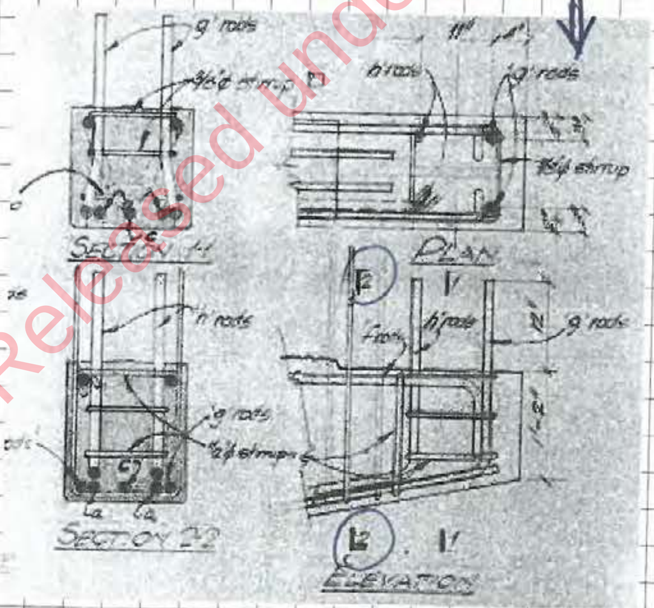
Torsional capcity of precast beam at 1st floor

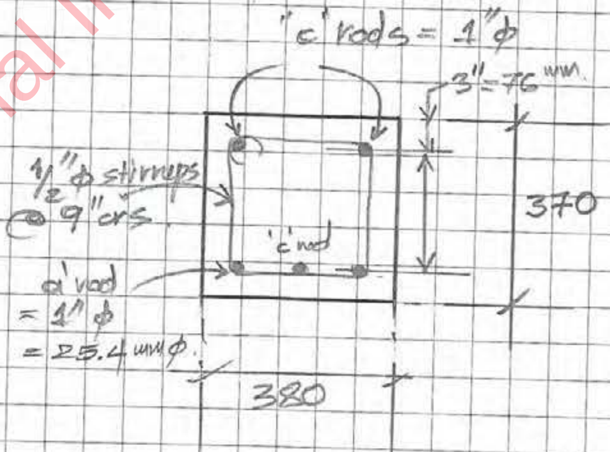
Cantilevering Column - longitudinal x-direction -



Check beam section for torsional resistance.

EXTRACTS OF DWG. No 28.





SECTION A-A.

SECTION Areas:

- $A_o = 220 \times 230 = 52900 \text{ mm}^2$
- $A_{co} = 370 \times 380 = 140600 \text{ mm}^2$

PERIMETERS:

- $P_o = 2 \times 220 + 2 \times 230$

$\therefore P_o = 900 \text{ mm}$

- $P_c = 2 \times 380 + 2 \times 370$

$\therefore P_c = 1500 \text{ mm}$

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DESIGN MOMENT FOR TORSION

Torsional Strength

should satisfy : $T^* \leq 0.1\phi A_{co} t_c \sqrt{f_c'}$ ‖ In order to satisfy equilibrium

Where : • $T^* = 80$ kNm – from Cantilever Column @ 1st LONGITUD.

• $\phi = 0.75$

• $f_c' = 30$ MPa

• A_{co} = Area enclosed by perimeter of section (mm^2)
($380 \times 370 = 140600$ mm^2)

• $t_c = 0.75 A_{co}/p_o$

$\therefore t_c = 0.75 \times \frac{140600}{900}$

$\therefore$ $t_c = 117.2$

• p_o = perimeter of area A_o (mm)

$\therefore$ $p_o = 900$ mm

• $A_{co} = 140600$ mm^2

$\Rightarrow 0.1\phi A_{co} t_c \sqrt{f_c'}$

$= 0.1 \times 0.75 \times 140600 \times 117 \sqrt{30}$

$= 6.75 \simeq 7$ kNm $< T^*$

Therefore, the torsional reinforcement is required.

$\therefore$ Existing reinforcement should be able to resist a nominal torsional moment, T_n, given by:

$$T_n = 0.44 A_{co} t_c \sqrt{f_c'} \left(1 + \frac{N^*}{0.33 A_g \sqrt{f_c'}}\right) \quad -(1)$$

OR

$$T_n = \frac{T^*}{\phi} \quad -(2)$$

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From (1): $T_n = 0.44 \times 140600 \times 117 \times \sqrt{30} = 39.6$ kNm

OR

From (2): $T_n = \frac{80}{0.75} = 106.7 \simeq 107$ kNm

HENCE:

∴ Existing Reinforcement should be able to resist a nominal torsional moment, T_n, equal to or greater than the larger from above.

i.e $T^* = 107$ kNm

Areas of Closed stirrups & longitudinal reinforcement

- Area of one leg of closed stirrup resisting torsion within a distance s, A_t, in mm^2.

$$A_{t\,req} = \frac{T_n s}{2 A_o f_{yt}}$$

$$\therefore A_{t\,req} = \frac{107 \times 10^6\,Nmm \times 230\,mm}{2 \times 52900\,mm^2 \times 300\,N/mm^2} = 775\,mm^2.$$

- Min. area of reinforcement, A_ℓ, around the perimeter p_o, shall be equal to: $A_\ell = \frac{T_n p_o}{2 A_o f_y}$

$$\therefore A_{\ell\,req} = \frac{107 \times 10^6\,Nmm \times 900\,mm}{2 \times 52900\,mm^2 \times 300\,N/mm^2} = 3034\,mm^2$$

66%.

$A_{prov.} = 4 \times \pi R^2 = 4 \times 507 = 2026\,mm^2$

↳ 4No. - 1" ϕ bars (25.4 mm ϕ)

CALCULATION SHEET

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Torsional Strength requirements

$$T^* \leq \phi T_n$$

Where:

- T^* : torsion at the section derived from the load on the structure
  $T^* = 80$ kNm
- T_n : nominal torsional strength of section.

$$T_n = 0.44 A_{co} t_c \sqrt{f_c'} \left(1 + \frac{N^* \rightarrow 0}{0.33 A_g \sqrt{f_c'}}\right)$$

$$\therefore T_n = 0.44 \times 140600 \times 117 \times \sqrt{30}$$

$$\therefore T_n = 39.6 \text{ kNm} \simeq 40 \text{ kNm}$$

HENCE:

$$\frac{T_n}{T^*} = \frac{40}{80} = 50\%. \text{ NBS.}$$

Torsional shear stress.

The torsional shear strength, V_{tn}:

$$V_{tn} = \frac{T_n}{2 A_o t_o}$$ shall not exceed $0.2 f_c'$ or 8 MPa.

Where:

- $T_n = 40$ kNm
- $A_o = 220 \times 230 = 52900 \text{ mm}^2$
- $t_o = 0.75 \times \frac{A_o}{P_c} = 0.75 \times \frac{52900}{1500} = 26.45$ mm

$$\Rightarrow V_{tn} = \frac{40 \times 10^6 \text{ Nmm}}{2 \times 5290 \times 26.45 \text{ mm}^3} = 142.9 \simeq 143 \frac{N}{mm^2}$$

$V_{max} = 0.2 \times 30 = 6$ MPa. or 8 MPa (whichever is the smaller)

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It is assumed that once the torsional shear stress on a section exceeds the value to cause cracking, tension reinforcement in the form of closed links must be provided to resist the full torsional moment (See Figure 2).

* Alternative to NZS3101.

Tension force in link $F = \frac{A_{sv}}{2} \times f_{yv}$

Moment of force F about centre line $= F \frac{x_1}{2}$ (for vertical leg)

$= F \frac{y_1}{2}$ (for horizontal leg)

A_{sv} = Cross-sectional area of the two legs of a link.

Total torsional moment provided by one closed link is given by the sum of the moments due to each leg of the link about the centre line of the section.

i.e. $T_{prov.} = F \frac{x_1}{2} \times 2 + F \frac{y_1}{2} \times 2$.

For links/stirrups at a distance s_v apart, the torsional resistance of the system of links is obtained by multiplying the moments due to each leg by the no. of legs crossing each crack. This number is given by:

- y_1/s_v - for vertical legs. and
- x_1/s_v - for horizontal legs (for cracks @ 45° approx)

CALCULATION SHEET

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Wellington East Girls College, Block 4- South Wing **Computed:**

Check:

The total torsional resistance provided:

$$T_{prov.} = \frac{A_{sv}}{2} \times f_y \times \frac{y_1}{s_v} \frac{x_1}{2} \times 2 + \frac{A_{sy}}{2} f_y \times \frac{x_1}{s_v} \times \frac{y_1}{2} \times 2.$$

$$\therefore \quad \underline{T_{prov.} = \frac{A_{sy}}{s_v} x_1 y_1 f_y}$$

Where:
- $A_{sy} = 2 \times \pi R^2 = 253\ mm^2$
  2 legs.
- $s_v = 230\ mm$.
- $x_1 = 228\ mm$ and $y_1 = 218\ mm$
- $f_y = 300\ MPa$.

$$\rightarrow T_{prov.} = \frac{253\ mm^2}{230\ mm} \times 228\ mm \times 218\ mm \times 300\ N/mm^2 \times 10^{-6}.$$

$\rightarrow \underline{\underline{T_{prov.} = 16.4\ kNm.}}$ $\therefore \quad \frac{16.4}{40} = 41\%$.

CALCULATION SHEET

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Torsional and flexural shear together

Where torsional and flexural shear stresses occur together at a section the following condition shall be satisfied:

$v_n + v_{tn} < v_{max}$.

Where:

- v_n = Nominal shear stress (MPa)
- v_{tn} = Torsional shear stress (MPa)

CALCULATION SHEET

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Project Description:

3.6 : Diaphragm Connection Capacity to the Concrete Walls

Office:

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SHEAR DEMAND – Y DIRECTION

- Shear Force Demand on subject wall No. W14 or No. W15 : $F_y^* = 403$ kN

Refer To TABLE 1 Y-Direction

SHEAR CAPACITY Provided by :

1/ Tie Bars

- 2 No. 3/4" rods = 19 mm ϕ ⟹ ($A_v = 2 \times 283.5 = 567$ mm²)
- $f_y = 300$ MPa

∴ Tension Capacity : $T_y = f_y A_y$

$T_y = 300 \frac{N}{mm^2} \times 567\ mm^2$

$T_y = 170$ kN

2/ On the capacity of the diaphragm conn contribute also the shear friction of reinforcing bar perpendicular to wall.

- shear friction : V_n (NZS 3101, §7.7)

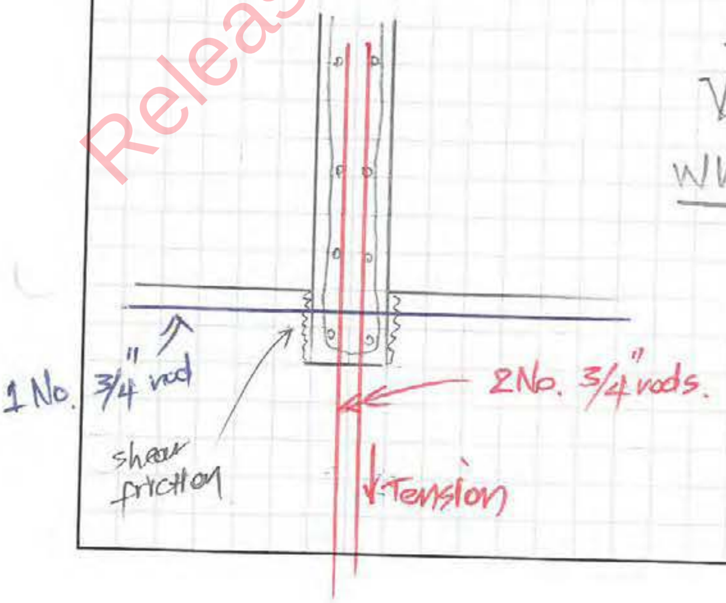
$V_n = (A_v f_y + N^*)^{=0} \mu$

Where :

- $A_v = 283.5\ mm^2$
- $f_y = 300$ MPa.
- $\mu = 1.0 \times \lambda. = 1.0.$

∴ $V_n = 283.5 \times 300 \times 10^{-3}$

∴ $V_n = 85$ kN



CALCULATION SHEET

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HENCE,

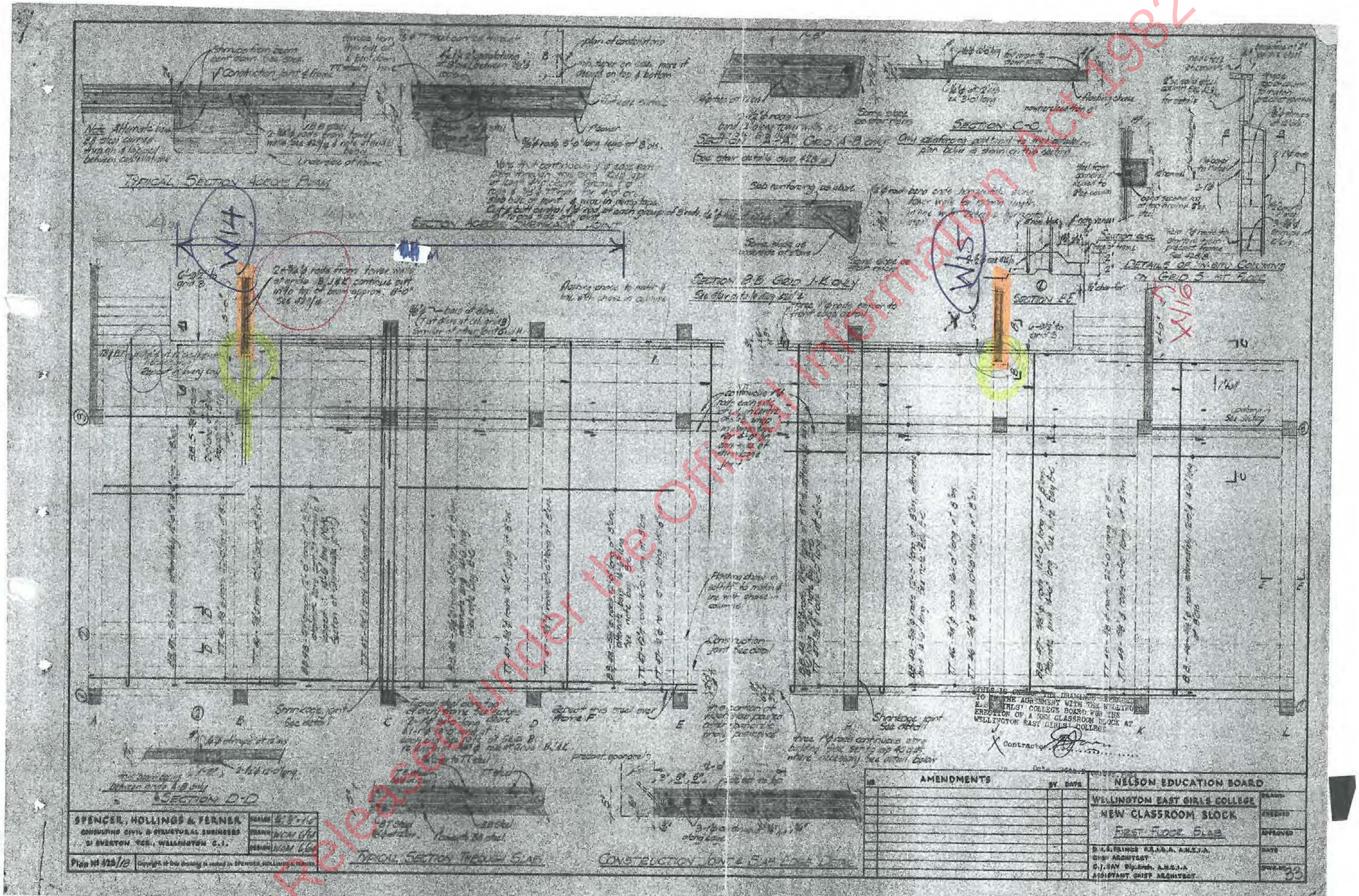
the diaphragm connection capacity at this wall location will be :

$$V_{c_{diaph.}} = 170 + 85 = \underline{\underline{255\ kN}}$$

∴

$$\% NBS = \frac{255}{403} = \underline{\underline{63\%}}.$$

OPUS


TYPICAL SECTION ACROSS BEAM
SECTION A-A GRID A-B ONLY
SECTION C-C
SECTION B-B GRID J-K ONLY
SECTION E-E
DETAILS OF IN-SITU COLUMNS ON GRID 5 AT FLOOR
W14
W15
SECTION D-D
TYPICAL SECTION THROUGH SLAB
CONSTRUCTION JOINT & SLAB EDGE DETAIL
WELLINGTON EAST GIRLS' COLLEGE
Contractor
SPENCER, HOLLINGS & FERNER
CONSULTING CIVIL & STRUCTURAL ENGINEERS
21 EVERTON TCE., WELLINGTON C.1.
Plan No 423/18
AMENDMENTS
BY
DATE
NELSON EDUCATION BOARD
WELLINGTON EAST GIRLS COLLEGE
NEW CLASSROOM BLOCK
FIRST FLOOR SLAB
CHIEF ARCHITECT
ASSISTANT CHIEF ARCHITECT
33

CALCULATION SHEET

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3.7. ROOF DIAPHRAGM | Computed: / /

Check: / /

- On each direction there are 3 No. steel flat bars ($3'' \times 1/4''$ → 76.2×6.35)

Tension Capacity : $T_y = 3\text{No.} \times (76.2 \times 6.35)^{mm^2} \times 260 \, N/mm^2$

$\therefore \; T_y = 377.4 \text{ kN}$

- Refer to page 62 for capacity of steel bracing per typical bay.

- The concrete beam at roof level has been checked below for biaxial bending.

Therefore, overal capacity > 67% for roof diaphragm.

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CALCULATION SHEET

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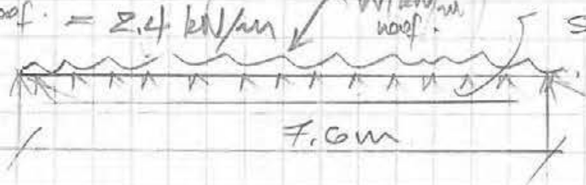
RC BEAMS @ ROOF LEVEL Computed: / /

CHECK FOR LATERAL LOADING: Check: / /

$F = Cd(T) \cdot W'$

$W_{roof} = 0.6 \, kN/m^2 \times 4 \, m =$

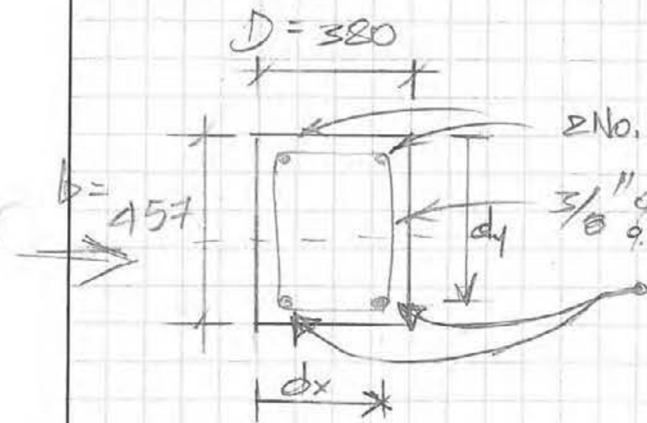
$= 2.4 \, kN/m$



$SW = 24 \times 0.457 \times 0.38 = 4.2 \, kN/m$

$\therefore F_h = 0.8 \times 4.2 = 3.3 \, kN/m$

Horizontal



2No. - 1/2"ϕ (12.7 mm ϕ)

3/8"ϕ stirrups @ 12" crs
9.5 ϕ @ 305 crs

2No. - 1"ϕ (25 mm ϕ)

- $As_{Y axis} = 127 + 490 = 617 \, mm^2$
- $As_{X axis} = 2 \times 490 = 980 \, mm^2$

- $d_x = 380 - 25 - 12.7 - 9.5 = 332.8 \, mm$
- $d_y = 457 - 25 - 12.5 - 9.5 = 410 \, mm$

FLEXURE CAPACITY ($\phi = 1.0$)

Y AXIS: $\phi M_{n_y} = \phi As f_y (d - a/2)$

- $a = \dfrac{As f_y}{0.85 f_c' b}$

$\phi M_{n_y} = 617 \times 300 \left(332.8 - \dfrac{15.8}{2}\right)$

$a = \dfrac{617 \times 300}{0.85 \times 30 \times 457} = 15.8 \, mm$

$\therefore \phi M_m = 60 \, kNm$

X AXIS: $\phi M_{n_x} = 980 \times 300 \left(410 - \dfrac{30.3}{2}\right)$

$a = \dfrac{980 \times 300}{0.85 \times 30 \times 380}$

$\therefore \phi M_{n_x} = 116 \, kNm$

$\therefore a = 30.3 \, mm$

FLEXURE DEMAND

$M_y^* = \dfrac{W_y L^2}{8} = \dfrac{3.3 \times 7.7^2}{8} = 24.4 \, kNm$ — due to lateral

$M_x^* = \dfrac{W_x L^2}{8} = \dfrac{2.4 \times 7.7^2}{8} = 17.8 \, kNm$ — due to gravity load.

Biaxial Bending Capacity =

$\left(\dfrac{M_x^*}{\phi M_{ox}}\right)^{1.4} + \left(\dfrac{M_y^*}{\phi M_{oy}}\right)^{1.40} \leq 1.0 \Rightarrow \dfrac{17.8}{116} + \dfrac{24.4}{60} = 0.35 < 1.0$

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CALCULATION SHEET

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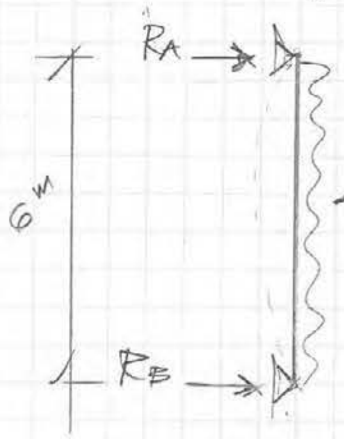
Per typical bay : Load applied : $\frac{855}{5} = 171$ kN

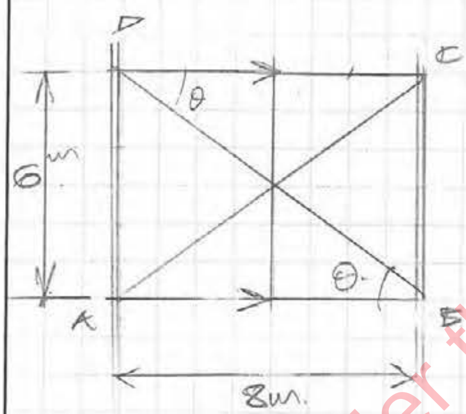


$\therefore$ $UDL = 171/6 = 28.5$ kN/m

$W = 28.5$ kN/m

$\therefore$ $R_A = R_B = \frac{WL}{2} = \frac{28.5 \times 6}{2} = \underline{85.5 \text{ kN}}$

- Axial Force acting on flat bar bracing is:



- $\cos\theta = 6/8 \Rightarrow \theta = 41.4°$
- $\cos 41.4 = \frac{F_{AB}}{F_{BD}} = \frac{85.5}{F_{BD}} \Rightarrow$

$\Rightarrow F_{AC} = F_{BD} = \frac{85.5}{\cos 41.4} = \underline{114}$ kN

- $F_{AC} = ? = F_{BD}$
- $F_{AB} = F_{CD} = 85.5$ kN

- Tension Capacity of a flat bar:

$T_y = A_g \times f_y = (76.2 \times 6.35)^{m^2} \times 260^{N/mm^2} = \underline{125 \text{ kN}}$.

$\therefore$ $\frac{125}{114} = 100\%$ NBS.

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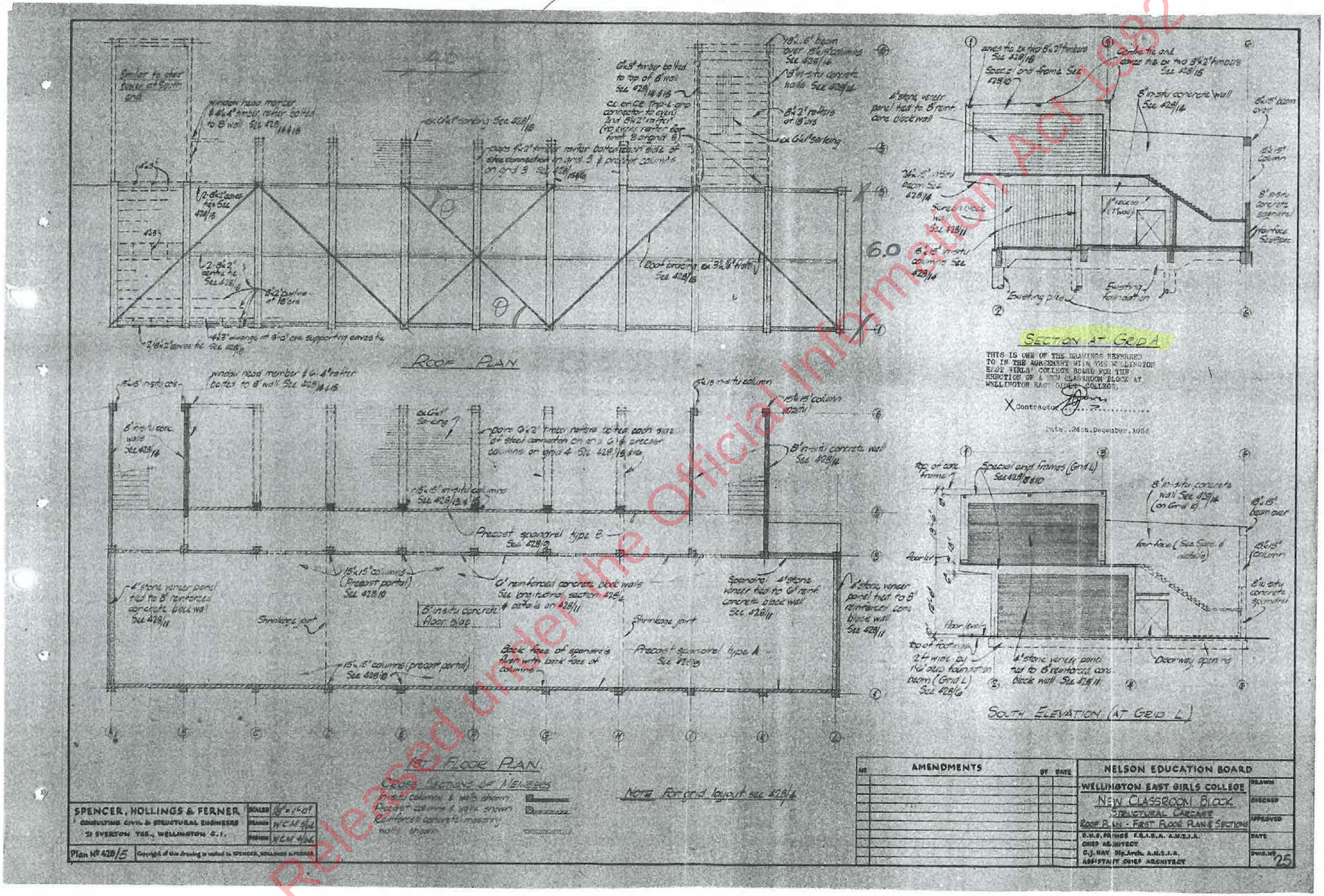
Distribution of Horizontal Forces

Floor Level	Storey height (m)	Weight Wi (kN)	h_i (m)	W_i*h_i (kNm)	Ratio $W_i*h_i / \Sigma(W_i*h_i)$	NZS 2006 Shear per floor Vx = Vy (kN)	NZS 2006 Shear per floor Vx = Vy (kN)
Roof	-	583	7.4	4314	0.25	1152	855
1	3.9	3690	3.5	12915	0.75	2560	2560
Gr	3.5	367	0.0	0	0.00	0	0
	Σ =	**4640**		**17229**		**3712**	**3415.0**
						(0.08V)	

Based on	NZS 2006
Sp =	0.925
μ =	1.25
Cd(T) =	0.8
Base shear-Vb	**3712.0**
0.08V	297.0
0.92V	3415.0

$\cos\theta = \frac{6^3}{8^4} = 0.75 \therefore \theta = 41.4^\circ$

-64.



Foundations - check rocking mode response

For both the longitudinal & transverse direction, a displacement method approach was used to estimate the foundation capacity at the portion of the seismic load at which uplift occurs for limiting deflection of 1% drift.

Summary of outcomes:

- Longitudinal direction

Based on horizontal displacement of 40mm, an uplift of 35mm takes place ⇒ Providing a capacity of 98%. (Rocking at 12% ratio M_R/M_o.)

- Transverse direction

Based on hor. displacement of 33mm, an uplift of 52mm occurs ⇒ Providing a capacity of 120%. (Rocking at 60% ratio of M_R/M_o)

CALCULATION SHEET

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Project Description: ALONG GRID LINE C – LONGITUDINAL DIRECTION – CHECK OVERTURNING OF PANEL Office:

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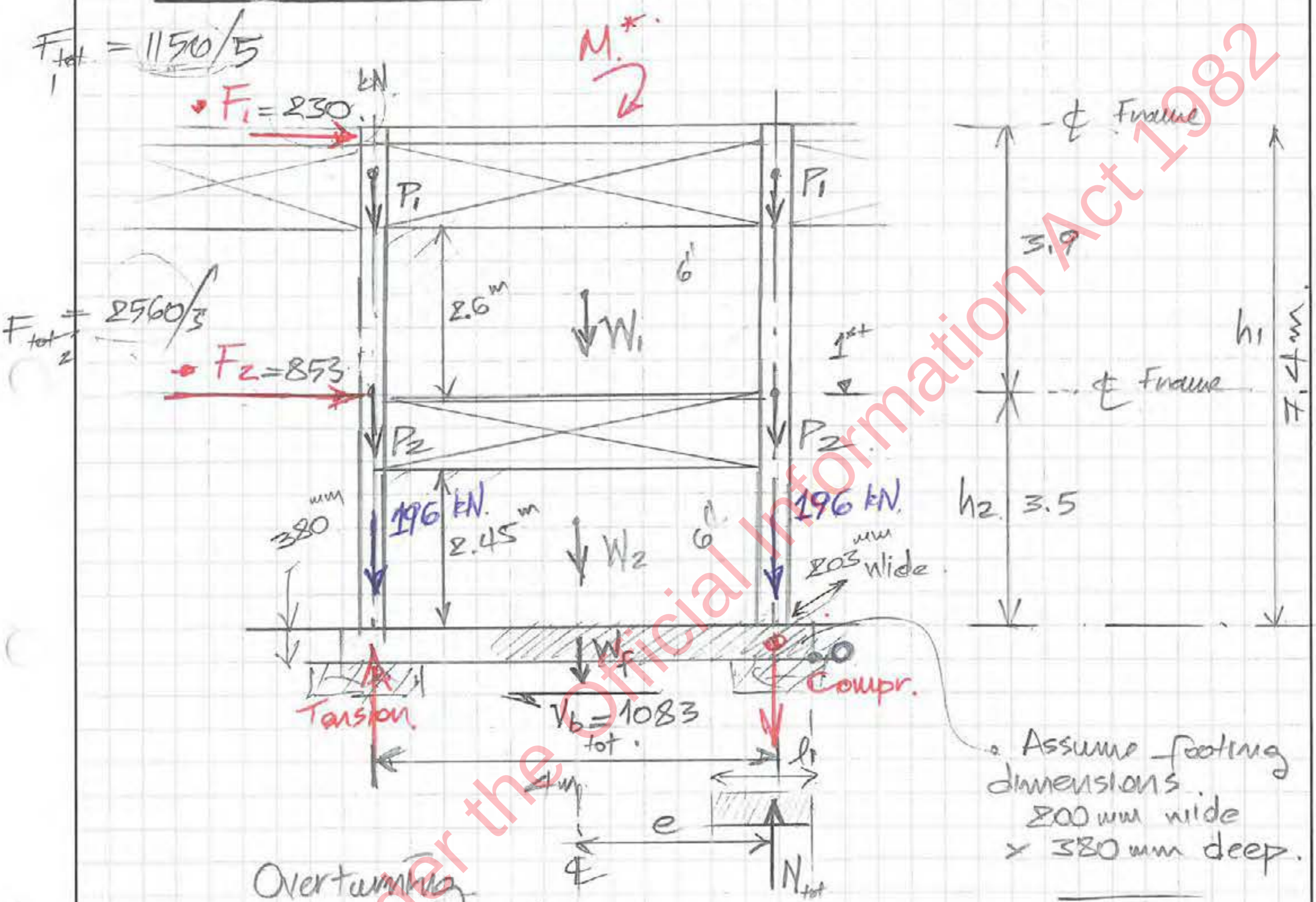
– TYPICAL CASE – · LONGITUDINAL DIRECTION



Assume footing dimensions 800 mm wide × 380 mm deep.

Overturning

Applied Moment (M_o^*):

$M_o^* = F_1 \times h_1 + F_2 \times h_2 = 230 \times 7.4 + 853 \times 3.5$

$\therefore M_o^* = 4687.5$ kNm

Resisting Moment (M_R):

- $l_1 = \dfrac{N}{q \times b} = \dfrac{400}{400 \times 0.914} = 1.1$ m

- $e = \dfrac{L}{2} - \dfrac{l_1}{2} = \dfrac{4}{2} - \dfrac{1.1}{2} = 1.45$ m,

$\therefore M_R = N \times e = 400 \times 1.45 = 580$ kNm

$\dfrac{M_R}{M_o^*} = 12\%$

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From previous, base shear on typical critical bay = $V_b' = 1083$ kN.
$\therefore V_i = 0.12 \times V_b' \cong 130$ kN.

Project: WEGC - Wellington East Girls College
Element: Foundations - Longitudinal direction
Calculation: Push-over curve, modified for mode shape, plotted on the NZS1170.5 Acceleration-Displacement demand spectra

Project Number: 5-PA010.37 **Sheet No:** of
Input By: MX
Date: Nov. 2015

Design Spectrum

Site Subsoil Class		B
Zone Factor	Z	0.4
Return Period Factor	R	1.3
Distance to Major Fault[1]	D (km)	1
Structural Performance Factor	Sp	0.7

(1) Equal to 21 if N/A or exceedance probability <1/250

Pushover Curve

Building weight	W (kN)	400.0					
Mode 1 participation factor	PF_1	1.00					
Mode 1 participating mass ratio (or mass coefficient)	α_1	1.00					
Pushover Curve Point	i	1	2 (at Δ_y)	3	4	5 (at Δ_{ULS})	6 (at Δ_{CLS})
Base shear	V_i (kN)	0.0	130.0	130.0	130.0	130.0	130.0
Roof level displacement	$\Delta_{i,roof}$ (mm)	0	20.0	20.0	20.0	40.0	150.0
Spectra acceleration	S_{ai} (g)	0.00	0.325	0.325	0.325	0.325	0.325
Spectral displacement	S_{di} (mm)	0.00	20.00	20.00	20.00	40.00	150.00
Period	T_i (s)		0.50	0.50	0.50	0.70	1.36

Structural Behaviour

Yield base shear	V_y (kN)	130.0
Yield displacement (roof)	Δ_y (mm)	20.0
System displacement ductility (@ ULS)	μ_Δ	2.00
Effective response period (@ ULS)	T_{eff} (s)	0.70

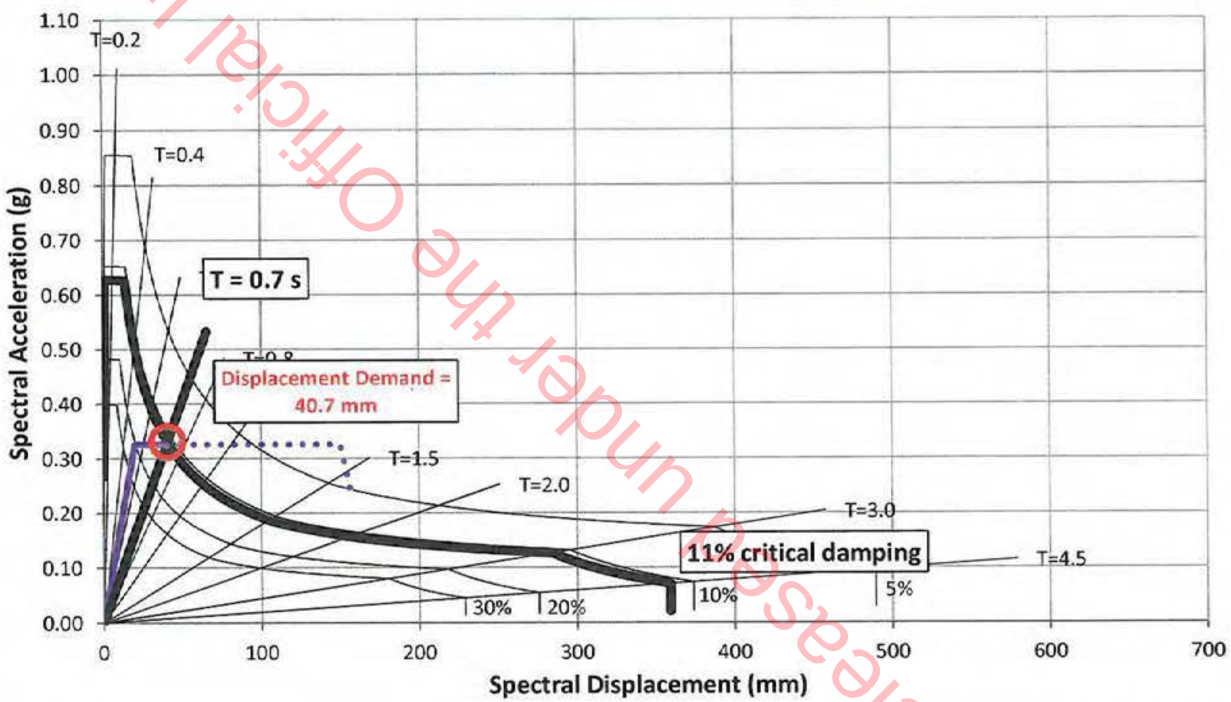
Effective Damping

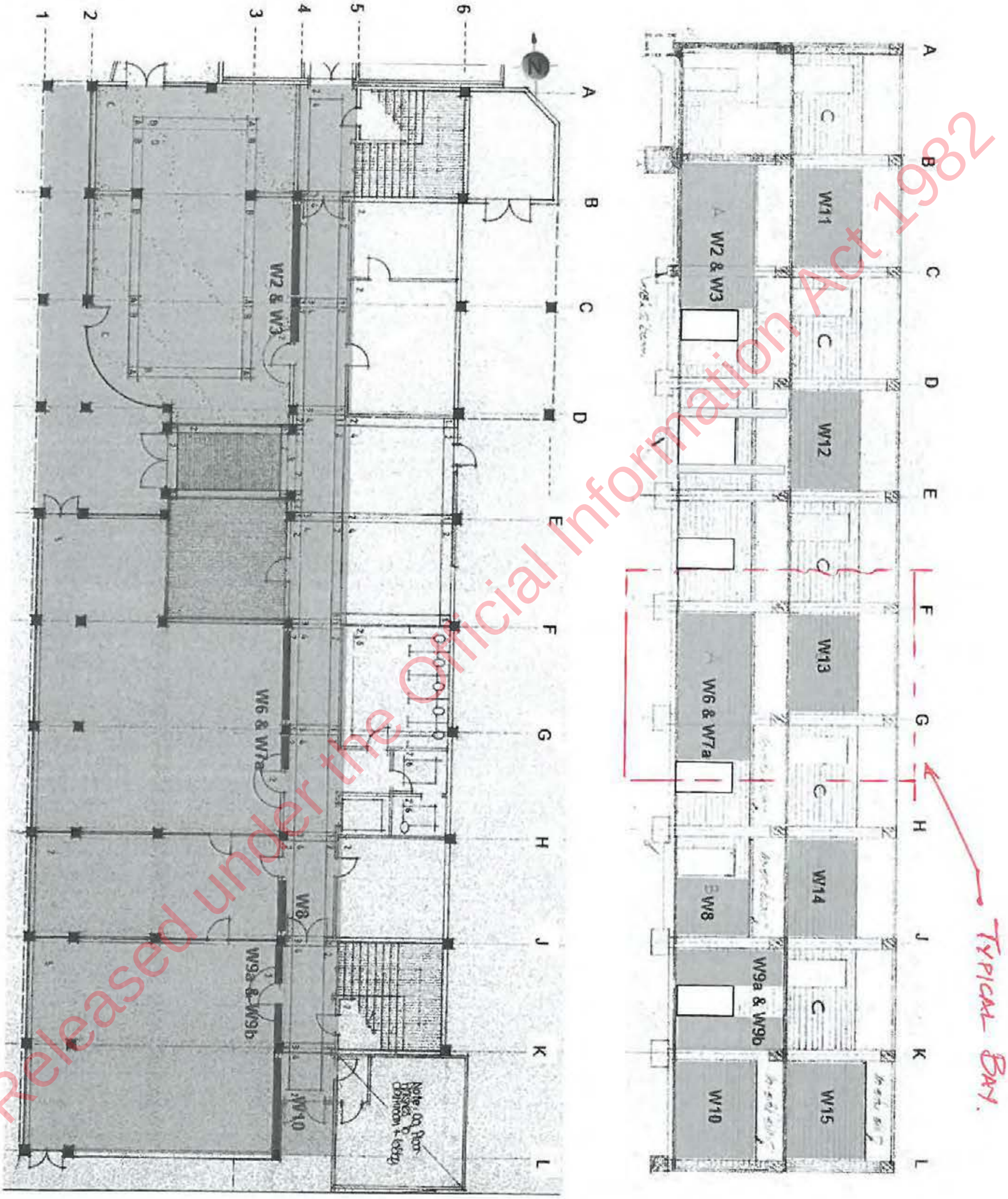
Method: Custom

Effective system damping (@ ULS)	ξ_{eff}	11.0%

Results

Spectral displacement capacity	S_{d_ult} (mm)	40.0
Spectral displacement demand	SD (mm)	40.7
Calculated capacity ratio	NBS	98%
with 11% critical damping	at 0.7 s, ductility 2	





1
2
3
4
5
6
A
B
C
D
E
F
G
H
J
K
L
W2 & W3
W6 & W7a
W8
W9a & W9b
W10
W11
W12
W13
W14
W15
C
TYPICAL BAY.

CALCULATION SHEET

Project/Task/File No:

Project Description:

THEORETICAL UPLIFT OF FOOTING
ON TYPICAL BAY – LONGITUDINAL DIR.

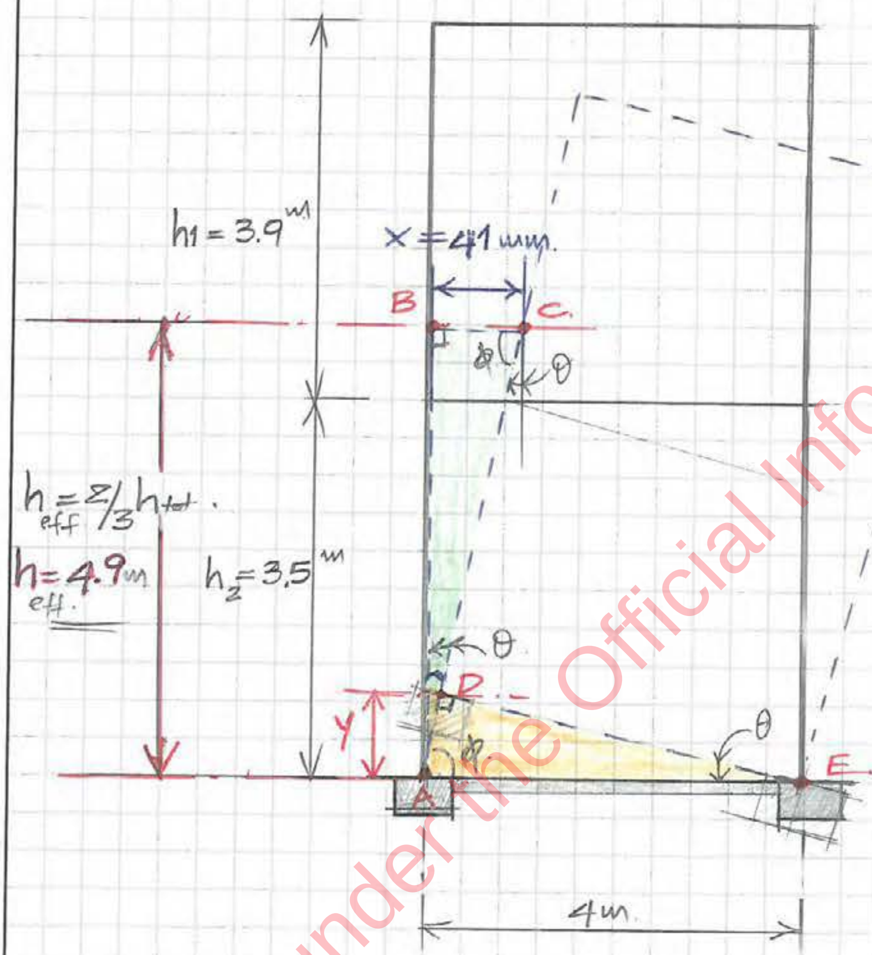
Sheet No 69 of

Office:

Computed: / /

Check: / /

• DRIFT → $\frac{41}{4900} \times 100 = 0.84\% < 1\%$.



$\phi + \theta + 90^\circ = 180^\circ$.

∴ $\hat{\phi} + \hat{\theta} = 90^\circ$

• From similar triangles: ΔABC and ΔADE

$\tan\theta = \frac{41}{4900} = \frac{Y}{4000}$

$\Rightarrow Y = 33.5 \approx 34$ mm – Vertical Uplift

NOTE:

Based on previous spreadsheet, the displacement demand is taken approx. 41mm.

CALCULATION SHEET

Project/Task/File No: | Sheet No 70 of

Project Description: | Office:

WEIGHT PER BAY BETWEEN FRAMES | Computed: / /

Check: / /

Gravity / Vertical Load:

- $P_1 = \underbrace{0.4\ kN/m^2 \times \underset{width}{4^m} \times (7.3/2 + \underset{length}{2})}_{Roof} + \underbrace{321/10.No}_{SW\ RC\ Beam} = \underline{41\ kN}$

- $P_2 = \underbrace{3.12\ kN/m^2 \times 4^m \times (7.3/2 + 2)}_{1^{st}\ fl.\ RC\ slab} + \underbrace{522/10.No}_{SW\ RC\ Beam} = 122.7$

- $W_1 = 3.3\ kN/m^2 \times 2.6 \times 3.835 = 32.9$

- $W_2 = 3.3\ kN/m^2 \times 2.45 \times 3.835 = 31.0$

∴ On each Column Side:

Tot. Load $N_{tot.} = [P_1 + P_2 + W_1/2 + W_2/2] \times 2$

$\therefore N_{tot.} = [41 + 122.7 + 32.9/2 + 31/2] \times 2$

$\therefore N_{tot.} = \underline{195.65\ kN} \times 2 = \underline{391.3\ kN}$

- Self-Weight of footing: (W_f)

$W_f = 24\ kN/m^3 \times 0.2 \times 0.38 \times 4 = \underline{7.3\ kN}$

Hence: $N'_{tot} = N_{tot} + W_f = 391.3 + 7.3 = 398.6$

$\simeq \underline{400\ kN}$

OPUS

CALCULATION SHEET

REVISED.

Project/Task/File No:

Sheet No 71 of

Project Description:

Office:

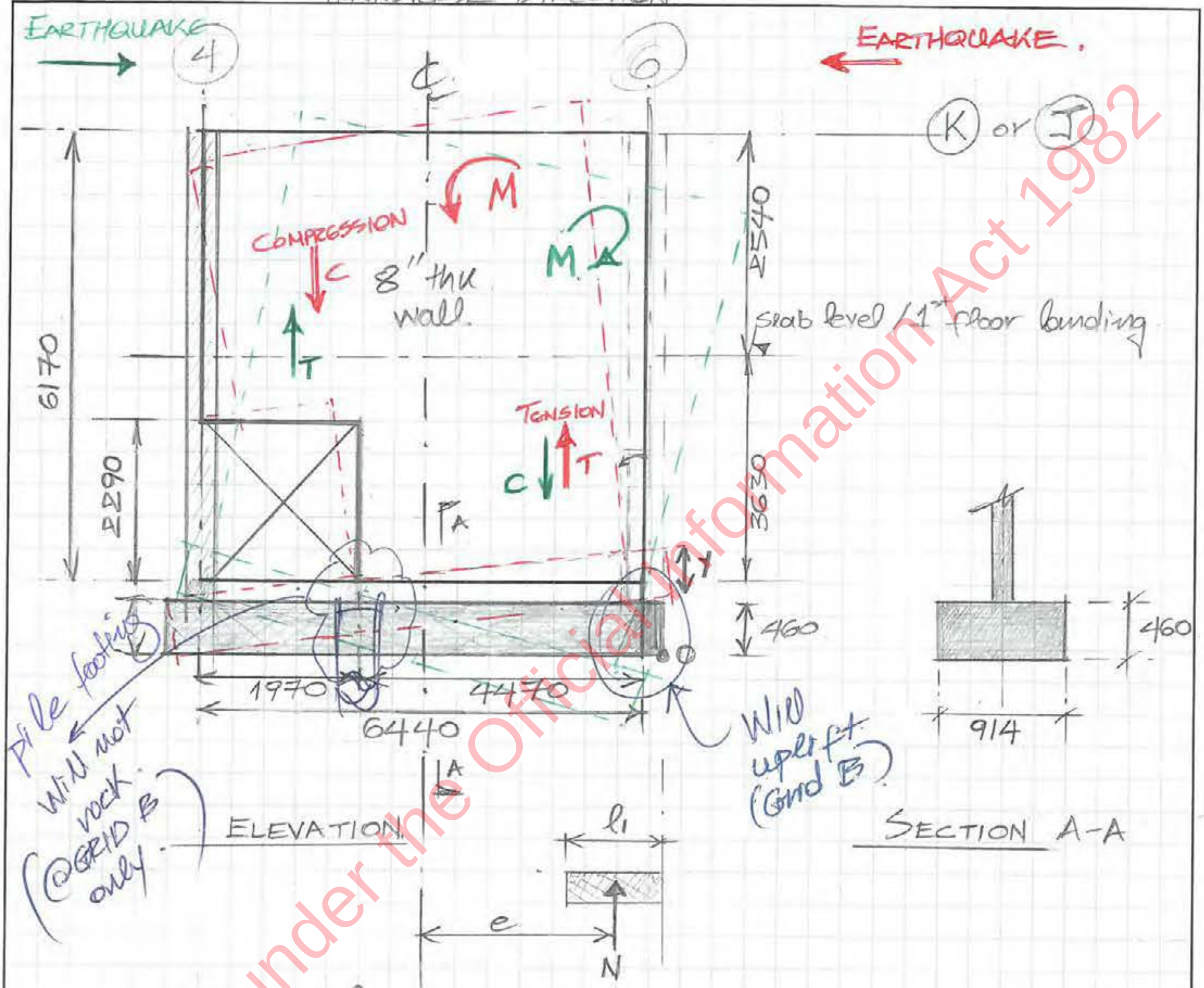
STAIR TOWER FOOTING.

Computed: / /

- CHECK ROCKING PER WALL SECTION -

TRANSVERSE DIRECTION

Check: / /



Bearing Length : (l_1)

$$l_1 = \frac{N}{q \times b} = \frac{290}{400 \times 0.914} = 0.79 \text{ m.}$$

Eccentricity : (e)

$$e = \frac{L}{2} - \frac{l_1}{2} = \frac{6.44}{2} - \frac{0.79}{2} = 2.825 \text{ m.}$$

Rocking Moment : (M_{rock})

$$M_{rock} = N \times e = 290 \times 2.825 = 819.25 \text{ kNm.}$$

Applied Moment : (M^*)

$$M^* = V^* \cdot h_{tot} = 200 \times (6.17 + 0.1 + 0.46) = 1346 \text{ kNm}$$

$\frac{M_R}{M_O} =$ 60%

-72

$V_i = 60\% \; V_b' = 0.6 \times 220 = 132$ kN

Project: WEGC - Wellington East Girls College | **Project Number:** 5-PA010.37 | **Sheet No:** of

Element: Foundations - Longitudinal direction | **Input By:** MX

Calculation: Push-over curve, modified for mode shape, plotted on the NZS1170.5 Acceleration-Displacement demand spectra | **Date:** Nov. 2015

Design Spectrum

Site Subsoil Class		B
Zone Factor	Z	0.4
Return Period Factor	R	1.3
Distance to Major Fault[1]	D (km)	1
Structural Performance Factor	Sp	0.7

(1) Equal to 21 if N/A or exceedance probability <1/250

Pushover Curve

Building weight	W (kN)	290.0					
Mode 1 participation factor	PF_1	1.00					
Mode 1 participating mass ratio (or mass coefficient)	α_1	1.00					
Pushover Curve Point	i	1	2 (at Δ_y)	3	4	5 (at Δ_{ULS})	6 (at Δ_{CLS})
Base shear	V_i (kN)	0.0	132.0	132.0	132.0	132.0	132.0
Roof level displacement	$\Delta_{i,roof}$ (mm)	0	20.0	20.0	20.0	40.0	100.0
Spectra acceleration	S_{ai} (g)	0.00	0.455	0.455	0.455	0.455	0.455
Spectral displacement	S_{di} (mm)	0.00	20.00	20.00	20.00	40.00	100.00
Period	T_i (s)		0.42	0.42	0.42	0.59	0.94

Structural Behaviour

Yield base shear	V_y (kN)	132.0
Yield displacement (roof)	Δ_y (mm)	20.0
System displacement ductility (@ ULS)	μ_Δ	2.00
Effective response period (@ ULS)	T_{eff} (s)	0.59

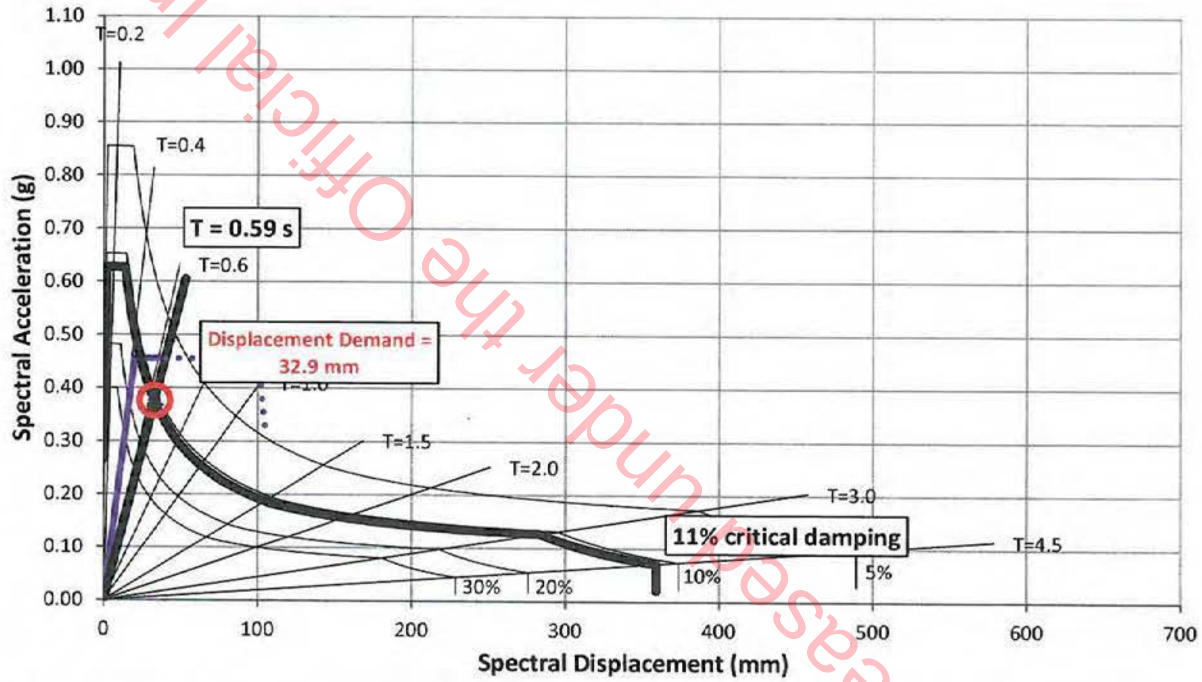
Effective Damping

Method: Custom

Effective system damping (@ ULS)	ξ_{eff}	11.0%

Results

Spectral displacement capacity	S_{d_ult} (mm)	40.0
Spectral displacement demand	SD (mm)	32.9
Calculated capacity ratio	NBS	121%
with 11% critical damping	at 0.59 s, ductility 2	



T-CED 272 (NZ): Building %NBS calculator for non-linear pushover method

Endorsed by the Building Structures PIN Committee. January 2013. Version 3.1

Modifications suggested by EL Blaikie April 2013

CALCULATION SHEET

Project/Task/File No:

Project Description:

Sheet No 73 of

Office:

Computed: / /

Check: / /

* Base shear on stairwell wall : $V_b = 220$ kN.

$\therefore$ $V_i = 0.6 \times V_b = 0.6 \times 220 = \underline{132 \text{ kN}}$.

* Estimate Vertical uplift (Y_{uplift})

- For eff height : $h_{eff} = 2/3\, h_{tot} = 2/3 \times 6.17 = \underline{4.1 \text{ m}}$.

- Horizontal Displacement Demand : $x_1 = 33$ mm

$\therefore$ $\tan\theta = \dfrac{x_1}{h_{eff}} = \dfrac{Y}{\ell}$

$\Rightarrow \dfrac{33}{4100} = \dfrac{Y}{6440} \Rightarrow \underline{Y_{up} = 51.8}$ mm.

Vertical Uplift

[Drift : $\dfrac{33}{4100} \times 100 = 0.8\%$]

CALCULATION SHEET

Project/Task/File No:

Project Description:

Sheet No 74 of

Office:

Computed: / /

Check: / /

Axial Load : N (kN)

+ Self-weight of wall

$25 \times [(0.8 \times 6.4 \times 6.17) - (0.8 \times 2.29 \times 1.97)] = 174.9$

+ Column self-weight

$25 \times 0.38 \times 0.38 \times 6.2 = 22.4$

+ Stairflight self-weight

$25 \times 0.127 \times 3.6 \times 4/2 = 22.9$

+ Footing self-weight

$25 \times 0.914 \times 0.46 \times 6.74 = 70.8$

291 kN

TABLE 4 SHEAR FORCE DISTRIBUTION based on ductility of μ=2 Y DIRECTION at Ground Storey

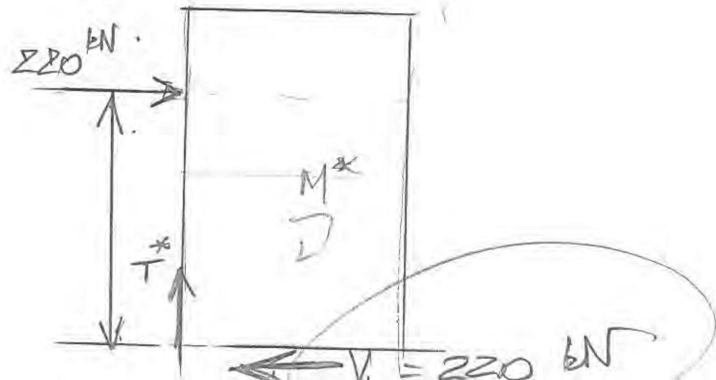
Modulus of Elasticity	E (MPa)	E (kPa)
Concrete walls	25000	25000000
Masonry walls	15000	15000000

$V_{o,\mu=2}$ (kN)	1934

	Wall Ref.	H (m)	Thick. t (m)	L_{yy} (m)	I (m^4)	E*I	EI/ΣEI	V^*_{Demand} (kN)
Concrete block walls	W1b	3.25	0.150	1.700	0.061	921188	0.004	8.17
	W1a	3.25	0.150	1.700	0.061	921188	0.004	8.17
	W11	3.20	0.200	7.315	6.524	97854995	0.449	867.42
	W12a	3.20	0.150	1.200	0.022	324000	0.001	2.87
	W12b	3.20	0.150	1.800	0.073	1093500	0.005	9.69
In-situ concrete walls	W12	3.80	0.180	3.910	0.897	22416177	0.103	198.70
	W13	3.80	0.180	3.910	0.897	22416177	0.103	198.70
	W14	3.80	0.200	3.910	0.996	24906863	0.114	220.78
	W15	3.80	0.200	3.910	0.996	24906863	0.114	220.78
	W16	3.80	0.180	3.910	0.897	22416177	0.103	198.70
			TOTAL	33.265		218177126	1.000	1934.00

Note: Distribution of forces was based on effective stiffness of walls (blockwalls + in-situ concrete)

BASED ON DUCTILITY OF μ=2.



$V_b = 220$ kN

FOR GRID B :

$M^* = 220 \times 4.9 = 1078$ kNm

$T^*_{pile} = \frac{1078}{4.0} = 269.5 \simeq 270$ kN.

Tension on Pile @ Grid B. on stairwall end.

Based on prorata for μ = 1.25 → $T^* = 1.6 \times 270$

$T^* = 432$ kN.

CALCULATION SHEET

Project/Task/File No: | Sheet No 76 of
Project Description: | Office:
Computed: / /
Check: / /

Tension Capacity on Pile.

For 12 No - 3/4" ϕ ~ 19 mm ϕ bars.

$T_u = 12 \times 283 \times 300 \times 10^{-3} = 1020$ kN.

$\therefore$ % NBS $= \dfrac{1020}{432} > 100\%$, ✓ok

+ Provided the starter bars at walls being 16 ϕ @ 250 crs, the flexure capacity of wall is $M_n = 2190$ kNm. (as calculated overleaf).

HENCE:

No issue with rebar yielding of pile &/ wall.

OPUS

FLEXURE CAPACITY OF WALL 76a

Opus International Consultants Limited

Checked : ______/____/____

WELCOME TO CONPROP(V 1.7) ** AN EXCEL SPREADSHEET FOR CALCULATING: GEOMETRIC PROPERTIES & MOMENTS FOR CONCRETE SECTIONS UNDER UNCRACKED, CRACKED & ULTIMATE CONDITIONS.

[Flanges (top and/or bottom) and axial load are optional. For walls/columns see note 2 or the MANUAL.]

Project:		
	Date: 18-Nov-15	Time: 13:29

STEP 1 ..... Describe the Uncracked Section...

(use consistent units.. e.g. N and mm through out the spreadsheet)

Total Section depth (d) =	3900	
Web width (w) =	200	
Top flange width **excluding** web (b1) =	0	<----
Top flange thickness (t) =	0	THESE
Bottom flange width **excluding** web (b2) =	0	6 values
Bottom flange thickness (b) =	0	may
Axial compressive load (P) and,	200,000	be
Depth from top surface of this load (di)	0	zero
Assumed tensile cracking stress (f't)	0	<------
Steel Elastic Modulus (Es)	200,000	

STEP 2 ..... Describe steel sizes and locations..................

describe location of the centroid of up to 10 bar bundles from either the top or the bottom surface. Describe Location of each bundle from only one surface.

Modular ratio (n=Es*(1+Ct)/Ec) =	8

TOP BARS No. Bars	Bar Diam	Distance From Top Surface	BOTTOM BARS No of Bars	Bar Diam	Distance From Bottom Surface
2	16.00	50.00	2	16.00	50.00
2	16.00	280.0	2	16.00	280.0
2	16.00	510.0	2	16.00	510.0
2	16.00	740.0	2	16.00	740.0
2	16.00	970.0	2	16.00	970.0
2	16.00	1200.0	2	16.00	1200.0
2	16.00	1430.0	2	16.00	1430.0
2	16.00	1660.0	2	16.00	1660.0
2	16.00	1890.0	2	16.00	1890.0
0	0.00	0.0	0	0.00	0.0

RESULTS FOR UN-CRACKED SECTION ANALYSIS:

(A) IGNORING EMBEDDED STEEL

Section Area (A)	780,000	7.80E+05
Depth Geometric Centroid (Ybar from top)	1950	1.95E+03
Moment of Inertia (Ig)	988650000000	9.89E+11
Cracking Moment (Mcr -tension bottom)	(260,000,000)	-2.60E+08
(Corresponding curvature)	0.0000000052	5.24E-09
Cracking Moment (Mcr - tension top)	520,000,000	5.20E+08
(Corresponding curvature)	0.0000000052	2.10E-08

(b) INCLUDING EMBEDDED STEEL (Using (n-1) times steel area)

Section Area (A)	830,473	8.30E+05
Depth Geometric Centroid (Ybar from top)	1950.0	1.95E+03
Moment of Inertia (Ig)	1054924136790	1.05E+12
Cracking Moment (Mcr -tension bottom)	(259,715,963)	-2.60E+08
(Corresponding curvature)	0.0000000	4.92E-09
Cracking Moment (Mcr - tension top)	520,284,037	5.20E+08
(Corresponding curvature)	0.00000000	4.92E-09

STEP 3 ..... Define variables for CRACKED SECTION ANALYSIS ....
(values entered in STEP 1 may be varied for this part of the analysis.)

Axial compressive load (P)	**200,000**
Depth from top surface of this load (di)	**0**
Crack root tensile stress (say 0.5f't)	**0**
Steel Yield Stress (Fy)...	**300**
* Max. Steel Stress (Tension side) (Fs) *........................	**300**
* Max tensile steel strain if steel yielding (es) *..............	**0.0015**
Steel Elastic Modulus (Es)...	**200,000**
Modular ratio (n=Es*(1+Ct)/Ec)	**8**

* Cracked section results below correspond to the conditions that exist when the Max tensile steel stress reaches Fs (if Fs< or = Fy) or to a yield strain of 'es' (if es>ey). Concrete is assumed to be ELASTIC.

RESULTS FOR CRACKED SECTION ANALYSIS (LINEAR CONCRETE PROPERTIES):

(a) CRACK PROPAGATING FROM BOTTOM ------------------------------------

Depth to N.A. from top (k).................	8.84E+2	
Maximum concrete stress (assumed elastic)............................	1.12E+1	
Crack Depth	3.02E+3	
Total Tension Force (including P)..	1.06E+6	Ratio T/C = 1.000 (=1.0 for iteration convergence)
Total Compression Force -incl. comp steel........................	1.06E+6	
Flexural Moment (M)---SEE NOTE 2.......................................	**2.19E+9**	
Curvature (= M/(Ec x Icr)- where Ec=Es/n)........................	5.06E-7	
Cracked MOI (=Icr from curvature)..	1.73E+11	
Crack width, mm (NZS3101-deformed bars & RC section only)	**0.34**	<< Reo centroid=970

* 'Top' bars & 'bottom' bars in compression assumed not to effect crack widths (see Note 3)

$M_y = 2194$ kNm.

(b) CRACK PROPAGATING FROM TOP ------------------------------------

Depth to N.A. from bottom (k)..	8.84E+2	
Maximum concrete stress (assumed elastic)............................	1.12E+1	
Crack Depth	3.02E+3	
Total Tension Force (including P)..	1.06E+6	Ratio T/C = 1.000 (=1.0 for iteration convergence)
Total Compression Force -incl. comp steel........................	1.06E+6	
Flexural Moment (M)---SEE NOTE 2.......................................	**2.97E+9**	
Curvature (= M/(Ec x Icr)- where Ec=Es/n)........................	5.06E-7	
Cracked MOI (=Icr from curvature)..	2.34E+11	
Crack width-mm (NZS3101-- deformed bars & RC section only	**0.34**	<< Reo centroid=970

** 'Bottom' bars & 'top' bars in compression assumed not to effect crack widths. When tension area (At) > flange area only bars in the tension flange are considered. Reo is assumed approx uniformly distributed in At. (see Note 3 for fuller details)

STEP 4 ..... Describe section properties for ULTIMATE conditions.........

(values entered in steps 1&2 may be varied for this part of the analysis.)

Concrete ultimate strain (e)	0.003
Ratio of (stress block)/(N.A.) depths..........................	0.850
Axial compressive load (P) and, ...	200,000
depth from top surface of this load (di)	0
Crack root tensile stress (say 0.5f't)	0.0
Concrete Elastic Modulus (Ec)	25,084
Concrete compressive strength (f'c)...	30
Steel Elastic Modulus (Es)..	200,000
Steel Yield Stress (Fy)..	300

Analysis results shown below correspond to the conditions that exist when the peak compression strain equals (e) given above. A rectangular stress block with average stress=0.85f'c is assumed.

RESULTS FOR ULTIMATE MOMENT SECTION ANALYSIS:

(a) CRACK PROPAGATING FROM BOTTOM

Depth to N.A. from top (c)..	4.31E+02	
Steel Stress (Maximum Tension).....................................	3.00E+02	
Crack Depth	3.47E+03	
Total Tension Force (including P)...................................	2.05E+06	Ratio T/C = 1.000
Total Compression Force -incl. comp steel.........................	2.05E+06	(=1.0 for iteration convergence)
Ideal Flex strength (Mi)---SEE NOTE 2.............................	3.79E+09	
Section Curvature (from curv = e/c)...............................	6.96E-06	

(b) CRACK PROPAGATING FROM TOP

Depth to N.A. from bottom (c)....................................	4.31E+02	
Steel Stress (Maximum Tension).....................................	3.00E+02	
Crack Depth	3.47E+03	
Total Tension Force (including P)...................................	2.05E+06	Ratio T/C = 1.000
Total Compression Force -incl. comp steel.........................	2.05E+06	(=1.0 for iteration convergence)
Ideal Flex strength (Mi)---SEE NOTE 2.............................	4.57E+09	
Section Curvature (from curv = e/c)...............................	6.96E-06	

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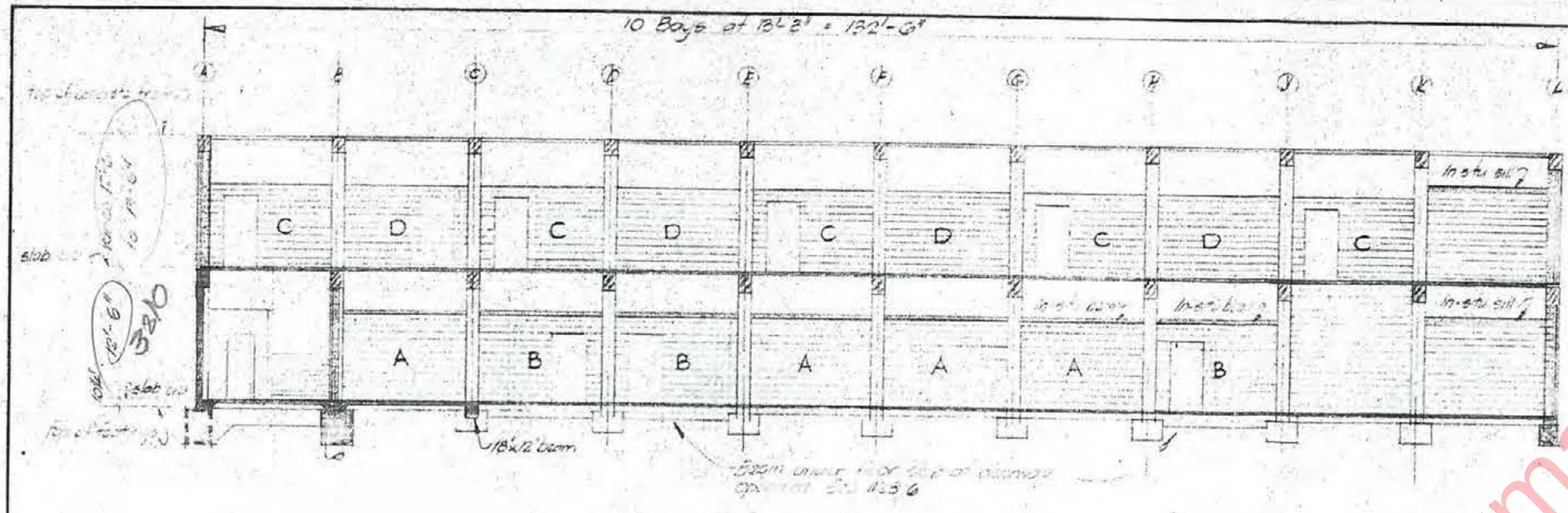
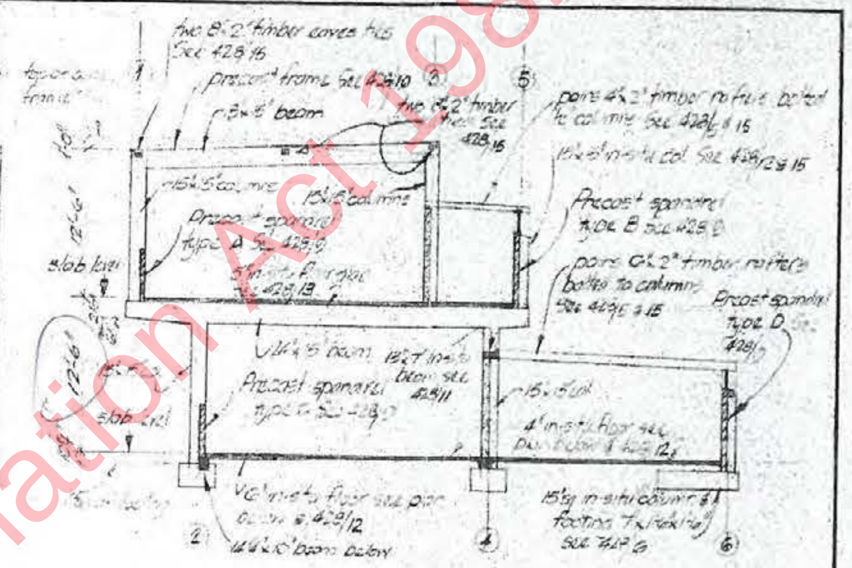


LONGITUDINAL SECTION AT GRID 3



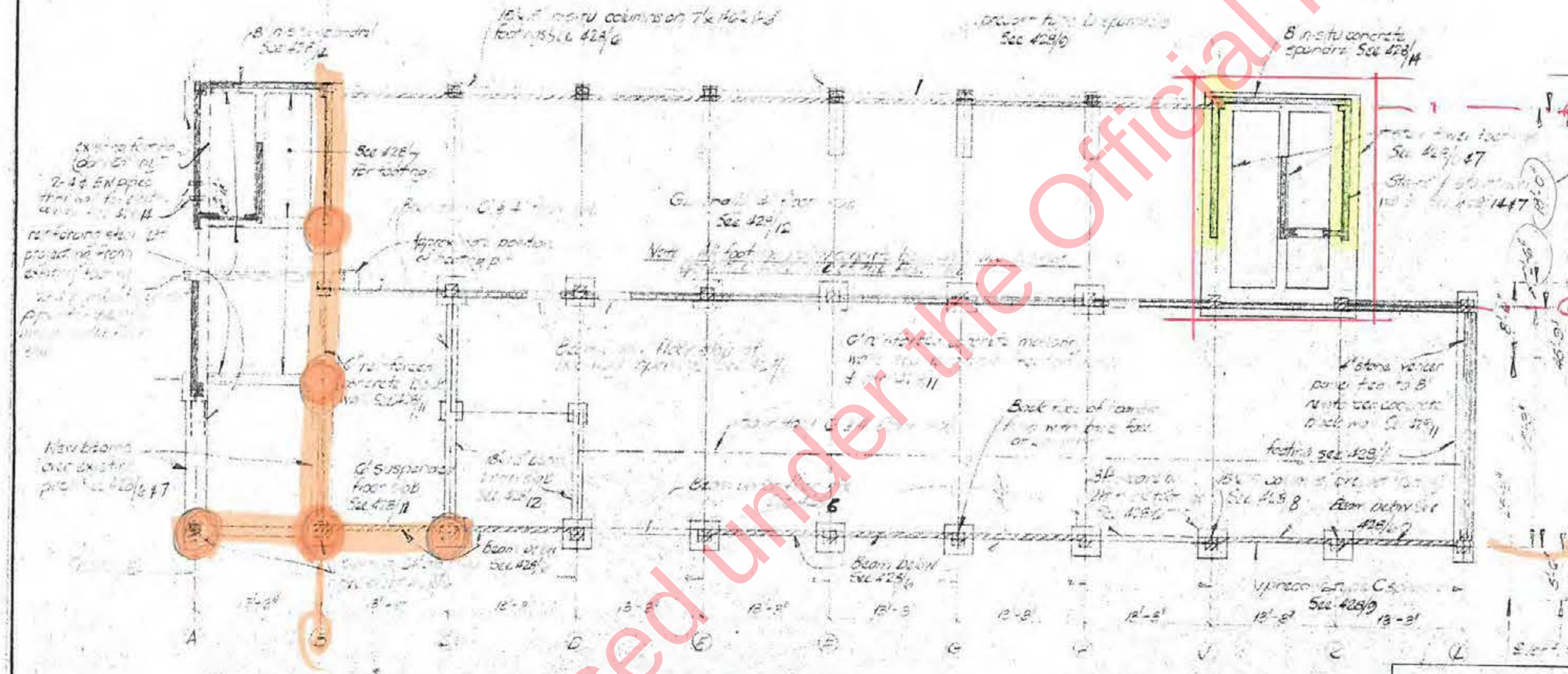
TYPICAL SECTION GRIDS C TO H

41
49100

THIS IS ONE OF THE DRAWINGS REFERRED TO IN THE AGREEMENT WITH THE WELLINGTON EAST GIRLS' COLLEGE BOARD FOR THE ERECTION OF A NEW CLASSROOM BLOCK AT WELLINGTON EAST GIRLS' COLLEGE.

X Contractor......................

Date..24th.December.1954



GROUND FLOOR PLAN & FOUNDATION LAYOUT

DRAWING INDEX

	Drg. No.
428/4 Structural Carcase - Ground floor - foundation layout & typical sections	24
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6 Foundation details	26
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12 Ground Floor Slab	32
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14 Concrete Stair Towers	34
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16 Ground floor upstands, wallbars & miscellaneous details	36

In-situ columns & walls shown -
Precast columns & walls -
Reinforced concrete masonry

Nº	AMENDMENTS	BY	DATE

SPENCER, HOLLINGS & FERNER
CONSULTING CIVIL & STRUCTURAL ENGINEERS
21 EVERTON TCE., WELLINGTON C.1.

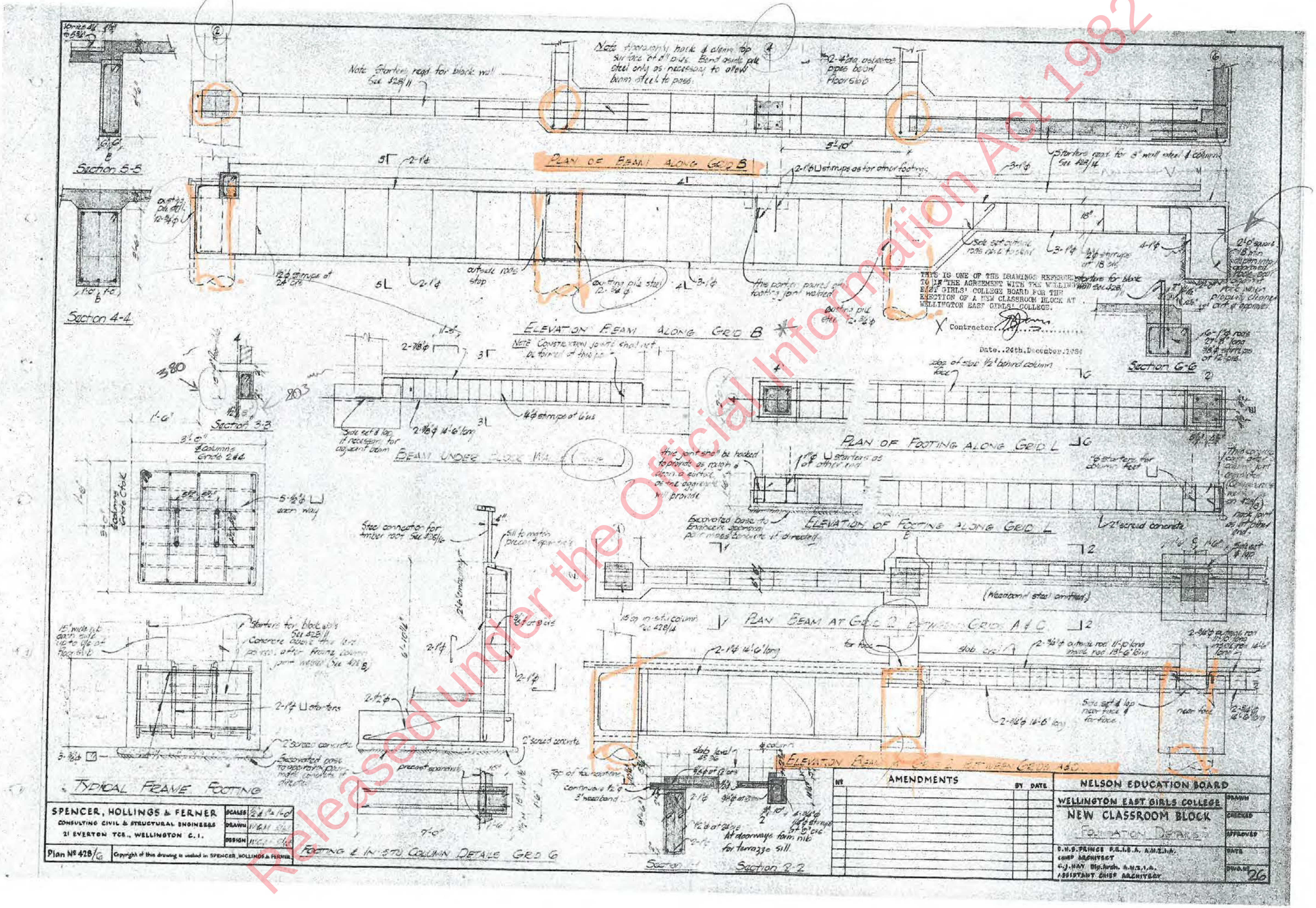
SCALES	1/8" = 1'-0"
DRAWN	WCM 4/64
DESIGN	WCM 4/64

Plan Nº 428/4

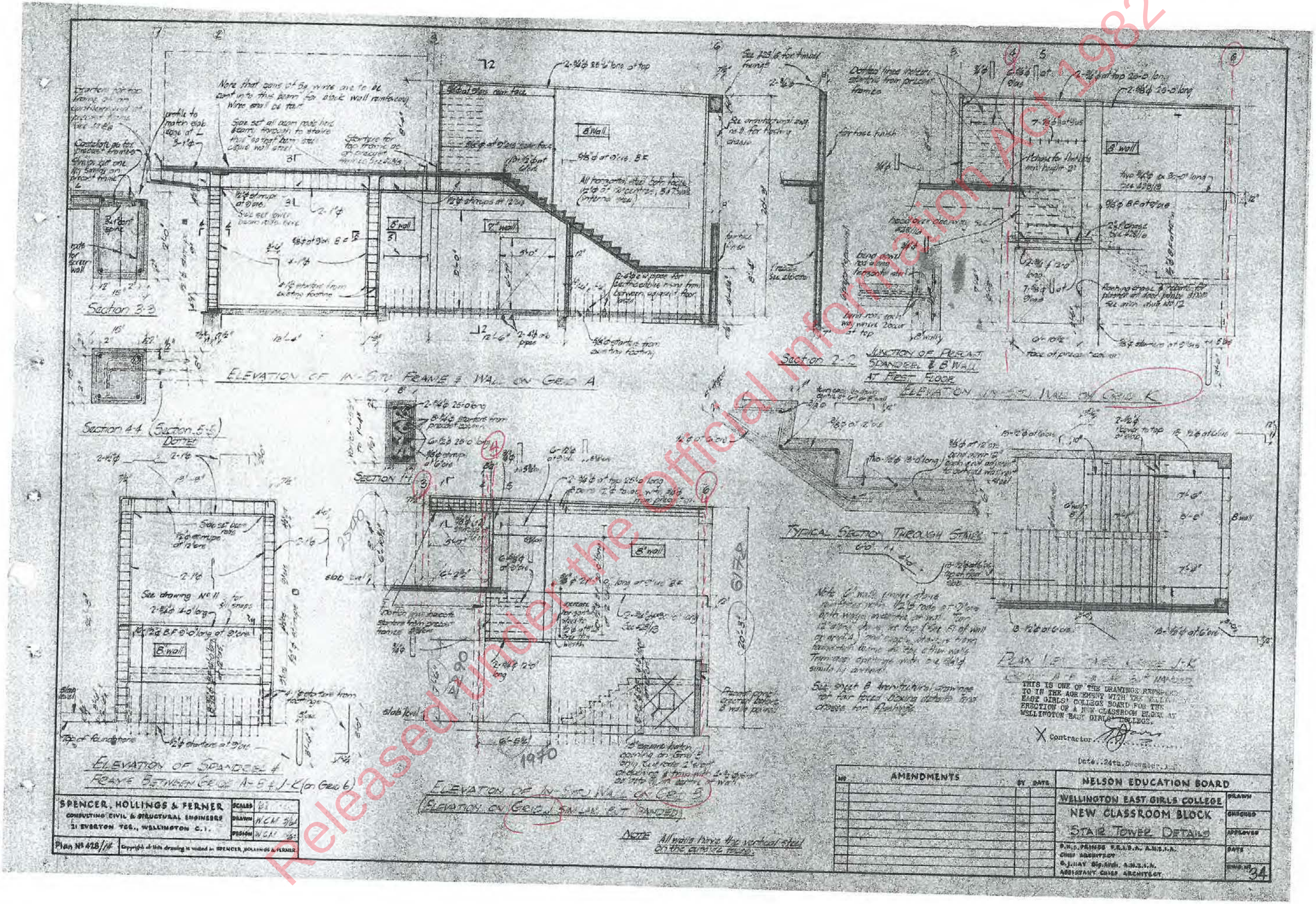
NELSON EDUCATION BOARD
WELLINGTON EAST GIRLS COLLEGE
NEW CLASSROOM BLOCK
STRUCTURAL CARCASE
GROUND FLOOR PLAN & TYPICAL SECTIONS
D.H.S. PRINCE F.R.I.B.A. A.N.Z.I.A.
CHIEF ARCHITECT
C.J. HAY Dip.Arch. A.N.Z.I.A.
ASSISTANT CHIEF ARCHITECT

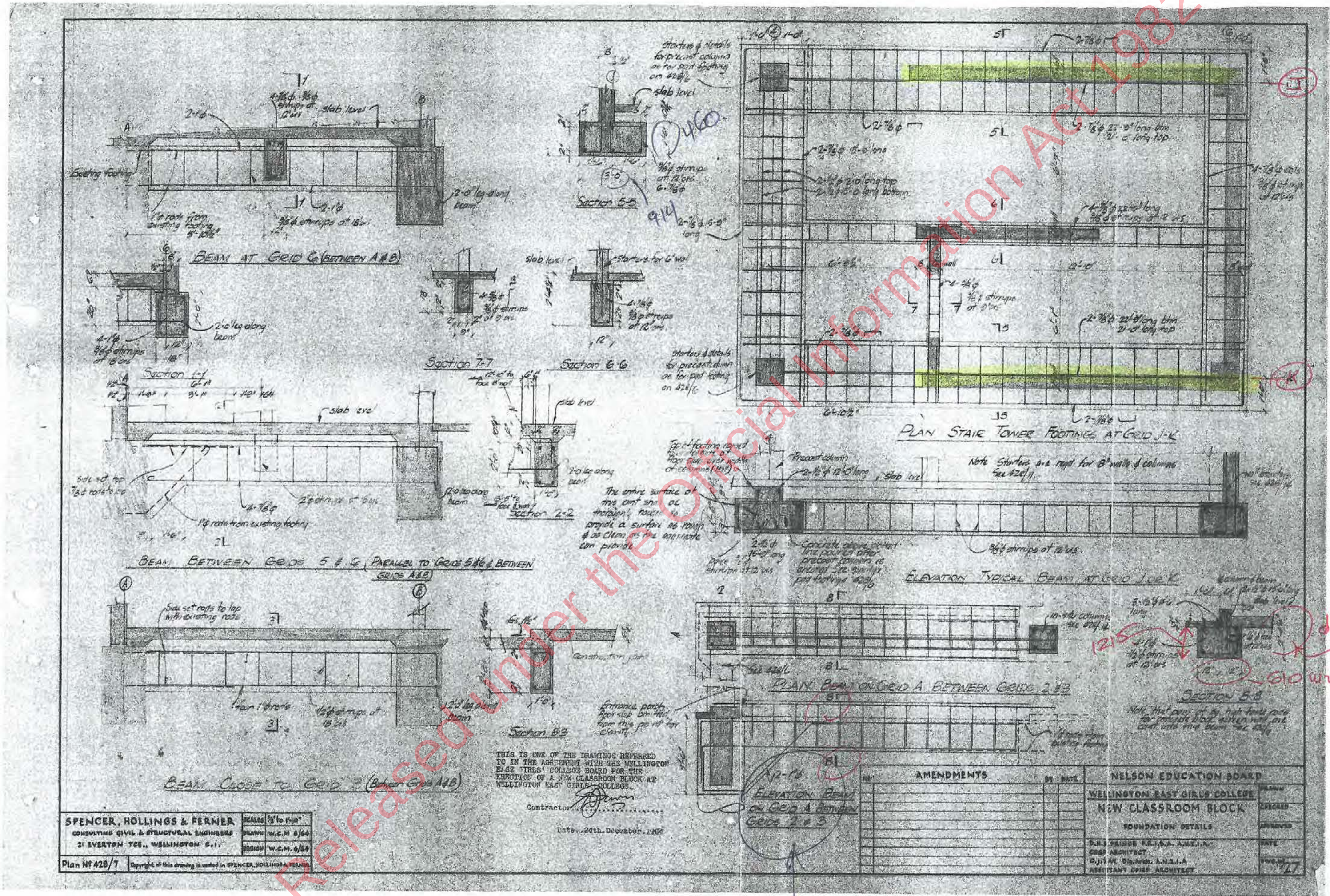
DRAWN
CHECKED
APPROVED
DATE
DWG. Nº 24

-78.



610×610 × 460 deep


Section 3-3
ELEVATION OF IN-SITU FRAME & WALL ON GRID A
Section 2-2
JUNCTION OF PRECAST SPANDREL & 8" WALL AT FIRST FLOOR
ELEVATION OF IN-SITU WALL ON GRID K
Section 4-4 (Section 5-5) Dotted
SECTION H-H
TYPICAL SECTION THROUGH STAIRS
ELEVATION OF SPANDREL & FRAME BETWEEN GRIDS A-B & J-K (on Grid 6)
ELEVATION OF IN-SITU WALL ON GRID 3
(ELEVATION ON GRID J SIMILAR BUT HANDED)
NOTE All walls have the vertical steel on the outside face.
8" wall
THIS IS ONE OF THE DRAWINGS REFERRED TO IN THE AGREEMENT WITH THE WELLINGTON EAST GIRLS' COLLEGE BOARD FOR THE ERECTION OF A NEW CLASSROOM BLOCK AT WELLINGTON EAST GIRLS' COLLEGE.
Contractor.
AMENDMENTS
NELSON EDUCATION BOARD
WELLINGTON EAST GIRLS COLLEGE
NEW CLASSROOM BLOCK
STAIR TOWER DETAILS
CHIEF ARCHITECT
ASSISTANT CHIEF ARCHITECT
SPENCER, HOLLINGS & FERNER
CONSULTING CIVIL & STRUCTURAL ENGINEERS
21 EVERTON TCE., WELLINGTON C.1.
Plan No 428/14
Copyright of this drawing is vested in SPENCER, HOLLINGS & FERNER
SCALES
DRAWN
DESIGN
CHECKED
APPROVED
DATE
34



For Wall ref. W12a/W12b.

Appendix B

Photos of Building

Photo 1: North elevation showing veneer on top floor infill panel

Photo 2: North-west corner elevation showing cracking at the additional column on west elevation

Photo 3: North end of West elevation with the short height of precast panels only at first floor level

Photo 4: West elevation having additional perimeter columns at ground floor to line up with original perimeter columns at first floor level

Photo 5: Mid-section of West elevation

Photo 6: Details of the concrete spalling at first floor level and the horizontal cracking of the additional perimeter columns at ground level

Photo 7: Addition of east elevation at north end

Photo 8: View of the concrete block walls at ground and metal cladding with timber framing at first floor.

Photo 9: East elevation

Photo 10: Detail of east elevation with the original precast panels at ground floor level

Photo 11: Corridor at ground floor level showing the addition/extension

Photo 12: Same elevation with new timber partition walls

Photo 13: Corridor at ground floor level showing the block wall line

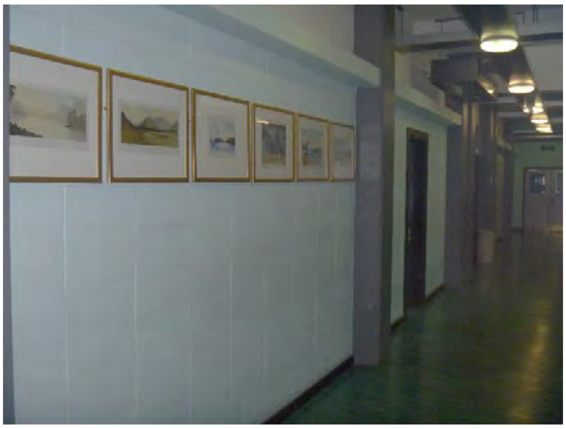


Photo 14: Same elevation further along

Photo 15: End of original concrete frame at ground at the corridor side

Photo 16: Concrete frame at ground floor towards the west end with the new column along perimeter

Photo 17: Corridor at 1st floor towards west elevation with the original concrete block walls

Photo 18: Corridor at 1st floor towards the east elevation with the new timber wall additions

Photo 19: Opening/ floor void facing towards the east elevation with part of the addition at 1st floor

Photo 20: Typical classroom at 1st floor facing at west elevation

Appendix C

Plans of Building

Wellington East Girls College: Source (LINZ Data Service)