The Ministry of Tourism and Antiquities
The Ministry of Planning

Detailed Design for Tourism Sector Development Project in the Hashemite Kingdom of Jordan

Final Report

Supporting Report
Volume 4SR
Dead Sea Parkway Sub-project

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The JICA D/D Study Team
Joint Venture:
Pacific Consultants International, Tokyo
Yamashita Sekkei Inc

DEAD SEA PARKWAY SUB-PROJECT

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BRIDGES STRUCTURAL CALCULATIONS

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1. General

1. General

This report contains the structural calculations of both WADI HIMARA BRIDGE and WADI EL-ASAL BRIDGE located in Jordan.

The design is performed in accordance with rules and regulations, valid for the project.

The design is performed for superstructure (including deck slab, main girder, diaphragm, bearing and expansions joint) and substructure (including pier and abutment). Rock bearing capacity is also estimated.

2. Superstructure

2.1 Deck slab Design:

Material Properties:

1.1 Concrete:

- Cylinder strength (Class 30): $f_{c'} := 26.1$ MPa
- Concrete Density: $\gamma_c := 25$ kN/m³

1.2 Reinforcing Steel:

- Deformed bar of high tensile strength, Grade 60, conforming to JSS/441/1986 or AASHTO M31M (ASTM, A615M) :
- Yield strength : $f_y := 410$ MPa

1.3 Asphalt wearing surface:

- Density: $\gamma_a := 18$ kN/m^3
- Thickness of asphalt overlay: $t_a := 0.05$ m
- In Final Design Report, 3.4.2 Bridge Design said that "Additional 5 cm of future wearing surface shall be considered", therefore, let say, future wearing surface thickness equals to 10 cm.

$t_a := 0.10$ m

Minimum slab thickness:

- Design width of slab:

$b := 1.00$ m

- Spacing between girder (slab span):

$S_{cc} := 2.20$ m

- Top flange width of I-girder:

$t_b := 1.05$ m

- Web thickness of I-girder

$w_b := 0.20$ m

- Ratio of top flange width to minimum thickness:

$\text{Ratio} := \dfrac{t_b}{w_b}$ Ratio = 5.25 > 4.0

- AASHTO 3.24.1.2 (b) said that "S" shall be the distance between edges of top flange plus one-half of stringer top flange width.
- So the effective slab span length is:

$S := S_{cc} - t_b + \dfrac{t_b}{2}$ S = 1.675 m.

- Required slab thickness: $t_{req} := \frac{S + 3.05}{9.15} \cdot 30.48$

  $t_{req} = 15.74$ cm.

- Provided slab thickness: $t := 20$ cm

- Since $t = 20$ cm > $t_{req} = 15.7$ cm OK

Loading:

1. Dead load (DL):

$DL := \frac{t}{100} \cdot \gamma_c$ $\quad DL = 5$ kN/m2

2. Superimposed Dead Load (SDL):

$SDL := t_a \cdot \gamma_a$ $\quad SDL = 1.8$ kN/m^2

Sum of dead loads & superimposed dead loads is

$DL := DL + SDL$ $\quad DL = 6.8$ kN/m^2

Moment due to DL+SDL:

$M_{DL} := \frac{DL \cdot S^2}{10}$ $\quad M_{DL} = 1.91$ kN-m

3. Live Load (LL):

AASHTO HS20-44 + 50%

Percent increase in LL: $Percent := 50\ \%$

AASHTO truck wheel load: $P := 72$ kN

For slab continuous over three or more supports:

$f_{cont} := 0.80$

AASHTO 3.24.3.1 Case A : Main R/F perpendicular to traffic

$$M_{LL} := \frac{(S + 0.61)}{9.74} \cdot P \cdot (1 + Percent) \cdot f_{cont}$$

$M_{LL} = 20.269$ kN-m

Impact factor for slab design with: $L := S$

$I_L := \frac{15.24}{L + 38}$ $\quad I_L = 0.384$ $\quad I_{allow} := 0.30$

$I := \left[I_L \ \ I_{allow} \right]$ $\quad I := \min(I)$ $\quad I = 0.3$

Load Combinations:

Group I: $Factored\_Load := \gamma \cdot \left[\beta_D \cdot D + \beta_L \cdot (L + I) \right]$

Where : $\gamma := 1.3$ $\beta_D := 1.0$ $\beta_L := 1.67$ $I = 0.3$

Design Moment:

Therefore, design ultimate moment is:

$$M_\gamma := \gamma \cdot \left[\beta_D \cdot (M_{DL}) + \beta_L \cdot (M_{LL} + I \cdot M_{LL}) \right]$$

$M_\gamma = 60$ kN-m

Reinforcement Design:

- Select reinforcement:

Bar diameter : $\phi_b := 16$ mm

Cross section area : $A_b := \dfrac{\pi \cdot \phi_b^2}{4}$ $A_b = 201.062$ mm^2

From Vol. (III) Design Drawings,
General Notes: minimum concrete cover for deck slab is 50 mm.

$cov := 50$ mm.

Effective depth: $d := \dfrac{t}{100} - \dfrac{cov}{1000} - \dfrac{\phi_b}{2 \cdot 1000}$ $d = 0.142$ m.

Reinforcement spacing: $s := 0.15$ m.

Number of rebars: $No := \dfrac{b}{s}$ $No = 6.667$

Total reinforcement area: $A_s := No \cdot A_b$ $A_s = 1.34 \cdot 10^3$ mm^2

For flexural design: $\phi := 0.90$

$a := \dfrac{A_s \cdot f_y \cdot 10^{-6}}{0.85 \cdot f_c{}' \cdot b}$ $a = 0.025$ m

$$M_u := \left[\phi \cdot A_s \cdot f_y \cdot \left(d - \frac{a}{2} \right) \right] \cdot 10^{-3}$$

$M_u = 64$ kN-m/m > $M_\gamma = 60$ kN-m/m. OK

USE DB16@0.15 c/c for Top & Bottom reinforcement

Distribution Reinforcement:

For main R/F perpendicular to traffic % of distribution steel should be:

$Percentage := \frac{121}{\sqrt{S}}$ $Percentage = 93.493$ %

$P_{allow} := 67$ %

$P := [Percentage \;\; P_{allow}]$ $P := \min(P)$ $P = 67$ %

So ⇒ $A_{s\_t} := \frac{P}{100} \cdot A_s$

$A_{s\_t} = 898$ mm^2

- Provided reinforcement:

Bar diameter :	$\phi_b := 14$	mm		
Cross section area :	$A_b := \frac{\pi \cdot \phi_b^{\,2}}{4}$		$A_b = 153.938$	mm^2
Spacing:	$s := 0.17$	m.		
Number of rebars:	$No := \frac{b}{s}$		$No = 5.882$	
Total reinforcement area:	$A_{s\_prov} := No \cdot A_b$		$A_{s\_prov} = 906$	mm^2 OK

USE DB14@0.17 c/c for Top & Bottom Distribution steel or :

- Select new bar size:

Bar diameter :	$\phi_b := 12$	mm		
Cross section area :	$A_b := \frac{\pi \cdot \phi_b^{\,2}}{4}$		$A_b = 113.097$	mm^2
Spacing:	$s := 0.125$	m.		
Number of rebars:	$No := \frac{b}{s}$		$No = 8$	
Total reinforcement area:	$A_{s\_prov} := No \cdot A_b$		$A_{s\_prov} = 905$	mm^2 OK

USE DB12 @ 0.125 c/c for Top & Bottom Distribution steels:

2.2 Main Girder:

Design Data:

- Prestressed Concrete I-Girder Bridge (Precast Post-tensioned I-Beam)
- Two traffic lanes, total lane width (m.): $w_{lane} = 9.80$
- Span length between piers (m.): $L = 30.00$
- Typical spacing between girders (m.): $S = 2.20$
- Live loads based on AASTHO 1996 + 50% as per MPWH : $HS20 - 44 + 50\,\%$
- Units are: m, kN, MPa and others as stated.

Material Properties:

1. Concrete:

- Deck Slab, Class C30 with: $f_{c'\_s} = 26.1$ MPa
- Prestressed I-Girder, Class C40 with: $f_{c'\_g} = 34.8$ MPa

  Strength at time of prestressing: $f_{ci'} = 32$ MPa
- Concrete density: $\gamma_c = 25$ kN/m^3

2. Reinforcing Steel:

- Deformed Bar (DB) Grade 60 per JSS/441/1986 or AASHTO M31M (ASTM A615M) :

  $f_{y\_DB} = 410$ MPa
- Young modulus of elasticity: $E_s = 2.0 \cdot 10^5$ MPa

3. Prestressing Steel:

- Strands: Cold Drawn, Low Relaxation 7-Wire Strands, as per AASHTO M203 (A416) Grade 270 K or equivalent.
- 0.5" strand:

  $\phi = 12.7$ mm. $A_{ps} = 98.71$ mm^2

  Min. breaking strength: $f_{s'} = 1861$ MPa

4. Wearing Surface:

- Wearing surface use 5 cm asphalt overlay + additional 5 cm in the future:

  $h_w = 0.10$ m.
- Asphaltic Concrete density: $\gamma_{aspt} = 18$ kN/m^3

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Allowable Stresses

1. Reinforcing Steel: $f_{s\_DB} := 0.5 \cdot f_{y\_DB}$ $f_{s\_DB} = 205$ MPa

2. Prestressing Steel:

- Initial prestressing, $f_{pi} := 0.70\ f_{s'}$ $f_{pi} = 1303$ MPa

3. Concrete:

- RC Deck slab: $f_{c\_s} := -0.4\ f_{c'\_s}$ $f_{c\_s} = -10.44$ MPa

- Prestressed Concrete I-Girder:

- At transfer: $f_{tt} := 0.249 \cdot \sqrt{f_{ci'}}$ $f_{tt} = 1.41$ MPa

$f_{ct} := -0.55 \cdot f_{ci'}$ $f_{ct} = -17.60$ MPa

- At working: $f_{tw} := 0.498 \cdot \sqrt{f_{c'\_g}}$ $f_{tw} = 2.94$ MPa

$f_{cw} := -0.4\ f_{c'\_g}$ $f_{cw} = -13.92$ MPa

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Section Properties:

1. Precast Girder:

1.1 Typical Cross section:

- Heigth of girder: $h_g := 1.80$ m
- Heigth of flange: $h_f := 0.125$ m
- Width of girder connected to deck slab: $b_{wg\_s} := 1.05$ m
- From NovaDesign result, see Fig. 2.1
  - Cross-sectional area: $A_g := 0.678125$ m^2
  - Moment of inertia: $I_g := 0.286453$ m^4
  - Center of gravity from bottom: $c_{bg} := 0.910$ m
  - Center of gravity from top: $c_{tg} := h_g - c_{bg}$ $\quad c_{tg} = 0.890$ m
- Section modulus

$$S_{tg} := \frac{I_g}{c_{tg}} \qquad S_{tg} = 0.322 \quad m^3$$

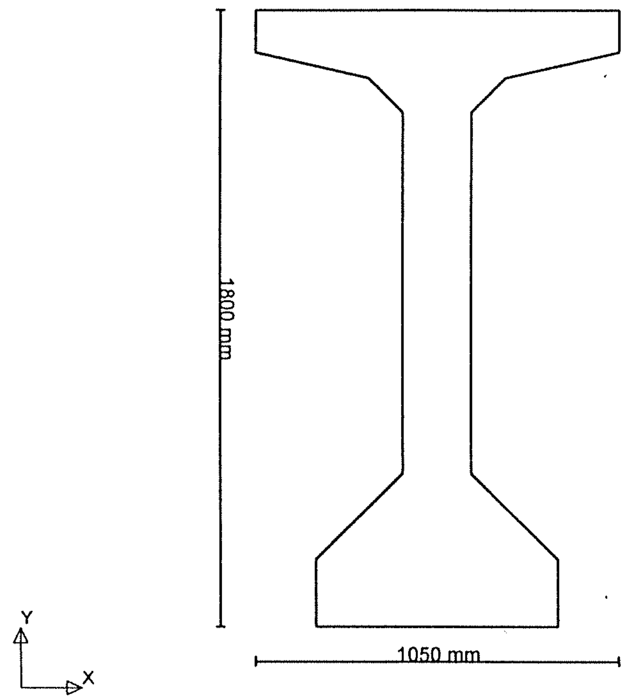
$$S_{bg} := \frac{I_g}{c_{bg}} \qquad S_{bg} = 0.315 \quad m^3$$

1.2 End Cross section (at support):

- From NovaDesign result, see Fig. 2.2
  - Cross-sectional area: $A_{gs} := 1.316875$ m^2
  - Moment of inertia: $I_{gs} := 0.376690$ m^4
  - Center of gravity from bottom: $c_{bs} := 0.935$ m
  - Center of gravity from top: $c_{ts} := h_g - c_{bs}$ $\quad c_{ts} = 0.865$ m
- Section modulus

$$S_{ts} := \frac{I_{gs}}{c_{ts}} \qquad S_{ts} = 0.435 \quad m^3$$

$$S_{bs} := \frac{I_{gs}}{c_{bs}} \qquad S_{bs} = 0.403 \quad m^3$$



Section:

No. **Name**
1.0 Interior girder

Concrete:
fck = 40 MPa
Rho = 2500 kg/m3

Section properties:
Ac = 0.678125 m2
Ix = 0.286453 m4
Iy = 0.024070 m4
It = 0.000000 m4

Center of gravity:
CoGx = 0.000 m
CoGy = 0.910 m

Total reinforcement area:
As = 0 mm2

Total prestressing area:
Asp = 0 mm2

Fig. 2.1 Typical Cross-section

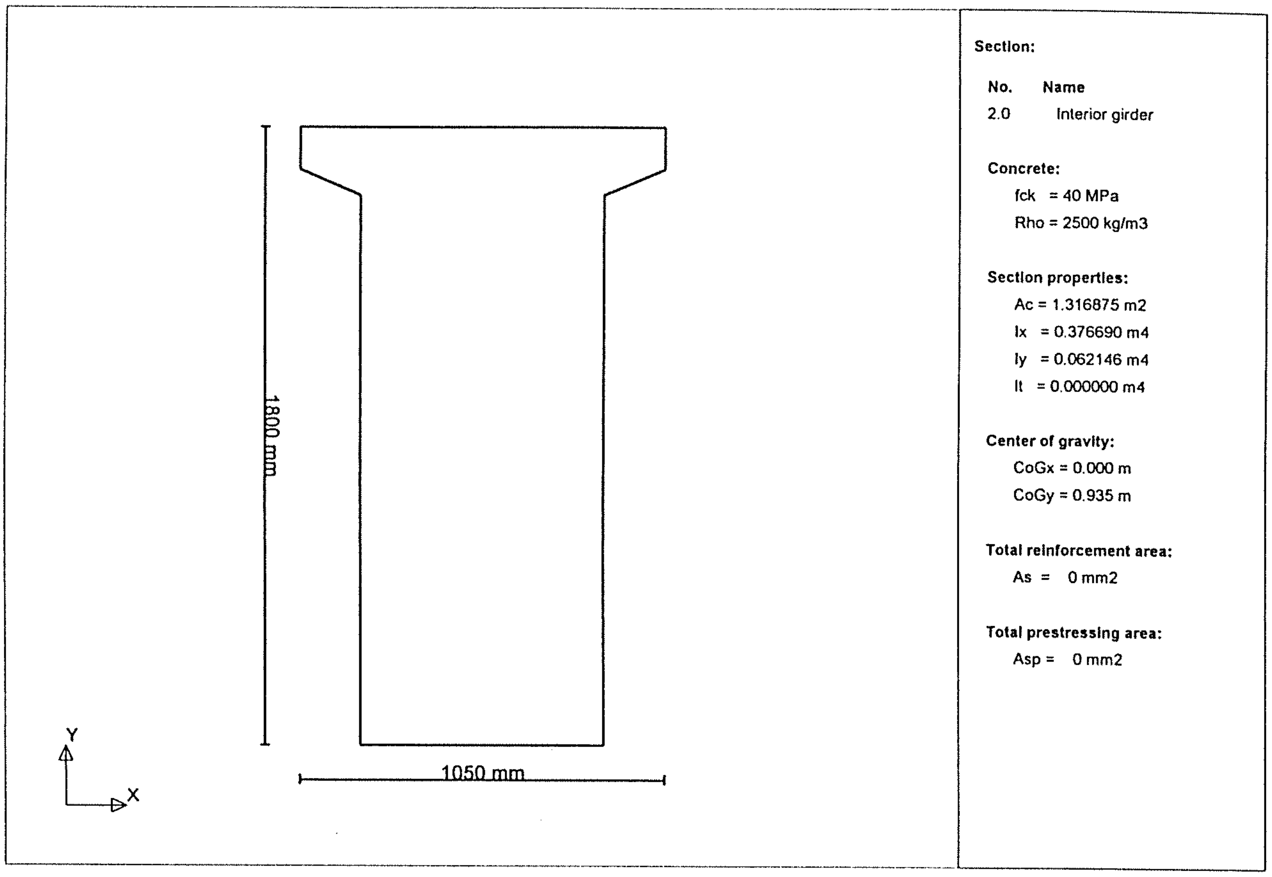


Fig. 2.2 End section

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2. Composite Section:

Thickness of RC deck slab: $h_s := 0.20$ m

Effective width of deck slab

1. 1/4 span length: $b_1 := \frac{1}{4}\cdot L$ $\quad b_1 = 7.500$ m
2. Girder spacing: $b_2 := S$ $\quad b_2 = 2.200$ m
3. 12 times flange width + width of web: $b_3 := 12\cdot h_f + b_{wg\_s}$ $\quad b_3 = 2.55$ m

$b := \begin{bmatrix} b_1 & b_2 & b_3 \end{bmatrix}$

Therefore, effective width of deck slab: $b_e := \min(b)$ $\quad b_e = 2.20$ m

Ratio of Young modulus of Slab & Girder is: $n_c := \sqrt{\frac{f_{c'\_s}}{f_{c'\_g}}}$ $\quad n_c = 0.866$

Composite effective width of deck slab: $b_{ec} := n_c\cdot b_e$ $\quad b_{ec} = 1.905$ m

Area of effective composite deck slab: $A_{cs} := h_s\cdot b_{ec}$ $\quad A_{cs} = 0.381$ m^2

Properties of Composite section:

$A_g = 0.678$ $\quad y_g := c_{bg}$ $\quad y_g = 0.910$ m (from bottom) $\quad I_g = 0.286$ m^4

$A_{cs} = 0.381$ $\quad y_s := h_g + \frac{h_s}{2}$ $\quad y_s = 1.900$ m

$A_c := A_g + A_{cs}$ $\quad A_c = 1.059$

$I_s := \frac{b_{ec}\cdot h_s^3}{12}$ $\quad I_s = 1.27\cdot 10^{-3}$ m^4

$y_c := \frac{y_g\cdot A_g + y_s\cdot A_{cs}}{A_g + A_{cs}}$ $\quad y_c = 1.266$ m

$I_c := I_g + A_g\cdot(y_c - y_g)^2 + I_s + A_{cs}\cdot(y_c - y_s)^2$ $\quad I_c = 0.527$ m^4

$c_{bc} := y_c$ $\quad c_{bc} = 1.266$ m $\quad c_{tc} := h_g + h_s - c_{bc}$ $\quad c_{tc} = 0.734$ m

$S_{tc} := \frac{I_c}{c_{tc}}$ $\quad S_{tc} = 0.718$ m^3 $\quad S_{bc} := \frac{I_c}{c_{bc}}$ $\quad S_{bc} = 0.416$ m^3

$m_t := \frac{S_{tg}}{S_{tc}}$ $\quad m_t = 0.448$ $\quad m_b := \frac{S_{bg}}{S_{bc}}$ $\quad m_b = 0.757$

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3. Prestressing Steel:

Number of cable to be used: $N_c = 6$

Number of strands to be used in Cable 1-4: $N_{c1} = 9$

Number of strands to be used in Cable 5- 6: $N_{c5} = 9$

Cross section area of prestressing steel:

Prestressing area of Cable 1-4: $A_{ps1} = N_{c1} \cdot A_{ps}$ $A_{ps1} = 888.390$ mm^2

Prestressing area of Cable 5-6: $A_{ps5} = N_{c5} \cdot A_{ps}$ $A_{ps5} = 888.390$ mm^2

Centroid of cable from bottom face:

- At Midspan:

Centroid of Cable 1: $y_{m1} = 0.10$ m.

Centroid of Cable 2: $y_{m2} = 0.10$ m.

Centroid of Cable 3: $y_{m3} = 0.10$ m.

Centroid of Cable 4: $y_{m4} = 0.20$ m.

Centroid of Cable 5: $y_{m5} = 0.30$ m.

Centroid of Cable 6: $y_{m6} = 0.40$ m.

- At Support:

Centroid of Cable 1: $y_{s1} = 0.3475$ m.

Centroid of Cable 2: $y_{s2} = 0.5685$ m.

Centroid of Cable 3: $y_{s3} = 0.7895$ m.

Centroid of Cable 4: $y_{s4} = 1.0105$ m.

Centroid of Cable 5: $y_{s5} = 1.2315$ m.

Centroid of Cable 6: $y_{s6} = 1.4525$ m.

- Eccentricity of each cable from C.G. of girder:

- At Midspan:

Eccentricity of Cable 1: $e_{m1} = c_{bg} - y_{m1}$ $e_{m1} = 0.81$ m.

Eccentricity of Cable 2: $e_{m2} = c_{bg} - y_{m2}$ $e_{m2} = 0.81$ m.

Eccentricity of Cable 3: $e_{m3} = c_{bg} - y_{m3}$ $e_{m3} = 0.81$ m.

Eccentricity of Cable 4: $e_{m4} = c_{bg} - y_{m4}$ $e_{m4} = 0.71$ m.

Eccentricity of Cable 5: $e_{m5} = c_{bg} - y_{m5}$ $e_{m5} = 0.61$ m.

Eccentricity of Cable 6: $e_{m6} = c_{bg} - y_{m6}$ $e_{m6} = 0.51$ m.

Average eccentricity: $$e_{mg} = \frac{e_{m1} + e_{m2} + e_{m3} + e_{m4} + e_{m5} + e_{m6}}{6}$$

$e_{mg} = 0.710$ m.

- At Support:

Eccentricity of Cable 1:	$e_{s1} = e_{bs} - y_{s1}$	$e_{s1} = 0.59$	m.
Eccentricity of Cable 2:	$e_{s2} = e_{bs} - y_{s2}$	$e_{s2} = 0.37$	m.
Eccentricity of Cable 3:	$e_{s3} = e_{bs} - y_{s3}$	$e_{s3} = 0.15$	m.
Eccentricity of Cable 4:	$e_{s4} = e_{bs} - y_{s4}$	$e_{s4} = -0.08$	m.
Eccentricity of Cable 5:	$e_{s5} = e_{bs} - y_{s5}$	$e_{s5} = -0.30$	m.
Eccentricity of Cable 6:	$e_{s6} = e_{bs} - y_{s6}$	$e_{s6} = -0.52$	m.

Average eccentricity: $e_{sg} = \dfrac{e_{s1} + e_{s2} + e_{s3} + e_{s4} + e_{s5} + e_{s6}}{6}$ $e_{sg} = 0.035$ m.

- **Eccentricity of each cable from C.G. of Composite section:**

- At Midspan:

Eccentricity of Cable 1:	$e_{mc1} = e_{bc} - y_{m1}$	$e_{mc1} = 1.17$	m.
Eccentricity of Cable 2:	$e_{mc2} = e_{bc} - y_{m2}$	$e_{mc2} = 1.17$	m.
Eccentricity of Cable 3:	$e_{mc3} = e_{bc} - y_{m3}$	$e_{mc3} = 1.17$	m.
Eccentricity of Cable 4:	$e_{mc4} = e_{bc} - y_{m4}$	$e_{mc4} = 1.07$	m.
Eccentricity of Cable 5:	$e_{mc5} = e_{bc} - y_{m5}$	$e_{mc5} = 0.97$	m.
Eccentricity of Cable 6:	$e_{mc6} = e_{bc} - y_{m6}$	$e_{mc6} = 0.87$	m.

Average eccentricity: $e_{mc} = \dfrac{e_{mc1} + e_{mc2} + e_{mc3} + e_{mc4} + e_{mc5} + e_{mc6}}{6}$ $e_{mc} = 1.066$ m.

- At Support:

Eccentricity of Cable 1:	$e_{sc1} = e_{bc} - y_{s1}$	$e_{sc1} = 0.92$	m.
Eccentricity of Cable 2:	$e_{sc2} = e_{bc} - y_{s2}$	$e_{sc2} = 0.70$	m.
Eccentricity of Cable 3:	$e_{sc3} = e_{bc} - y_{s3}$	$e_{sc3} = 0.48$	m.
Eccentricity of Cable 4:	$e_{sc4} = e_{bc} - y_{s4}$	$e_{sc4} = 0.26$	m.
Eccentricity of Cable 5:	$e_{sc5} = e_{bc} - y_{s5}$	$e_{sc5} = 0.03$	m.
Eccentricity of Cable 6:	$e_{sc6} = e_{bc} - y_{s6}$	$e_{sc6} = -0.19$	m.

Average eccentricity: $e_{sc} = \dfrac{e_{sc1} + e_{sc2} + e_{sc3} + e_{sc4} + e_{sc5} + e_{sc6}}{6}$ $e_{sc} = 0.366$ m.

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2.2.1 Service Limit State (SLS):

2.2.1.1 At Transfering:

Only dead loads from Precast I-Girder & prestressing force to be considered:

I-Girder:

- Dist. c/l bearing to c/l of pier (m.): $W_b = 0.60$
- Net span length of girder (m.): $L_g = L - W_b$ $L_g = 29.40$
- Typical section: $A_g = 0.678$ m^2
- End section: $A_{gs} = 1.317$ m^2
- Average area: $A_g = \dfrac{A_g \cdot 26.7 + A_{gs} \cdot 3.3}{L}$ $A_g = 0.748$ m^2
- Girder seft-weigth: $w_g = A_g \cdot \gamma_c$ $w_g = 19$ kN/m
- Max. Moment at mid span: $M_g = \dfrac{w_g \cdot L_g^2}{8}$ $M_g = 2021$ kN-m.
- Moment of inertia: $I_g = 0.286$ m^4

  $c_{bg} = 0.910$ $c_{tg} = 0.890$ m
- Initial Jacking force :

  Cable 1-4: $P_{m1} = f_{pi} \cdot A_{ps1} \cdot 10^{-3}$ $P_{m1} = 1157$ kN.

  Cable 5-6: $P_{m5} = f_{pi} \cdot A_{ps5} \cdot 10^{-3}$ $P_{m5} = 1157$ kN.
- Total force : $P_m = 4 \cdot P_{m1} + 2 \cdot P_{m5}$ $P_m = 6944$ kN.
- Total strands area: $A_{ps} = 4 \cdot A_{ps1} + 2 \cdot A_{ps5}$ $A_{ps} = 5330$ mm^2

- At Midspan:

- AASHTO 9.16.2 : Prestress Losses:

- Total Losses (excluding friction losses) : $\Delta f_s = SH + ES + CR_c + CR_s$

- However, losses at transfering is due to friction (FR) and elastic shortening (ES):

- Friction Loss (FR) :

- Friction losses occur prior to anchoring, nevertheless, this estimation is done for design and checked during stressing operations.

- For post-tensioned member, AASHTO Art. 9.16.1 give friction Loss:

$$T_0 = T_x \cdot e^{(K \cdot L + \mu \cdot \alpha)}$$

IF $(KL+\mu\alpha) < 0.30$

$$T_0 = T_x \cdot (1 + K \cdot L + \mu \cdot \alpha)$$

- From Dwg. # DSPW-75 & -85 stated that:

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$K = .0049 \qquad m^{-1}$

$\mu = 0.25$

- Tendons profile is curve, therefore α will be:

$$\alpha = \frac{8 \cdot e_{mg}}{L} \qquad \alpha = 0.189$$

$$K \cdot L + \mu \cdot \alpha = 0.194 \qquad < 0.30$$

With : $T_0 = P_m \qquad T_0 = 6944$ kN.

$$T_0 = T_x \cdot (1 + K \cdot L + \mu \cdot \alpha)$$

$$T_x = \frac{T_0}{(1 + K \cdot L + \mu \cdot \alpha)} \qquad T_x = 5813.983 \text{ kN.}$$

- Loss due to friction : $FR_f = T_0 - T_x \qquad FR_f = 1129.851$ kN.

- Prestressing force loss : $FR = \frac{FR_f}{A_{ps}} \cdot 10^3 \qquad FR = 211.966$ MPa.

- Percent loss : $\%L_{FR} = \frac{FR}{f_{pi}} \cdot 100 \qquad \%L_{FR} = 16.271$ %

- **Elastic Shortening loss (ES) :**

$$ES = 0.5 \cdot \frac{E_{ps}}{E_{ci}} \cdot f_{cir}$$

- Where:

$$E_{ps} = 28 \cdot 10^6 \cdot 0.0069 \qquad E_{ps} = 1.9 \cdot 10^5 \text{ MPa}$$

$$E_{ci} = 0.043 \cdot \left(\gamma_c \cdot 10^2\right)^{1.5} \cdot \sqrt{f_{ci'}} \qquad E_{ci} = 3.0 \cdot 10^4 \text{ MPa}$$

$$n = \frac{E_{ps}}{E_{ci}} \qquad n = 6.35$$

$M_g = 2021$ kN-m

- Find concrete stress at c.g.s. due to prestressing force and dead load of girder
- AASHTO 9.16.2.1.2 said that initial stress in the tendon has been reduced by elastic shortening of the concrete and tendon friction, the reduced tendon stess can be taken as 0.63fs´ for stress relieved strand or 0.69fs´ for low relaxation strand. Therefore, tendons stresses are

$$f_{ps} = 0.69 \cdot f_{s'} \qquad f_{ps} = 1284.090 \text{ Mpa}$$

- Total prestressing forces:

$$P_m = \left(4 \cdot A_{ps1} \cdot f_{ps} + 2 \cdot A_{ps5} \cdot f_{ps}\right) \cdot 10^{-3} \qquad P_m = 6845 \text{ kN.}$$

(In Dwg. # DSPW-75 & -85 say Fj (force required at the jacking end after anchorage slip (if any))
Fj < or = 677 Tons. OK

$$f_{cir} = 10^{-3}\cdot\left(\frac{P_m}{A_g} + \frac{P_m\cdot e_{mg}^2}{I_g} - \frac{M_g\cdot e_{mg}}{I_g}\right)$$

$f_{cir} = 16.18$ MPa

$ES = 0.5\cdot\frac{E_{ps}}{E_{ci}}\cdot f_{cir}$ $\quad ES = 51.41$ MPa

- Percent loss : $\%L_{ES} = \frac{ES}{f_{pi}}\cdot 100$ $\quad \%L_{ES} = 3.9$ %

Therefore, losses at transfering stage : $\%L_{tf} = \%L_{ES}$ $\quad \%L_{tf} = 3.9$ %

Prestressing force at tranfering after losses equal to :

$f_{p\_tf} = \left(1 - \frac{\%L_{tf}}{100}\right)\cdot f_{ps}$ $\quad f_{p\_tf} = 1233$ MPa

Cable 1-4: $P_{m1} = f_{p\_tf}\cdot A_{ps1}\cdot 10^{-3}$ $\quad P_{m1} = 1096$ kN.

Cable 5-6: $P_{m5} = f_{p\_tf}\cdot A_{ps5}\cdot 10^{-3}$ $\quad P_{m5} = 1096$ kN.

- Total force : $P_m = 4\cdot P_{m1} + 2\cdot P_{m5}$ $\quad P_m = 6575$ kN.

Check stresses at top & bottom of girder at mid-span:

- General formular for checking stresses:

$$f = \frac{P}{A} + \frac{P\cdot e\cdot c}{I} - \frac{M\cdot c}{I}$$

where
f = stress (MPa)
P = effective prestressing force (kN)
e = cable eccentricity (m)
c = distance from C.G. to top or bottom fibers (or point in consideration), m
I = moment of inertia (m^4)
M = moment (kN-m)

- Sign convention: - is compression + is tension

Therefore, total stresses at transferring are:

At top of girder:

$$f_{t\_tf} = \left(\frac{-P_m}{A_g} + \frac{P_m\cdot e_{mg}\cdot c_{tg}}{I_g} - \frac{M_g\cdot c_{tg}}{I_g}\right)\cdot 10^{-3}$$

$f_{t\_tf} = -0.563$ MPa < $f_{tt} = 1.41$ MPa OK

At bottom of girder:

$$f_{b\_tf} = \left(\frac{-P_m}{A_g} - \frac{P_m\cdot e_{mg}\cdot c_{bg}}{I_g} + \frac{M_g\cdot c_{bg}}{I_g}\right)\cdot 10^{-3}$$

$f_{b\_tf} = 17.19$ MPa < $f_{ct} = -17.60$ MPa OK

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2.2.1.2 Cast In Situ Deck Slab:

Only dead loads from I-Girder, Diaphragm, Deck slab and Prestressing force to be considered:

Diaphragm:

- Reaction: $R := (1.35\cdot 0.6\cdot 2.2 + 0.2\cdot 0.2\cdot 2.2)\cdot\gamma_c$ $R = 46.750$ kN
- Moment: $M_D := R\cdot\frac{L_g}{2} - R\cdot 5$ $M_D = 453.475$ kN-m

Deck Slab:

- Slab seft-weigth: $w_s := h_s\cdot S\cdot\gamma_c$ $w_s = 11.00$ kN/m
- Max. Moment at mid span: $M_s := \frac{w_s\cdot L_g^2}{8}$ $M_s = 1188$ kN-m.

- Prestress Losses:

- Losses at this stage is due to shrinkage (SH) & creep (CR_c) of concrete and relaxation of steel (CR_s)

Total Losses: $\Delta f_s = SH + CR_c + CR_s$

- SHrinkage loss (SH) :

- Assume 80% mean relative humidity

$RH := 80$

- Post-tensiones Member:

$SH := 0.80\,(117.21 - 1.034\cdot RH)$ $SH = 27.59$ MPa

$\%L_{SH} := \frac{SH}{f_{p\_tf}}\cdot 100$ $\%L_{SH} = 2.2$ %

- CReep of concrete loss (CRc) :

$CR_c = 12\cdot f_{cir} - 7\cdot f_{cds}$

- Find concrete stress at c.g.s. due to all dead loads except the dead load present at the time the prestressing force is applied.

$\Delta M_{dl} := M_s + M_D$ $\Delta M_{dl} = 1642$ kN-m.

$f_{cds} := \frac{\Delta M_{dl}\cdot e_{mg}\cdot 10^{-3}}{I_g}$ $f_{cds} = 4.07$ MPa

$CR_c := 12\cdot f_{cir} - 7\cdot f_{cds}$ $CR_c = 165.70$ MPa

- Percent loss : $\%L_{CRc} := \frac{CR_c}{f_{p\_tf}}\cdot 100$ $\%L_{CRc} = 13.434$ %

- Relaxation of Prestressing Steel (CRs) :

- For low-relaxation strand:

$CR_s := 34.48 - 0.07\cdot FR - 0.1\cdot ES - 0.05\cdot(SH + CR_c)$

$CR_s = 4.837$ MPa

- Percent loss : $\%L_{CRs} = \frac{CR_s}{f_{p\_tf}} \cdot 100$ $\%L_{CRs} = 0.392$ %

- Total Prestressing Loss:

$\Delta f_s = SH + CR_c + CR_s$ $\Delta f_s = 198$ MPa

- Loss in percentage:

$\%L_{SCR} = \frac{\Delta f_s}{f_{p\_tf}} \cdot 100$ $\%L_{SCR} = 16.1$ %

- Total loss since transfering to complete state : $\%L_{tot} = \%L_{tf} + \%L_{SCR}$ $\%L_{tot} = 20.0$ %

- Total prestressing losses : $f_{p\_tot} = \frac{\%L_{tot}}{100} \cdot f_{pi}$ $f_{p\_tot} = 261$ Mpa

- Limiting Maximum Loss (ACI-ASCE Committee), Ref." Design of Prestressd Concrete Structures", by T.Y. Lin and NED H. Burns, 3rd Edit., SI Version

- Table 4-8, page 117, for Normal concrete & Low-relaxation strand : $Loss_{max} = 275$ Mpa

Normal concrete Stress-relieved strand : $Loss_{max} = 345$ Mpa

- Assuming losses at this stage due to shrinkage, creep, and relaxation would be haft of total losses due to these causes :

$\%L_{cis} = \frac{\%L_{SCR}}{2}$ $\%L_{cis} = 8.0$ %

- Total prestressing force of each cable at mid-span :

Cable 1-4: $P_{m1} = P_{m1} \cdot \left(1 - \frac{\%L_{cis}}{100}\right)$ $P_{m1} = 1008$ kN.

Cable 5-6: $P_{m5} = P_{m5} \cdot \left(1 - \frac{\%L_{cis}}{100}\right)$ $P_{m5} = 1008$ kN.

- Total force : $P_m = 4 \cdot P_{m1} + 2 \cdot P_{m5}$ $P_m = 6046$ kN.

Check stresses at top & bottom of girder at mid-span:

At top of girder: $$f_{t\_cs} = \left| \frac{-P_m}{A_g} + \frac{P_m \cdot e_{mg} \cdot c_{tg}}{I_g} - \frac{(M_g + M_s + M_D) \cdot c_{tg}}{I_g} \right| \cdot 10^{-3}$$

$f_{t\_cs} = -6.12$ MPa < $f_{cw} = -13.92$ MPa OK

At bottom of girder: $$f_{b\_cs} = \left| \frac{-P_m}{A_g} - \frac{P_m \cdot e_{mg} \cdot c_{bg}}{I_g} + \frac{(M_g + M_s + M_D) \cdot c_{bg}}{I_g} \right| \cdot 10^{-3}$$

$f_{b\_cs} = -10.08$ MPa < $f_{tw} = 2.94$ MPa OK

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2.2.1.3 At Working: I-Girder, Deck Slab, Wearing surface & Live loads

- Assuming losses at this stage due to shrinkage, creep, and relaxation would be haft of total losses due to these causes :

$$\%L_{wk} := \frac{\%L_{SCR}}{2} \qquad \%L_{wk} = 8.0 \quad \%$$

- Total prestressing force of each cable at mid-span :

Cable 1-4: $$P_{m1} := P_{m1'} \cdot \left(1 - \frac{\%L_{wk}}{100}\right) \qquad P_{m1} = 927 \quad \text{kN.}$$

Cable 5-6: $$P_{m5} := P_{m5'} \cdot \left(1 - \frac{\%L_{wk}}{100}\right) \qquad P_{m5} = 927 \quad \text{kN.}$$

- Total force : $$P_m := 4 \cdot P_{m1} + 2 \cdot P_{m5} \qquad P_m = 5561 \quad \text{kN.}$$

Check stresses at top & bottom of girder at mid-span :

At top of girder:

$$f_{t\_wk} := \left[\frac{-P_m}{A_g} + \frac{P_m \cdot e_{mg} \cdot c_{tg}}{I_g} - \frac{(M_g + M_s + M_D) \cdot c_{tg}}{I_g}\right] \cdot 10^{-3}$$

$f_{t\_wk} = -6.55$ MPa < $f_{cw} = -13.92$ MPa OK

At bottom of girder:

$$f_{b\_wk} := \left[\frac{-P_m}{A_g} - \frac{P_m \cdot e_{mg} \cdot c_{bg}}{I_g} + \frac{(M_g + M_s + M_D) \cdot c_{bg}}{I_g}\right] \cdot 10^{-3}$$

$f_{b\_wk} = -8.34$ MPa < $f_{tw} = 2.94$ MPa OK

- Loads Effect

Superimposed Dead Loads (SDL):

- Wearing Surface: $w_w := \gamma_{aspt} \cdot h_w \cdot S$ $\qquad w_w = 3.96$ kN/m.

$$M_w := \frac{w_w \cdot L_g^2}{8} \qquad M_w = 428 \quad \text{kN-m}$$

- Parapet + sidewalk: one side

$$A_p := 0.3 \cdot 0.55 + \left(\frac{0.3 + 0.36}{2}\right) \cdot 1.5 \qquad A_p = 0.660 \quad m^2$$

- Distrib. loads for each girder: $$w_p := \frac{A_p \cdot \gamma_c \cdot 2}{6} \qquad w_p = 5.500 \quad \text{kN/m}$$

$$M_p := \frac{w_p \cdot L_g^2}{8} \qquad M_p = 594 \quad \text{kN-m}$$

- Moment due to SDL: $M_{SDL} := M_w + M_p$ $\qquad M_{SDL} = 1022$ kN-m

---

Live Loads:

- Net span length of girder: $L_g = 29.40$ m

$L := L_g \cdot 3.28$ $\quad L = 96.432$ ft.

- Impact factor: $I := \dfrac{50}{(L + 125)}$

$I = 0.226$

Truck Load: HS20-44+50%

see Fig. 2.3(a) for truck loading configuration:

$L_2 := 4.27$ m. $\quad L_3 := 4.27$ m. $\quad L_4 := 4.27$ m.

$L_1 := \dfrac{L_g}{2} + L_4 - L_2 - L_3$ $\quad L_1 = 10.430$ m.

$P_1 := 36 \cdot 1.5$ $\quad P_1 = 54$ kN. $\quad P_2 := 144 \cdot 1.5$ $\quad P_2 = 216$ kN.

$$R_B := \frac{P_1 \cdot L_1 + P_2 \cdot (L_1 + L_2) + P_2 \cdot (L_1 + L_2 + L_3)}{L_g} \qquad R_B = 266.53 \text{ kN.}$$

$R_A := (P_1 + 2 \cdot P_2) - R_B$ $\quad R_A = 219.47$ kN.

$M_{mid} := R_B \cdot \dfrac{L_g}{2} - P_2 \cdot L_4$ $\quad M_{mid} = 2996$ kN-m

$M_{ti} := M_{mid} \cdot (1 + I)$ $\quad M_{ti} = 3672$ kN-m

Lane Loads:

see Fig. 2.3(b) for lane loading configuration:

$P_{lm} := 80 \cdot 1.5$ $\quad P_{lm} = 120$ kN $\quad w_1 := 9.33 \cdot 1.5$ $\quad w_1 = 14.00$ kN/m

$M_{mid} := \dfrac{w_1 \cdot L_g^2}{8} + \dfrac{P_{lm} \cdot L_g}{4}$ $\quad M_{mid} = 2394$ kN-m

$M_{li} := M_{mid} \cdot (1 + I)$ $\quad M_{li} = 2935$ kN-m

$M_{LI} := \left| M_{ti} \;\; M_{li} \right|$ $\quad M_{LI} = [\, 3672 \;\; 2935 \,]$

$M_{LI} := \max(M_{LI})$ $\quad M_{LI} = 3672$ kN-m.

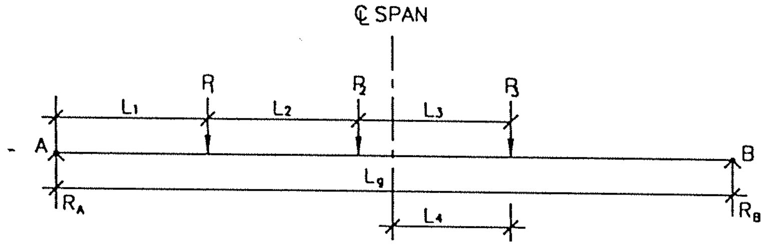
Therefore, for live loads, truck loading is critical with: $\quad M_{LI} = 3672$ kN-m/one truck

STANDARD HS20-44 TRUCK.

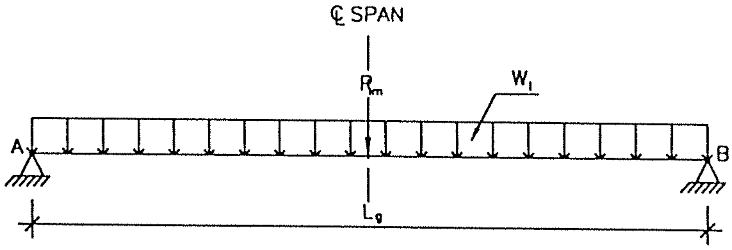
$P_1 = 36$ kN

$P_2 = P_3 = 144$ kN

Traffic Loadings



(a) Truck Loading



$W_l = 9.33$ kN/m

$P_m = 80$ kN for moment

$= 116$ kN for shear

(b) Lane Loading

Fig 2.3 : HS20-44+50%, Live Loads to maximize moment

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Interior Girder:

- Distribution factor of wheel loads: $DF := \frac{S \cdot 3.28}{5.5}$ $DF = 1.31$
- Distribution of wheel load moment on an interior girder is:

$M_{WLI} := \frac{DF \cdot M_{LI}}{2}$ $M_{WLI} = 2409$ kN-m.

- For load combination group I: $G1 = \gamma\left(\beta_D \cdot D + \beta_L \cdot (L + I)\right)$
- For service loads: we have

$\gamma := 1.0$ $\beta_D := 1.0$ $\beta_L := 1.0$

$M_{inc} := \gamma \cdot \left(\beta_D \cdot M_{SDL} + \beta_L \cdot M_{WLI}\right)$

$M_{inc} = 3431$ kN-m.

Check stresses:

- Increment stress due to superimposed dead loads & live loads is
- At top of deck slab: $f_{t\_ds} := \frac{-M_{inc} \cdot c_{tc} \cdot 10^{-3}}{I_c}$ $f_{t\_ds} = -4.78$ MPa.
- At top of girder: $f_{t\_gd} := \frac{-M_{inc} \cdot (c_{tc} - h_s) \cdot 10^{-3}}{I_c}$ $f_{t\_gd} = -3.48$ MPa.
- At bottom of girder: $f_{b\_gd} := \frac{M_{inc} \cdot c_{bc} \cdot 10^{-3}}{I_c}$ $f_{b\_gd} = 8.25$ MPa.

Total stresses of girder and deck slab, in services condition:

- Girder:

$f_{top} := f_{t\_wk} + f_{t\_gd}$ $f_{top} = -10.02$ MPa < $f_{cw} = -13.92$ MPa OK

$f_{bot} := f_{b\_wk} + f_{b\_gd}$ $f_{bot} = -0.09$ MPa < $f_{tw} = 2.94$ MPa OK

- Deck Slab:

$f_{top} := n_c \cdot f_{t\_ds}$ $f_{top} = -4.14$ MPa < $f_{c\_s} = -10.44$ MPa OK

$f_{bot} := n_c \cdot f_{t\_gd}$ $f_{bot} = -3.01$ MPa < $f_{c\_s} = -10.44$ MPa OK

2.2.2 Ultimate Limit State (ULS)

2.2.2.1 Moment Capacity:

- Compression Reinforcement:

$\phi' := 20$ mm. $A_{\phi'} := \frac{\pi \cdot \phi'^2}{4}$ $A_{\phi'} = 314.159$ mm^2

$N' := 8$ $A_{s'} := N' \cdot A_{\phi'}$ $A_{s'} = 2513.274$ mm^2

$cgs_{top} := h_s + 0.07$ $cgs_{top} = 0.270$ m.

$d' := h_g + h_s - cgs_{top}$ $d' = 1.730$ m.

$f_{y'} := f_{y\_DB}$ $f_{y'} = 410$ Mpa

- Tension Reinforcement:

$\phi t := 25$ mm. $A_{\phi t} := \frac{\pi \cdot \phi t^2}{4}$ $A_{\phi t} = 490.874$ mm^2

$N := 4$ $A_s := N \cdot A_{\phi t}$ $A_s = 1963.495$ mm^2

$cgs_{bot} := 0.05 + 0.014 + \frac{\phi t}{1000}$ $cgs_{bot} = 0.089$ m.

$d_t := h_s + h_g - cgs_{bot}$ $d_t = 1.911$ m.

$f_{sy} := f_{y\_DB}$ $f_{sy} = 410$ MPa.

- Effective depth of prestressing steel from top surface of deck slab:

- At Midspan:

Cable 1-3: $d_{p1} := h_g + h_s - y_{m1}$ $d_{p1} = 1.900$ m.

Cable 4: $d_{p4} := h_g + h_s - y_{m4}$ $d_{p4} = 1.800$ m.

Cable 5: $d_{p5} := h_g + h_s - y_{m5}$ $d_{p5} = 1.700$ m.

Cable 6: $d_{p6} := h_g + h_s - y_{m6}$ $d_{p6} = 1.600$ m.

- Total area of prestressing steel:

$A_{ps} := 4 \cdot A_{ps1} + 2 \cdot A_{ps5}$ $A_{ps} = 5330.340$ mm^2

- Effective depth of group of cable:

$$d := \frac{(3 \cdot d_{p1} + d_{p4}) \cdot A_{ps1} + (d_{p5} + d_{p6}) \cdot A_{ps5}}{A_{ps}}$$ $d = 1.800$ m

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- Breaking strength : $f_{s'} = 1861$ MPa.

- AASHTO Art. 9.17.4, Steel Stress :

$f_{c'} := f_{c'\_g}$ $\quad f_{c'} = 34.800$ MPa.

$b := b_c$ $\quad b = 2.200$ m.

- Bonded members with non-prestressed tension reinforcement included:

$$f_{su} = f_{s'}\cdot\left[1-\left(\frac{\gamma}{\beta_1}\right)\cdot\left[\rho_p\cdot\frac{f_{s'}}{f_{c'}}+\frac{d_t}{d}\cdot\left(\rho\cdot\frac{f_{sy}}{f_{c'}}\right)\right]\right]$$

$\gamma := 0.28$ for low-relaxation steel

$\beta_1 := 0.85 - 0.05\cdot\left(\frac{f_{c'}-28}{7}\right)$ when $f_{c'} > 28$ MPa.

$\beta_1 = 0.801$

$\rho := \frac{A_s\cdot 10^{-6}}{b_c\cdot d_t}$ $\quad \rho = 0.00047$

$\rho' := \frac{A_{s'}\cdot 10^{-6}}{b_c\cdot d'}$ $\quad \rho' = 0.00066$

$\rho_p := \frac{A_{ps}\cdot 10^{-6}}{b_c\cdot d}$ $\quad \rho_p = 0.00135$

$$f_{su} := f_{s'}\cdot\left[1-\left(\frac{\gamma}{\beta_1}\right)\cdot\left[\rho_p\cdot\frac{f_{s'}}{f_{c'}}+\frac{d_t}{d}\cdot\left(\rho\cdot\frac{f_{sy}}{f_{c'}}\right)\right]\right]$$

$f_{su} = 1810$ Mpa.

- AASHTO Art. 9.17 FLEXURAL STRENGTH :
Check if section is Rectangular or Flanged Sections:

$$a := \frac{A_{ps}\cdot f_{su} + A_s\cdot f_{sy}}{0.85\cdot f_{c'}\cdot b\cdot 10^4}$$

$a = 16.07$ cm. < $h_s\cdot 100 = 20.00$ cm. **Rectangular section**

- AASHTO Art. 9.18.1 Maximum Prestressing Steel :
Check the reinforcement index :

$$RF_{index} := \left(\frac{\rho\cdot f_{sy}}{f_{c'}}\right)\cdot\frac{d_t}{d} + \left(\frac{\rho_p\cdot f_{su}}{f_{c'}}\right) - \left(\frac{\rho'\cdot f_{y'}}{f_{c'}}\right)$$ $\quad RF_{index} = 0.068$

$RF_{index} = 0.068$ < $0.36\cdot\beta_1 = 0.289$ OK

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- AASHTO Art. 9.17.2 FLEXURAL STRENGTH for Rectangular Section, Eq. 9-13a :

$$M_n := \left[A_{ps}\cdot f_{su}\cdot d\cdot\left[1 - 0.6\left(\frac{\rho_p\cdot f_{su}}{f_{c'}} + \frac{d_t}{d}\cdot\frac{\rho\cdot f_{sy}}{f_{c'}}\right)\right] + A_s\cdot f_{sy}\cdot d_t\cdot\left[1 - 0.6\cdot\left(\frac{d}{d_t}\cdot\frac{\rho_p\cdot f_{su}}{f_{c'}} + \frac{\rho\cdot f_{sy}}{f_{c'}}\right)\right]\right]\cdot 10^{-3}$$

$M_n = 18052$ kN-m.

$\phi := 1.00$ for precast prestressed concrete members (AASHTO 9.14)

$\phi M_n := \phi\cdot M_n$ $\phi M_n = 18052$ kN-m.

Load Effects:

$M_g = 2021$ kN-m. $M_{SDL} = 1022$ kN-m.

$M_D = 453.475$ kN-m $M_{WLI} = 2409$ kN-m.

$M_s = 1188$ kN-m.

- For load combination group I: $G1 = \gamma\left(\beta_D\cdot D + \beta_L(L + I)\right)$

- For ultimate state : we have

$\gamma := 1.3$ $\beta_D := 1.0$ $\beta_L := 1.67$

$M_u := \gamma\cdot\left[\beta_D\cdot\left(M_g + M_{SDL} + M_s + M_D\right) + \beta_L\cdot M_{WLI}\right]$ $M_u = 11321$ kN-m.

Since $\phi M_n = 18052$ kN-m. > $M_u = 11321$ kN-m. ***Cross section is OK at Ultimate***

AASHTO Art. 9.18.2 Minimum Steel :

Art. 9.18.2 : Check cracking moment:

$$M_{cr} := \left(f_r + f_{pe}\right)\cdot S_{bc}\cdot 10^3 - M_{d\_c}\cdot\left(\frac{S_{bc}}{S_{bg}} - 1\right)$$

$M_{d\_c} := M_g + M_s + M_w + M_D$ $M_{d\_c} = 4091$ kN-m.

$P_m := 4\cdot P_{m1} + 2\cdot P_{m5}$ $P_m = 5561$ kN.

$f_{pe} := \left(\frac{P_m}{A_c} + \frac{P_m\cdot e_{mc}\cdot y_c}{I_c}\right)\cdot 10^{-3}$ $f_{pe} = 19.499$ MPa

$f_r := 0.623\sqrt{f_{c'\_g}}$ $f_r = 3.68$ MPa.

$$M_{cr} := \left(f_r + f_{pe}\right)\cdot S_{bc}\cdot 10^3 - M_{d\_c}\cdot\left(\frac{S_{bc}}{S_{bg}} - 1\right)$$

$M_{cr} = 8326$ kN-m.

Since $\phi M_n = 18052$ kN-m. > $1.2\ M_{cr} = 9991$ kN-m.

Cross section is OK at Ultimate compared with 1.2 times Cracking moment

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2.2.2.2 Shear Capacity:

Maximum shear: see Fig. 2.4 for loading configuration

Dead Loads: $L_g := 29.40$ m. $h_g := 1.80$ m

$w_g := 17$ kN/m $w_s := 11$ kN/m $w_w := 9.46$ kN/m

- Girder : $R_g := \frac{w_g \cdot L_g}{2}$ $R_g = 250$ kN.

- Deck slab: $R_s := \frac{w_s \cdot L_g}{2}$ $R_s = 162$ kN.

- Superimposed DL: $R_w := \frac{w_w \cdot L_g}{2}$ $R_w = 139$ kN.

$R_{Ad} := R_g + R_s + R_w$ $R_{Ad} = 551$ kN.

$R_{Bd} := R_{Ad}$ $R_{Bd} = 551$ kN.

Live Load:

1. Axle Load:

$P_1 := 54$ kN. $P_2 := 216$ m.

$L_1 := 4.27$ m. $L_2 := 4.27$ m.

$$R_{At} := \frac{P_2 \cdot L_g + P_2 \cdot (L_g - L_1) + P_1 \cdot (L_g - L_1 - L_2)}{L_g}$$

$R_{At} = 439$ kN.

$R_{Bt} := P_1 + 2 \cdot P_2 - R_{At}$ $R_{Bt} = 47$ kN.

2. Lane Load:

$P_{ls} := 116 \cdot 1.5$ $P_{ls} = 174$ kN. $w_l := 14$ kN. $I := 0.226$

$R_{Al} := \frac{w_l \cdot L_g}{2} + P_{ls}$ $R_{Al} = 380$ kN.

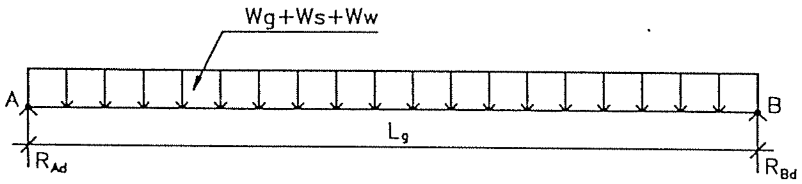
$R_{Bl} := R_{Al}$ $R_{Bl} = 380$ kN.

$R_L := [R_{At} \;\; R_{Al}]$ $R_L = [\, 439 \;\; 380 \,]$

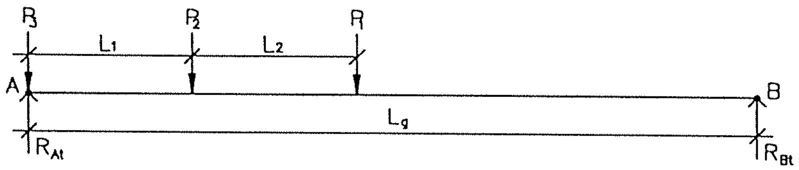
$R_L := \max(R_L)$ $R_L = 439$ kN.

$R_{LI} := R_L \cdot (1 + I)$ $R_{LI} = 538$ kN.

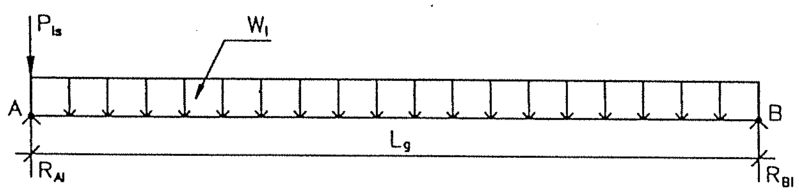
Therefore, for live loads, truck loading is critical with: $R_{LI} = 538$ kN/one truck



(a) Dead load of girder + deck slab + wearing surface + parapet & sidewalk

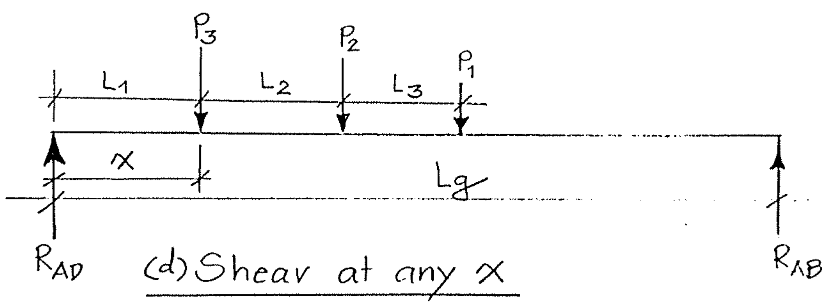


(b) Truck Loading



(c) Lane Loading

Fig 2.4: Dead Loads & Live Loads to maximize shear at support



(d) Shear at any x

Interior Girder: $S := 2.20$ m.

- Distribution factor of wheel loads: $DF := \frac{S \cdot 3.28}{5.5}$ $DF = 1.31$

- Distribution of wheel load shear on an interior girder is:

$$R_{WLI} := \frac{DF \cdot R_{LI}}{2} \qquad R_{WLI} = 353.02 \text{ kN.}$$

- For load combination group I: $G1 := \gamma(\beta_D \cdot D + \beta_L(L + I))$
- For ultimate limit state: we have

$\gamma := 1.3$ $\beta_D := 1.0$ $\beta_L := 1.67$

$$R_u := \gamma \cdot (\beta_D \cdot R_{Ad} + \beta_L \cdot R_{WLI})$$

$R_u = 1482$ kN.

AASHTO Art. 9.20.2 Shear-Strength Provided by Concrete:

$f_{c'} := 34.8$ Mpa. $f_{sy} := 410$ MPa

$$V_{cw} := \left(0.29 \cdot \sqrt{f_{c'}} + 0.3 \cdot f_{pc}\right) \cdot b' \cdot d \cdot 10^3 + V_p \qquad \text{kN.}$$

$$V_{ci} := 49.8 \cdot \sqrt{f_{c'}} \cdot b \cdot d + V_d + \frac{V_i \cdot M_{cr}}{M_{max}} \quad > \text{or} = \quad 218.8 \cdot \sqrt{f_{c'}} \cdot b' \cdot d$$

The lesser of the values Vci or Vcw will govern the design:

$$V_c := \min\left[V_{cw} \;\; V_{ci}\right]$$

Assume prestressing profile is parabolar :

Length of girder : $L := L_g$ $L = 29.40$ m.

Distance from C.G. of girder alone to extreme top fiber : $c_{tg} := 0.89$ m.

Distance from C.G. of girder alone to extreme bottom fiber : $c_{bg} := 0.91$ m.

Eccentricity of group of cables at mid-span : $e_{mg} := 0.71$ m.

Eccentricity of group of cables at the end : $e_{sg} := 0.035$ m.

Heigth of parabolar at mid-span is : $h_{para} := e_{mg} - e_{sg}$ $h_{para} = 0.67$ m.

Parabolar Equation: $y := 4 \cdot h_{para} \cdot \left[\frac{x}{L} - \left(\frac{x}{L}\right)^2\right]$

Slope Equation: $y' := \frac{dy}{dx}$ $y' := \frac{4 \cdot h_{para}}{L} \cdot \left(1 - 2 \cdot \frac{x}{L}\right)$

Assume effective prestress is linearly decreased from the end to mid-span

$f_{pi} := 1303$ MPa. $\%L_{tot} := 24$ % $A_{ps} := 5725$ mm^2 $P_m := 5218$ kN.

Effective prestress at the end : $P_s := f_{pi} \cdot \left(1 - \frac{\%L_{tot}}{100}\right) \cdot A_{ps} \cdot 10^{-3}$ $P_s = 5669$ kN.

Effective prestress at mid-span : $P_e := P_m$ $P_m = 5218$ kN.

Therefore, at any x distance: $P_e := P_s - \left(\frac{P_s - P_m}{L}\right) \cdot 2 \cdot x$

Total heigth of girder is: $h := h_g$ $h = 1.80$ m.

****

At section : $x := \frac{h}{2}$ $x = 0.90$ m

****

- Crossection will be the end cross section with :

$b' := 0.70$ m $A_{gs} := 1.32$ m^2 $I_{gs} := 0.38$ m^4 $c_{ts} := 0.865$ m $c_{bs} := 0.935$ m

Find Vci:

$$V_d := \frac{\frac{L_g}{2} - x}{\frac{L_g}{2}} \cdot R_{Ad} \qquad V_d = 516.95 \quad \text{kN}$$

$$y := 4 \cdot h_{para} \cdot \left[\frac{x}{L} - \left(\frac{x}{L} \right)^2 \right] \qquad y = 0.08 \quad \text{m.}$$

- Distance from extreme compression fiber :

$$d := c_{ts} + e_{sg} + y \qquad d = 0.98 \quad \text{m.}$$

$$d := (d \quad 0.8 \cdot h) \qquad d = [\, 0.98 \quad 1.44 \,]$$

$$d := \max(d) \qquad d = 1.44 \quad \text{m.}$$

- Eccentricity of cables from C.G. of girder :

$$e_x := e_{sg} + y \qquad e_x = 0.12 \quad \text{m.}$$

- Calculate Vi & Mmax : Truck load is critical

$L_1 := x$ $L_2 := 4.27$ m $L_3 := 4.27$ m

$$V_x := \frac{(L_g - L_1) \cdot P_2 + (L_g - L_2 - L_1) \cdot P_2 + (L_g - L_3 - L_2 - L_1) \cdot P_1}{L_g}$$

$V_x = 424.07$ kN $V_{x\_i} := (1 + I) \cdot V_x$ $V_{x\_i} = 519.90$ kN.

$$V_i := \frac{V_{x\_i}}{2} \cdot DF \qquad V_i = 341 \quad \text{kN.}$$

$$M_{max} := V_i \cdot x \qquad M_{max} = 306.95 \quad \text{kN-m}$$

$$P_e := P_s - \left(\frac{P_s - P_m}{L} \right) \cdot 2 \cdot x \qquad P_e = 5641.72 \quad \text{kN.}$$

$$f_{pe} := \frac{P_e \cdot 10^{-3}}{A_{gs}} + \frac{P_e \cdot e_x \cdot c_{bs} \cdot 10^{-3}}{I_{gs}} \qquad f_{pe} = 5.87 \quad \text{Mpa.}$$

$$w_d := w_g + w_s + w_w \qquad w_d = 37 \quad \text{kN/m}$$

$$M_d := R_{Ad} \cdot x - \frac{w_d \cdot x^2}{2} \qquad M_d = 480.42 \quad \text{kN-m}$$

$f_d := \dfrac{M_d \cdot c_{bs} \cdot 10^{-3}}{I_{gs}}$ $f_d = 1.18$ MPa.

$Y_t := c_{bs}$ $Y_t = 0.94$ m.

$M_{cr} := \dfrac{I_{gs}}{Y_t} \cdot \left(0.498 \cdot \sqrt{f_{c'}} + f_{pe} - f_d\right) \cdot 10^3$ $M_{cr} = 3100$ kN-m.

$V_{ci} := 49.8 \cdot \sqrt{f_{c'}} \cdot b' \cdot d + V_d + \dfrac{V_i \cdot M_{cr}}{M_{max}}$ > or = $218.76 \cdot \sqrt{f_{c'}} \cdot b' \cdot d = 1301$ kN.

$V_{ci} = 4258$ kN.

Find Vcw:

$f_{pc} := \dfrac{P_e}{A_{gs}} \cdot 10^{-3}$ $f_{pc} = 4.27$ MPa.

$y' := \dfrac{4 \cdot h_{para}}{L} \cdot \left(1 - 2 \cdot \dfrac{x}{L}\right)$ $y' = 0.09$

$V_p := P_e \cdot y'$ $V_p = 486$ kN.

$V_{cw} := \left(0.29 \cdot \sqrt{f_{c'}} + 0.3 \cdot f_{pc}\right) \cdot b' \cdot d \cdot 10^3 + V_p$

$V_{cw} = 3503$ kN.

Find Shear capacity of concrete:

$V_c := \left[V_{ci} \;\; V_{cw}\right]$ $V_c := \min(V_c)$ $V_c = 3503$ kN.

- For ultimate limit state: we have

$\gamma := 1.3$ $\beta_D := 1.0$ $\beta_L := 1.67$

$V_u := \gamma \cdot \left(\beta_D \cdot V_d + \beta_L \cdot V_i\right)$ $V_u = 1412$ kN.

- **AASHTO 9.14** for shear :

$\phi := 0.90$

$\phi V_c := \phi \cdot V_c$ $\phi V_c = 3153$ kN.   > $V_u = 1412$ kN.

- Provide minimum shear reinforcement :

- **AASHTO Art. 9.20.3.3**, the minimum area of web reinforcement shall be :

$A_{vmin} := \dfrac{0.345 \cdot b' \cdot s}{f_{sy}}$ $b' = 0.70$

$A_{vmin\%s} := \dfrac{0.345 \cdot b' \cdot 10^6}{f_{sy}}$ $A_{vmin\%s} = 589.02$ $\dfrac{mm^2}{m}$

- Choose ***DB14 @ 0.075 as web reinforcement*** - 2 legs (U-shaped) stirrups :

$\phi_b := 14$ mm. $\quad A_s := \frac{\pi \cdot \phi_b^2}{4}$ $\quad A_s = 153.94$ mm^2

$A_v := 2 \cdot A_s$ $\quad A_v = 307.88$ mm^2 $\quad s_{prov} := 0.075$ m

$A_{v\_prov} := \frac{A_v}{s_{prov}}$ $\quad A_{v\_prov} = 4105.01$ $\frac{mm^2}{m}$ OK.

- **AASHTO Art. 9.20.3.2,** check maximum allowable spacing :

$s_{max1} := 0.75 \cdot h$ $\quad s_{max1} = 1.35$ m

$s_{max2} := 24 \cdot \frac{25.4}{1000}$ $\quad s_{max2} = 0.61$ m

$s_{prov} = 0.075$ m. < $s_{max2} = 0.61$ m. OK.

PROJECT : Design Bridges in Jordan

SHEET NO.......OF.........

SUBJECT : Design Post-tensioned I-Girder, Span 30 m.

JOB NO......1045...........

DRAWING NO...............

DATE......MAY....1999...

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At section : $x := 2.00$ m

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- Crossection will be the typical cross section with :

$b' \ 0.20$ m $\quad A_g \ 0.68$ m^2 $\quad I_g \ 0.29$ m^4 $\quad c_{tg} \ 0.89$ m $\quad c_{bg} \ 0.91$ m

Find Vci:

$$V_d := \frac{\frac{L_g}{2} - x}{\frac{L_g}{2}} \cdot R_{Ad} \qquad V_d = 475.74 \quad \text{kN}$$

$$y := 4 \cdot h_{para} \cdot \left[\frac{x}{L} - \left(\frac{x}{L}\right)^2 \right] \qquad y = 0.17 \quad \text{m.}$$

- Distance from extreme compression fiber :

$$d := c_{tg} + e_{sg} + y \qquad d = 1.10 \quad \text{m.}$$

$$d := (d \quad 0.8 \cdot h) \qquad d = [\ 1.10 \quad 1.44\ ]$$

$$d := \max(d) \qquad d = 1.44 \quad \text{m.}$$

- Eccentricity of cables from C.G. of girder :

$$e_x := e_{sg} + y \qquad e_x = 0.21 \quad \text{m.}$$

- Calculate Vi & Mmax : Truck load is critical

$L_1 := x \qquad L_2 := 4.27$ m $\qquad L_3 := 4.27$ m

$$V_x := \frac{(L_g - L_1) \cdot P_2 + (L_g - L_2 - L_1) \cdot P_2 + (L_g - L_3 - L_2 - L_1) \cdot P_1}{L_g}$$

$V_x = 405.88$ kN $\qquad V_{x\_i} := (1 + I) \cdot V_x \qquad V_{x\_i} = 497.61$ kN.

$$V_i := \frac{V_{x\_i}}{2} \cdot DF \qquad V_i = 326 \quad \text{kN.}$$

$$M_{max} := V_i \cdot x \qquad M_{max} = 652.87 \quad \text{kN-m}$$

$$P_e := P_s - \left(\frac{P_s - P_m}{L}\right) \cdot 2 \cdot x \qquad P_e = 5607.94 \quad \text{kN.}$$

$$f_{pe} := \frac{P_e \cdot 10^{-3}}{A_g} + \frac{P_e \cdot e_x \cdot c_{bg} \cdot 10^{-3}}{I_g} \qquad f_{pe} = 11.88 \quad \text{Mpa.}$$

$$w_d := w_g + w_s + w_w \qquad w_d = 37 \quad \text{kN/m}$$

$$M_d := R_{Ad} \cdot x - \frac{w_d \cdot x^2}{2} \qquad M_d = 1026.40 \quad \text{kN-m}$$

$f_d := \frac{M_d \cdot c_{bg} \cdot 10^{-3}}{I_g}$ $\quad f_d = 3.22$ MPa.

$Y_t := c_{bg}$ $\quad Y_t = 0.91$ m.

$M_{cr} := \frac{I_g}{Y_t} \cdot \left(0.498 \cdot \sqrt{f_{c'}} + f_{pe} - f_d\right) \cdot 10^3$ $\quad M_{cr} = 3694$ kN-m.

$V_{ci} := 49.8 \cdot \sqrt{f_{c'}} \cdot b' \cdot d + V_d + \frac{V_i \cdot M_{cr}}{M_{max}}$ > or = $218.76 \cdot \sqrt{f_{c'}} \cdot b' \cdot d = 372$ kN.

$V_{ci} = 2407$ kN.

Find Vcw:

$f_{pc} := \frac{P_e}{A_g} \cdot 10^{-3}$ $\quad f_{pc} = 8.25$ MPa.

$y' := \frac{4 \cdot h_{para}}{L} \cdot \left(1 - 2 \cdot \frac{x}{L}\right)$ $\quad y' = 0.08$

$V_p := P_e \cdot y'$ $\quad V_p = 445$ kN.

$V_{cw} := \left(0.29 \cdot \sqrt{f_{c'}} + 0.3 \cdot f_{pc}\right) \cdot b' \cdot d \cdot 10^3 + V_p$

$V_{cw} = 1650$ kN.

Find Shear capacity of concrete:

$V_c := \left[V_{ci} \;\; V_{cw}\right]$ $\quad V_c := \min(V_c)$ $\quad V_c = 1650$ kN.

$V_u := \gamma \cdot \left(\beta_D \cdot V_d + \beta_L \cdot V_i\right)$ $\quad V_u = 1327$ kN.

- **AASHTO 9.14** for shear :

$\phi V_c := \phi \cdot V_c$ $\quad \phi V_c = 1485$ kN. > $V_u = 1327$ kN.

- Since $V_u = 1327$ kN > $\frac{\phi V_c}{2} = 743$ kN - Need minimum shear reinforcement :

$A_{vmin} := \frac{0.345 \cdot b' \cdot s}{f_{sy}}$ $\quad b' = 0.20$

$A_{vmin\%s} := \frac{0.345 \cdot b' \, 10^6}{f_{sy}}$ $\quad A_{vmin\%s} = 168.29 \; \frac{mm^2}{m}$

- Choose ***DB14 @ 0.15 as web reinforcement*** - 2 legs (U-shaped) stirrups : $s_{prov} := 0.15$ m

$A_{v\_prov} := \frac{A_v}{s_{prov}}$ $\quad A_{v\_prov} = 2052.51 \; \frac{mm^2}{m}$ OK.

PROJECT : Design Bridges in Jordan

SHEET NO.......OF.........

SUBJECT : Design Post-tensioned I-Girder, Span 30 m.

JOB NO......1045...........

DRAWING NO...............

DATE......MAY....1999...

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At section : $x := 7.00$ m

****

Find Vci:

$V_d := \dfrac{\dfrac{L_g}{2} - x}{\dfrac{L_g}{2}} \cdot R_{Ad}$ $\quad V_d = 288.44$ kN

$y := 4 \cdot h_{para} \cdot \left[\dfrac{x}{L} - \left(\dfrac{x}{L}\right)^2\right]$ $\quad y = 0.49$ m.

- Distance from extreme compression fiber :

$d := c_{tg} + e_{sg} + y$ $\quad d = 1.41$ m.

$d := (d \quad 0.8 \cdot h)$ $\quad d = [\,1.41 \quad 1.44\,]$

$d := \max(d)$ $\quad d = 1.44$ m.

- Eccentricity of cables from C.G. of girder :

$e_x := e_{sg} + y$ $\quad e_x = 0.52$ m.

- Calculate Vi & Mmax : Truck load is critical

$L_1 := x$ $\quad L_2 := 4.27$ m $\quad L_3 := 4.27$ m

$$V_x := \frac{(L_g - L_1) \cdot P_2 + (L_g - L_2 - L_1) \cdot P_2 + (L_g - L_3 - L_2 - L_1) \cdot P_1}{L_g}$$

$V_x = 323.23$ kN $\quad V_{x\_i} := (1 + I) \cdot V_x$ $\quad V_{x\_i} = 396.28$ kN.

$V_i := \dfrac{V_{x\_i}}{2} \cdot DF$ $\quad V_i = 260$ kN.

$M_{max} := V_i \cdot x$ $\quad M_{max} = 1819.71$ kN-m

$P_e := P_s - \left(\dfrac{P_s - P_m}{L}\right) \cdot 2 \cdot x$ $\quad P_e = 5454.42$ kN.

$f_{pe} := \dfrac{P_e \cdot 10^{-3}}{A_g} + \dfrac{P_e \cdot e_x \cdot c_{bg} \cdot 10^{-3}}{I_g}$ $\quad f_{pe} = 17.00$ Mpa.

$w_d := w_g + w_s + w_w$ $\quad w_d = 37$ kN/m

$M_d := R_{Ad} \cdot x - \dfrac{w_d \cdot x^2}{2}$ $\quad M_d = 2936.86$ kN-m

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$f_d := \dfrac{M_d \cdot c_{bg} \cdot 10^{-3}}{I_g}$ $\quad f_d = 9.22$ MPa.

$Y_t := c_{bg}$ $\quad Y_t = 0.91$ m.

$M_{cr} := \dfrac{I_g}{Y_t} \cdot \left(0.498 \cdot \sqrt{f_{c'}} + f_{pe} - f_d\right) \cdot 10^3$ $\quad M_{cr} = 3418$ kN-m.

$V_{ci} := 49.8 \cdot \sqrt{f_{c'}} \cdot b' \cdot d + V_d + \dfrac{V_i \cdot M_{cr}}{M_{max}}$ > or = $218.76 \cdot \sqrt{f_{c'}} \cdot b' \cdot d = 372$ kN.

$V_{ci} = 861$ kN.

Find Vcw:

$f_{pc} := \dfrac{P_e}{A_g} \cdot 10^{-3}$ $\quad f_{pc} = 8.02$ MPa.

$y' := \dfrac{4 \cdot h_{para}}{L} \cdot \left(1 - 2 \cdot \dfrac{x}{L}\right)$ $\quad y' = 0.05$

$V_p := P_e \cdot y'$ $\quad V_p = 262$ kN.

$V_{cw} := \left(0.29 \cdot \sqrt{f_{c'}} + 0.3 \cdot f_{pc}\right) \cdot b' \cdot d \cdot 10^3 + V_p$

$V_{cw} = 1448$ kN.

Find Shear capacity of concrete:

$V_c := \left[V_{ci} \;\; V_{cw}\right]$ $\quad V_c := \min(V_c)$ $\quad V_c = 861$ kN.

$V_u := \gamma \cdot \left(\beta_D \cdot V_d + \beta_L \cdot V_i\right)$ $\quad V_u = 939$ kN.

- AASHTO 9.14 for shear :

$\phi V_c := \phi \cdot V_c$ $\quad \phi V_c = 775$ kN.

$V_u = 939$ kN > $\phi V_c = 775$ kN.

- Need shear reinforcement :

- AASHTO Art. 9.20.3.1, web reinforcement shall be :

$V_s := \dfrac{A_v \cdot f_{sy} \cdot d}{s}$ $\quad V_s := \dfrac{V_u - \phi V_c}{\phi}$ $\quad V_s = 182.38$ kN.

$A_{v\%s} := \dfrac{V_s \cdot 10^3}{f_{sy} \cdot d}$ $\quad A_{v\%s} = 308.91 \dfrac{mm^2}{m}$ > $A_{vmin\%s} = 168.29 \dfrac{mm^2}{m}$

- Choose ***DB14 @ 0.15 as web reinforcement*** with : $A_{v\_prov} = 2053 \dfrac{mm^2}{m}$ OK

At section : $x := 12.00$ m

Find Vci:

$$V_d := \frac{\frac{L_g}{2} - x}{\frac{L_g}{2}} \cdot R_{Ad} \qquad V_d = 101.14 \quad \text{kN}$$

$$y := 4 \cdot h_{para} \cdot \left[\frac{x}{L} - \left(\frac{x}{L} \right)^2 \right] \qquad y = 0.65 \quad \text{m.}$$

- Distance from extreme compression fiber :

$$d := c_{tg} + e_{sg} + y \qquad d = 1.58 \quad \text{m.}$$

$$d := (d \quad 0.8 \cdot h) \qquad d = [\, 1.58 \quad 1.44 \,]$$

$$d := \max(d) \qquad d = 1.58 \quad \text{m.}$$

- Eccentricity of cables from C.G. of girder :

$$e_x := e_{sg} + y \qquad e_x = 0.69 \quad \text{m.}$$

- Calculate Vi & Mmax : Truck load is critical

$$L_1 := x \qquad L_2 := 4.27 \text{ m} \qquad L_3 := 4.27 \text{ m}$$

$$V_x := \frac{(L_g - L_1) \cdot P_2 + (L_g - L_2 - L_1) \cdot P_2 + (L_g - L_3 - L_2 - L_1) \cdot P_1}{L_g}$$

$$V_x = 240.58 \text{ kN} \qquad V_{x\_i} := (1 + I) \cdot V_x \qquad V_{x\_i} = 294.95 \quad \text{kN.}$$

$$V_i := \frac{V_{x\_i}}{2} \cdot DF \qquad V_i = 193 \quad \text{kN.}$$

$$M_{max} := V_i \cdot x \qquad M_{max} = 2321.81 \quad \text{kN-m}$$

$$P_e := P_s - \left(\frac{P_s - P_m}{L} \right) \cdot 2 \cdot x \qquad P_e = 5300.90 \quad \text{kN.}$$

$$f_{pe} := \frac{P_e \cdot 10^{-3}}{A_g} + \frac{P_e \cdot e_x \cdot c_{bg} \cdot 10^{-3}}{I_g} \qquad f_{pe} = 19.23 \quad \text{Mpa.}$$

$$w_d := w_g + w_s + w_w \qquad w_d = 37 \quad \text{kN/m}$$

$$M_d := R_{Ad} \cdot x - \frac{w_d \cdot x^2}{2} \qquad M_d = 3910.82 \quad \text{kN-m}$$

$f_d := \frac{M_d \cdot c_{bg} \cdot 10^{-3}}{I_g}$ $f_d = 12.27$ MPa.

$Y_t := c_{bg}$ $Y_t = 0.91$ m.

$M_{cr} := \frac{I_g}{Y_t} \cdot \left(0.498 \cdot \sqrt{f_{c'}} + f_{pe} - f_d\right) \cdot 10^3$ $M_{cr} = 3153$ kN-m.

$V_{ci} := 49.8 \cdot \sqrt{f_{c'}} \cdot b' \cdot d + V_d + \frac{V_i \cdot M_{cr}}{M_{max}}$ > or = $218.76 \cdot \sqrt{f_{c'}} \cdot b' \cdot d = 407$ kN.

$V_{ci} = 457$ kN.

Find Vcw:

$f_{pc} := \frac{P_e}{A_g} \cdot 10^{-3}$ $f_{pc} = 7.80$ MPa.

$y' := \frac{4 \cdot h_{para}}{L} \cdot \left(1 - 2 \cdot \frac{x}{L}\right)$ $y' = 0.02$

$V_p := P_e \cdot y'$ $V_p = 89$ kN.

$V_{cw} := \left(0.29 \cdot \sqrt{f_{c'}} + 0.3 \cdot f_{pc}\right) \cdot b' \cdot d \cdot 10^3 + V_p$

$V_{cw} = 1367$ kN.

Find Shear capacity of concrete:

$V_c := \left[V_{ci} \;\; V_{cw}\right]$ $V_c := \min(V_c)$ $V_c = 457$ kN.

$V_u := \gamma \cdot \left(\beta_D \cdot V_d + \beta_L \cdot V_i\right)$ $V_u = 552$ kN.

- **AASHTO 9.14** for shear :

$\phi V_c := \phi \cdot V_c$ $\phi V_c = 411$ kN. < $V_u = 552$ kN

- Need shear reinforcement :
- **AASHTO Art. 9.20.3.1**, web reinforcement shall be :

$V_s := \frac{A_v \cdot f_{sy} \cdot d}{s}$ $V_s := \frac{V_u - \phi V_c}{\phi}$ $V_s = 156.29$ kN.

$A_{v\%s} := \frac{V_s \cdot 10^3}{f_{sy} \cdot d}$ $A_{v\%s} = 241.69 \; \frac{mm^2}{m}$ > $A_{vmin\%s} = 168.29 \; \frac{mm^2}{m}$

- Choose ***DB14 @ 0.20 as web reinforcement*** - with : $s_{prov} := 0.20$ m

$A_{v\_prov} := \frac{A_v}{s_{prov}}$ $A_{v\_prov} = 1539.38 \; \frac{mm^2}{m}$ OK.

****

At section : $x := \frac{L}{2}$ $x = 14.70$ m

****

Find Vci:

$V_d := \dfrac{\frac{L_g}{2} - x}{\frac{L_g}{2}} \cdot R_{Ad}$ $V_d = 0.00$ kN

$y := 4 \cdot h_{para} \cdot \left[\frac{x}{L} - \left(\frac{x}{L} \right)^2 \right]$ $y = 0.67$ m.

- Distance from extreme compression fiber :

$d := c_{tg} + e_{sg} + y$ $d = 1.60$ m.

$d := (d \quad 0.8 \cdot h)$ $d = [\, 1.60 \quad 1.44 \,]$

$d := \max(d)$ $d = 1.60$ m.

- Eccentricity of cables from C.G. of girder :

$e_x := e_{sg} + y$ $e_x = 0.71$ m.

- Calculate Vi & Mmax : Truck load is critical

$L_1 := x$ $L_2 := 4.27$ m $L_3 := 4.27$ m

$$V_x := \frac{(L_g - L_1) \cdot P_2 + (L_g - L_2 - L_1) \cdot P_2 + (L_g - L_3 - L_2 - L_1) \cdot P_1}{L_g}$$

$V_x = 195.94$ kN $V_{x\_i} := (1 + I) \cdot V_x$ $V_{x\_i} = 240.23$ kN.

$V_i := \frac{V_{x\_i}}{2} \cdot DF$ $V_i = 158$ kN.

$M_{max} := V_i \cdot x$ $M_{max} = 2316.55$ kN-m

$P_e := P_s - \left(\frac{P_s - P_m}{L} \right) \cdot 2 \cdot x$ $P_e = 5218.00$ kN.

$f_{pe} := \frac{P_e \cdot 10^{-3}}{A_g} + \frac{P_e \cdot e_x \cdot c_{bg} \cdot 10^{-3}}{I_g}$ $f_{pe} = 19.30$ Mpa.

$w_d := w_g + w_s + w_w$ $w_d = 37$ kN/m

$M_d := R_{Ad} \cdot x - \frac{w_d \cdot x^2}{2}$ $M_d = 4047.37$ kN-m

$f_d := \dfrac{M_d \cdot c_{bg} \cdot 10^{-3}}{I_g}$ $\quad f_d = 12.70$ MPa.

$Y_t := c_{bg}$ $\quad Y_t = 0.91$ m.

$M_{cr} := \dfrac{I_g}{Y_t} \cdot \left(0.498 \cdot \sqrt{f_{c'}} + f_{pe} - f_d\right) \cdot 10^3$ $\quad M_{cr} = 3039$ kN-m.

$b' := 0.20$ m

$V_{ci} := 49.8 \cdot \sqrt{f_{c'}} \cdot b' \cdot d + V_d + \dfrac{V_i \cdot M_{cr}}{M_{max}}$ > or = $218.76 \cdot \sqrt{f_{c'}} \cdot b' \cdot d = 413$ kN.

$V_{ci} = 301$ kN.

Find Vcw:

$f_{pc} := \dfrac{P_e}{A_g} \cdot 10^{-3}$ $\quad f_{pc} = 7.67$ MPa.

$y' := \dfrac{4 \cdot h_{para}}{L} \cdot \left(1 - 2 \cdot \dfrac{x}{L}\right)$ $\quad y' = 0.00$

$V_p := P_e \cdot y'$ $\quad V_p = 0$ kN.

$V_{cw} := \left(0.29 \cdot \sqrt{f_{c'}} + 0.3 \cdot f_{pc}\right) \cdot b' \cdot d \cdot 10^3 + V_p$

$V_{cw} = 1284$ kN.

Find Shear capacity of concrete:

$V_c := \left[V_{ci} \ \ V_{cw}\right]$ $\quad V_c := \min(V_c)$ $\quad V_c = 301$ kN.

$V_u := \gamma \cdot \left(\beta_D \cdot V_d + \beta_L \cdot V_i\right)$ $\quad V_u = 342$ kN > $\phi \cdot V_c = 271$ kN.

- Need shear reinforcement :

- AASHTO Art. 9.20.3.1, web reinforcement shall be :

$V_s := \dfrac{A_v \cdot f_{sy} \cdot d}{s}$ $\quad V_s := \dfrac{V_u - \phi \cdot V_c}{\phi}$ $\quad V_s = 79.39$ kN.

$A_{v\%s} := \dfrac{V_s \cdot 10^3}{f_{sy} \cdot d}$ $\quad A_{v\%s} = 121.02 \ \dfrac{mm^2}{m}$ < $A_{vmin\%s} = 168.29 \ \dfrac{mm^2}{m}$

- Choose ***DB14 @ 0.20 as web reinforcement*** with : $A_{v\_prov} = 1539 \ \dfrac{mm^2}{m}$ OK

PROJECT : Design Bridges in Jordan

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SUBJECT : Diaphragm Design for Prestressed I-Girder, Span 30 m.

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2.3 Diaphragm Design:

Material Properties:

1.1 Concrete:

- Cylinder strength (Class 30): $f_{c'} := 26.1$ MPa
- Concrete Density: $\gamma_c := 25$ kN/m³

1.2 Reinforcing Steel:

- Deformed bar of high tensile strength, Grade 60, conforming to JSS/441/1986 or AASHTO M31M (ASTM, A615M) :
- Yield strength : $f_y := 410$ MPa

Intermediate Diaphragm:

- Spacing between girder: $S_{cc} := 2.20$ m
- Depth of the diaphragm: $D := 1.35$ m
- Width of the diaphragm: $b := 0.60$ m

Dead Loads:

- Self-weigth: $w_g := (1.35 \cdot 0.6 + 0.2 \cdot 0.2) \cdot \gamma_c$ $w_g = 21.25$ kN/m
- Deck slab: $w_s := (0.20 \cdot 0.60) \cdot 25$ $w_s = 3$ kN/m
- Wearing surface: $w_a := (0.10 \cdot 0.60) \cdot 18$ $w_a = 1.08$ kN/m
- Sum of dead loads: $w_D := w_g + w_s + w_a$ $w_D = 25.33$ kN/m
- Moment: $M_D := \frac{1}{10} \cdot w_D \cdot S_{cc}^2$ $M_D = 12.26$ kN-m

Live Loads:

$L := S_{cc} \cdot 3.28$ $L = 7.216$ ft.

- Impact factor: $I := \frac{50}{(L + 125)}$

  $I = 0.378$ > 0.30 $I := 0.30$

- Wheel load: $P := 144 \cdot 1.5$ $P = 216$ kN.
- Moment: $M_L := \frac{P \cdot S_{cc}}{4}$ $M_L = 118.8$ kN-m

Load Combinations:

Group I: $Factored\_Load := \gamma \cdot \left[\beta_D \cdot D + \beta_L \cdot (L + I)\right]$

Where : $\gamma := 1.3$ $\beta_D := 1.0$ $\beta_L := 1.67$ $I = 0.3$

Design Moment:

Therefore, design ultimate moment is:

$$M_\gamma := \gamma \cdot \left(\beta_D \cdot M_D + \beta_L \cdot M_L\right)$$

$M_\gamma = 274$ kN-m

Reinforcement Design:

- Select reinforcement:

Bar diameter : $\phi_b := 25$ mm

Cross section area : $A_b := \dfrac{\pi \cdot \phi_b^{\,2}}{4}$ $A_b = 490.874$ mm^2

From Vol. (III) Design Drawings,
General Notes: minimum concrete cover for girder diaphragm is 50 mm.

$cov := 50$ mm.

Effective depth: $d := D - \dfrac{cov}{1000} - \dfrac{\phi_b}{2 \cdot 1000}$ $d = 1.288$ m.

Number of rebars: $No := 5$

Total reinforcement area: $A_s := No \cdot A_b$ $A_s = 2.454 \cdot 10^3$ mm^2

For flexural design: $\phi := 0.90$

$a := \dfrac{A_s \cdot f_y \cdot 10^{-6}}{0.85 \cdot f_{c'} \cdot b}$ $a = 0.076$ m

$$M_u := \left[\phi \cdot A_s \cdot f_y \cdot \left(d - \frac{a}{2}\right)\right] \cdot 10^{-3}$$

$M_u = 1132$ kN-m > $M_\gamma = 274$ kN-m **OK.**

2.4 Bearing Design:

Superstructure & other relevant members Dead Loads:

Concrete density:		$\gamma_c := 25$	m		
Asphalt overlay density:		$\gamma_a := 18$	m		
Bridge span length:		$L_{span} := 30$	m		
Beidge width:		$B := 12.80$	m		
Girder spacing:		$G_{sp} := 2.20$	m		
1. Girder:	Typical section:	$A_{gt} := 0.678$	m^2		
	End section:	$A_{gs} := 1.317$	m^2		
	Average area:	$A_g := \dfrac{A_{gt}\cdot 26.7 + A_{gs}\cdot 3.3}{L_{span}}$		$A_g = 0.748$	m^2
	Distributed load:	$w_g := A_g\cdot\gamma_c\cdot 6$		$w_g = 112.244$	kN/m
2. Girder-Diapragm:	Intermediate diapraghm:	$A_{di} := 0.6\cdot 1.35 + 0.2\cdot 0.2$		$A_{di} = 0.85$	m^2
	End diapraghm:	$A_{de} := 0.65\cdot 1.55 + \frac{1}{2}\cdot 0.2\cdot 0.2$		$A_{de} = 1.028$	m^2
	Length of diapraghm:	$L_d := 2.2\cdot 5 - 0.2\cdot 6$		$L_d = 9.8$	m
	Distributed load:	$w_d := \left(A_{de} + A_{di}\right)\cdot\dfrac{L_d\cdot\gamma_c\cdot 2}{L_{span}}$		$w_d = 30.666$	kN/m
3. Deck slab:	Slab thickness:	$t := 0.20$			
	Distributed load:	$w_s := t\cdot B\cdot\gamma_c$		$w_s = 64$	kN/m
4. Parapet+Sidewalk:	Cross section area:	$A_p := \left[0.3\cdot 0.55 + \left(\dfrac{0.3 + 0.36}{2}\right)\cdot 1.5\right]\cdot 2$		$A_p = 1.32$	m^2
	Distributed load:	$w_p := A_p\cdot\gamma_c$		$w_p = 33$	kN/m
5. Asphalt wearing surface:	thickness:	$t_a := 0.10$	m		
	Distributed load:	$w_a := t_a\cdot(B - 3)\cdot\gamma_a$		$w_a = 17.64$	kN/m
Total distributed load:		$w := w_g + w_d + w_s + w_p + w_a$		$w = 257.549$	kN/m
Total superstructure dead loads:		$DL_{sup} := w\cdot L_{span}$		$DL_{sup} = 7726$	kN
Dead load reaction for one girder:		$R_g := w\cdot\dfrac{L_{span}}{2\cdot 6}$		$R_g = 643.873$	kN

6. Live loads:

- From section 2.2.2.2, maximum shear due to wheel load plus impact:

$R_{WLI} := 353.02$ kN

- Impact factor for main girder: $I := 0.226$

- Live load not include impact: $R_L := \dfrac{R_{WLI}}{(1+I)}$ $R_L = 287.945$ kN

- Reaction: $R := R_g + R_L$ $R = 931.818$ kN

- Thermal forces: computed based on a range of temperature variation of temp. rise of 22 C and temp. fall of 20 C.

- Therefore: $\Delta T := 22 + 20$ $\Delta T = 42$ C

- Thermal coefficient: $\alpha := 1.08 \cdot 10^{-5}$

- Horizontal movement due to temperature: $\Delta h := \alpha \cdot L_{span} \cdot \Delta T$ $\Delta h = 0.014$ m

$\Delta h := \Delta h \cdot 100$ $\Delta h = 1.36$ cm

- Minimum thickness required: $T_{req} := \dfrac{\Delta h}{0.5}$ $T_{req} = 2.722$ cm

- Try elastomeric bearing with: 4 layers of 6.25 mm thick of neoprene &
5 steel plates with 3 mm thick &
neoprene cover both top & bottom = 3 mm

- Total bearing thickness: $T := 4 \cdot 6.25 + 5 \cdot 3 + 2 \cdot 3$ $T = 46$ mm > $T_{req} \cdot 10 = 27$ mm
OK

$A_{sreq} := \dfrac{R_g \cdot 10^3}{3.448}$ $A_{sreq} = 186738$ mm^2 = 1867 cm^2

$A_{sreq} := \dfrac{R \cdot 10^3}{5.517}$ $A_{sreq} = 168899$ mm^2 = 1689 cm^2

$f_{DL} := \dfrac{R_g \cdot 10^3}{300 \cdot 600}$ $f_{DL} = 3.577$ MPa > 1.38 MPa

2.5 Expansion Joint Design:

- Thermal forces: computed based on a range of temperature variation of temp. rise of 22 C and temp. fall of 20 C.

- Therefore: $T_{rise} := 22$ C

  $T_{fall} := -20$ C

- Thermal coefficient: $\alpha := 1.08 \cdot 10^{-5}$

Contraction:

- Due to temperature: $\varepsilon_{tc} := T_{fall} \cdot \alpha \cdot 10^{3}$ $\quad \varepsilon_{tc} = -0.216$ mm/m

- Due to shrinkage (AASHTO 8.5.4): $\varepsilon_{sh} := -0.2$ mm/m

- Creep stress: $\sigma_{cr} := -16.18$ MPa

- Prestressing modulus: $E_{ps} := 1.95 \cdot 10^{5}$ MPa

- Contraction due to creep: $\varepsilon_{cr} := \dfrac{\sigma_{cr}}{E_{ps}}$ $\quad \varepsilon_{cr} = -0.0001$ $\quad \varepsilon_{cr} := -0.1$ mm/m

- Total contraction: $\varepsilon_{c} := \varepsilon_{tc} + \varepsilon_{sh} + \varepsilon_{cr}$

  $\varepsilon_{c} = -0.516$ mm/m

Expansion:

- Due to temperature: $\varepsilon_{te} := T_{rise} \cdot \alpha \cdot 10^{3}$ $\quad \varepsilon_{te} = 0.238$ mm/m

- Total length of Wadi Himara Bridge: $L_{bridge} := 120$ m

- Expansion joint: gap required: $Gap_{req} := \left(\varepsilon_{te} - \varepsilon_{c}\right) \cdot \dfrac{L_{bridge}}{2}$

  $Gap_{req} = 45.216$ mm

- Provide gap for expansion joint: $Gap_{prov} := 55$ mm > $Gap_{req} = 45$ mm OK

3. Substructure

3.1 Abutment Design:

Material Properties & Loads:

1.1 Concrete:

- Cylinder strength (Class C25): $f_{c'} := 21.75$ MPa
- Modulus of Elasticity: $E_c := 4730 \cdot \sqrt{f_{c'}}$ $E_c = 2.206 \cdot 10^4$ MPa
- Concrete Density: $\gamma_c := 25$ kN/m³

1.2 Reinforcing Steel:

- Deformed bar of high tensile strength, Grade 60, conforming to JSS/441/1986 or AASHTO M31M (ASTM, A615M) :
- Yield strength : $f_y := 410$ MPa

1.3 Soil:

- Density: $\gamma_s := 18$ kN/m^3
- Bearing Capacity: $B_r := 500$ kN/m^2
- Angle of internal friction: $\phi_s := 30$ Degrees
- Original soil angle: $\psi := 40$ Degrees

1.4 Surcharge:

- Surcharge: $w_{sur} := 10$ kN/m^2

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SUBJECT : Abutment Design

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3.1.1 Wadi Himara Bridge

Abutment dimension: see Fig.3.1 for details

- Typical width of stem:	$w_{stem} := 1.05$	m		
- Stem heigth:	$h_{stem} := 8.00 + 2.05$	$h_{stem} = 10.05$	m	
- Toe width:	$w_{toe} := 1.50$	m		
- Heel width:	$w_{heel} := 3.45$	m		
- Abutment base:	$w_{base} := w_{toe} + w_{stem} + w_{heel}$		$w_{base} = 6.00$	m
- Depth of abutment base:	$h_{base} := 0.80$	m		

.1.1.1 Stability Analysis:

Resisting Moments:

- Load due to:	$W_1 := 0.30 \cdot 2.05 \cdot \gamma_c$	$W_1 = 15.375$	kN
- Moment arm:	$x_1 := w_{toe} + 0.75 + \frac{0.30}{2}$	$x_1 = 2.4$	m
- Moment:	$M_1 := W_1 \cdot x_1$	$M_1 = 36.9$	kN-m
- Load due to:	$W_2 := w_{stem} \cdot 8.00 \cdot \gamma_c$	$W_2 = 210$	kN
- Moment arm:	$x_2 := w_{toe} + \frac{w_{stem}}{2}$	$x_2 = 2.025$	m
- Moment:	$M_2 := W_2 \cdot x_2$	$M_2 = 425.25$	kN-m
- Load due to:	$W_3 := 0.50 \cdot 0.60 \cdot \gamma_s$	$W_3 = 5.4$	kN
- Moment arm:	$x_3 := w_{toe} + w_{stem} + \frac{0.50}{2}$	$x_3 = 2.8$	m
- Moment:	$M_3 := W_3 \cdot x_3$	$M_3 = 15.12$	kN-m
- Load due to:	$W_4 := 0.50 \cdot 0.30 \cdot \gamma_c$	$W_4 = 3.75$	kN
- Moment arm:	$x_4 := w_{toe} + w_{stem} + \frac{0.50}{2}$	$x_4 = 2.8$	m
- Moment:	$M_4 := W_4 \cdot x_4$	$M_4 = 10.5$	kN-m
- Load due to:	$W_5 := \frac{1}{2} \cdot 0.5 \cdot 0.5 \cdot \gamma_c$	$W_5 = 3.125$	kN
- Moment arm:	$x_5 := w_{toe} + w_{stem} + \frac{0.50}{3}$	$x_5 = 2.717$	m
- Moment:	$M_5 := W_5 \cdot x_5$	$M_5 = 8.49$	kN-m
- Load due to:	$W_6 := \frac{1}{2} \cdot 0.50 \cdot 0.50 \cdot \gamma_s$	$W_6 = 2.25$	kN
- Moment arm:	$x_6 := w_{toe} + w_{stem} + 0.50 \cdot \frac{2}{3}$	$x_6 = 2.883$	m
- Moment:	$M_6 := W_6 \cdot x_6$	$M_6 = 6.487$	kN-m
- Load due to:	$W_7 := 0.50 \cdot 8.65 \cdot \gamma_s$	$W_7 = 77.85$	kN
- Moment arm:	$x_7 := w_{toe} + w_{stem} + \frac{0.50}{2}$	$x_7 = 2.8$	m
- Moment:	$M_7 := W_7 \cdot x_7$	$M_7 = 217.98$	kN-m

- Load due to: $W_8 := (w_{heel} - 0.50) \cdot h_{stem} \cdot \gamma_s$ $W_8 = 533.655$ kN

- Moment arm: $x_8 := w_{toe} + w_{stem} + 0.50 + \dfrac{w_{heel} - 0.5}{2}$ $x_8 = 4.525$ m

- Moment: $M_8 := W_8 \cdot x_8$ $M_8 = 2.415 \cdot 10^3$ kN-m

- Load due to: $W_9 := w_{base} \cdot h_{base} \cdot \gamma_c$ $W_9 = 120$ kN

- Moment arm: $x_9 := \dfrac{w_{base}}{2}$ $x_9 = 3$ m

- Moment: $M_9 := W_9 \cdot x_9$ $M_9 = 360$ kN-m

- Surcharge load: $W_{10} := w_{heel} \cdot w_{sur}$ $W_{10} = 34.5$ kN

- Moment arm: $x_{10} := w_{toe} + w_{stem} + \dfrac{w_{heel}}{2}$ $x_{10} = 4.275$ m

- Moment: $M_{10} := W_{10} \cdot x_{10}$ $M_{10} = 147.488$ kN-m

- Load due to DL: $W_{11} := \dfrac{643.87 \cdot 6}{12.8}$ $W_{11} = 301.814$ kN

- Moment arm: $x_{11} := w_{toe} + 0.45$ $x_{11} = 1.95$ m

- Moment: $M_{11} := W_{11} \cdot x_{11}$ $M_{11} = 588.537$ kN-m

- Total weigth: $\Sigma W := W_1 + W_2 + W_3 + W_4 + W_5 + W_6 + W_7 + W_8 + W_9 + W_{10} + W_{11}$

$\Sigma W = 1308$ kN

- Resisting moment: $M_r := M_1 + M_2 + M_3 + M_4 + M_5 + M_6 + M_7 + M_8 + M_9 + M_{10} + M_{11}$

$M_r = 4232$ kN-m

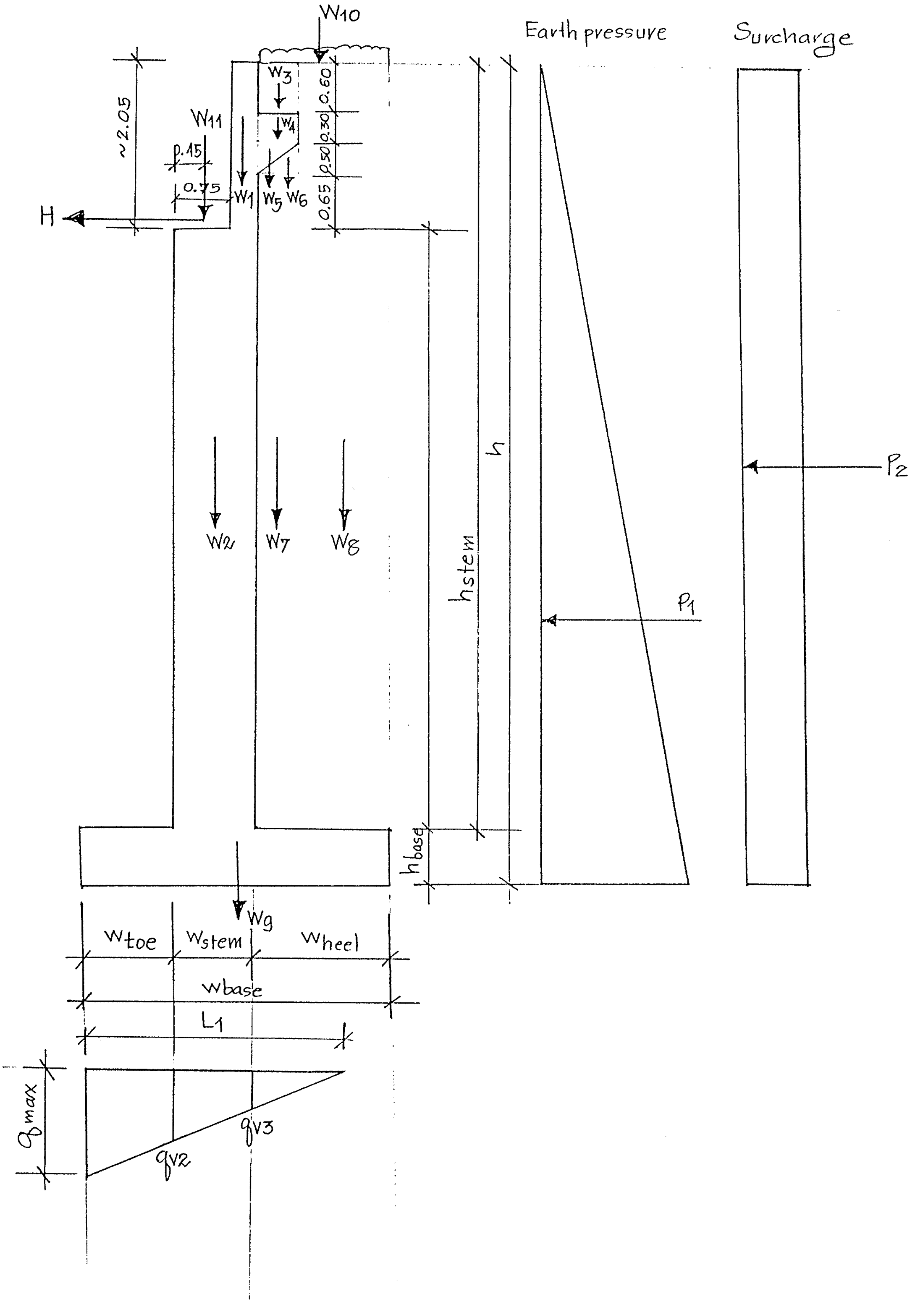

Fig. 3.1 Abutment Dimension & Loading

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SUBJECT : Abutment Design

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Overturning Moments:

- Earth pressure: $q := \gamma_s \cdot h \cdot k_q$

$\theta := \left(45 - \frac{\phi_s}{2}\right) \cdot \frac{\pi}{180}$	$\theta = 0.524$	rad
$k_q := (\tan(\theta))^2$	$k_q = 0.333$	
$h := h_{stem} + 0.80$	$h = 10.85$	m
$q_1 := \gamma_s \cdot h \cdot k_q$	$q_1 = 65.1$	kN/m^2
$P_1 := \frac{1}{2} \cdot q_1 \cdot h$	$P_1 = 353.168$	kN/m
$M_{01} := P_1 \cdot \frac{h}{3}$	$M_{01} = 1277$	kN-m

- Surcharge:

$q_2 := w_{sur} \cdot k_q$	$q_2 = 3.333$	kN/m^2
$P_2 := q_2 \cdot h$	$P_2 = 36.167$	kN
$M_{02} := P_2 \cdot \frac{h}{2}$	$M_{02} = 196$	kN-m

- Horizontal shear force for each elastomeric bearing:

- Shear moduls of elastomer:	$G := 1.24$	MPa	
- Area of bearing pad:	$A := 300 \cdot 600$	$A = 1.8 \cdot 10^5$	mm^2
- Horizontal movement of superstructure:	$\Delta h := 13.6$	mm	
- Total elastomer thickness of the bearing:	$h_{rt} := 31$	mm	
- Design shear force on bearing:	$H_1 := G \cdot A \cdot \frac{\Delta h}{h_{rt}} \cdot 10^{-3}$	$H_1 = 97.92$	kN

- Average horizontal shear force per meter of abutment:

$H := \frac{6 \cdot H_1}{12.8}$	$H = 45.9$	kN
$h_3 := h_{base} + 8.00 + 0.05$	$h_3 = 8.85$	m
$M_{03} := H \cdot h_3$	$M_{03} = 406$	kN-m

- Total horizontal force:

$\Sigma P_h := P_1 + P_2 + H$	$\Sigma P_h = 435.234$	kN

- Sum of overturning moments:

$M_0 := M_{01} + M_{02} + M_{03}$	$M_0 = 1880$	kN-m

PROJECT : Design Bridges in Jordan

SUBJECT : Abutment Design

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Check for Overturning:

$M_0 = 1880$ kN-m

$M_r = 4232$ kN-m

$FS := \frac{M_r}{M_0}$ $FS = 2.251$ > 2.0 OK

Check for Sliding:

$FS := \frac{\Sigma W \cdot \tan\left(\frac{2}{3} \cdot \psi\right)}{\Sigma P_h}$ $FS = 1.51$ > 1.5 OK

Check for Bearing stress:

$B := 1$ m

$e_L := \frac{L}{2} - \frac{\Delta M}{\Sigma W}$

$L := w_{toe} + w_{stem} + w_{heel}$ $L = 6$ m

$\Delta M := M_r - M_0$ $\Delta M = 2.352 \cdot 10^3$ kN-m

$\Sigma W = 1.308 \cdot 10^3$ kN

$e_L := \frac{L}{2} - \frac{\Delta M}{\Sigma W}$ $e_L = 1.202$ > $\frac{L}{6} = 1$

< $\frac{L}{2} = 3$

AASHTO 4.4.7.1.1.1 Eccentric Loading:

$Q := \Sigma W$ $Q = 1.308 \cdot 10^3$ kN

$q_{max} := \frac{2 \cdot Q}{3 \cdot B \cdot \left(\frac{L}{2} - e_L\right)}$ $q_{max} = 485$ kN/m^2 < 500 kN/m^2 OK

$q_{min} := 0$

$L_1 := 3\left(\frac{L}{2} - e_L\right)$ $L_1 = 5.395$ m

3.1.1.2 Stem Design:

- Earth pressure: $h_{stem} = 10.05$ m

$q_1 := \gamma_s \cdot h_{stem} \cdot k_q$ $q_1 = 60.3$ kN/m^2

$P_1 := \frac{1}{2} \cdot q_1 \cdot h_{stem}$ $P_1 = 303.007$ kN/m

$M_{01} := P_1 \cdot \frac{h_{stem}}{3}$ $M_{01} = 1015$ kN-m

- Surcharge: $q_2 := w_{sur} \cdot k_q$ $q_2 = 3.333$ kN/m^2

$P_2 := q_2 \cdot h_{stem}$ $P_2 = 33.5$ kN

$M_{02} := P_2 \cdot \frac{h_{stem}}{2}$ $M_{02} = 168$ kN-m

- Horizontal shear force:

$H = 45.9$ kN

$h_3 := 8.00 + 0.05$ $h_3 = 8.05$ m

$M_{03} := H \cdot h_3$ $M_{03} = 369$ kN-m

- Total horizontal force:

$\Sigma P_h := P_1 + P_2 + H$ $\Sigma P_h = 382.407$ kN

- Sum of moments:

$M_0 := M_{01} + M_{02} + M_{03}$ $M_0 = 1553$ kN-m

Design Moment:

Therefore, design ultimate moment is:

$M_\gamma := \gamma \cdot \beta_E \cdot M_0$

Where : $\gamma := 1.3$ $\beta_E := 1.3$ for lateral earth pressure for retaining walls and reigid frames

$M_\gamma := \gamma \cdot \beta_E \cdot M_0$

$M_\gamma = 2624$ kN-m

Reinforcement Design:

Maximum depth of stem: $D := w_{stem}$ $D = 1.05$ m

Design width of the stem: $b := 1.00$ m

Bar diameter : $\phi_b := 32$ mm

Cross section area : $A_b := \frac{\pi \cdot \phi_b^2}{4}$ $A_b = 804.248$ mm^2

From Vol. (III) Design Drawings, General Notes: minimum concrete cover for abutment walls is 70 mm.

$cov := 70$ mm.

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Effective depth: $d := D - \frac{cov}{1000} - \frac{\phi_b}{2 \cdot 1000}$ $d = 0.964$ m.

Bar spacing: $s := 0.10$

Number of rebars: $No := \frac{b}{s}$ $No = 10$

Total reinforcement area: $A_s := No \cdot A_b$ $A_s = 8.042 \cdot 10^3$ mm^2

For flexural design: $\phi := 0.90$

$a := \frac{A_s \cdot f_y \cdot 10^{-6}}{0.85 \cdot f_{c'} \cdot b}$ $a = 0.178$ m

$M_u := \left[\phi \cdot A_s \cdot f_y \cdot \left(d - \frac{a}{2} \right) \right] \cdot 10^{-3}$

$M_u = 2596$ kN-m < $M_\gamma = 2624$ kN-m **Say OK.**

<u>Consider the point at 5.00 m from top of Abutment base:</u>

- Heigth of the stem above 5 m: $h_5 := h_{stem} - 5$ $h_5 = 5.05$ m
- Earth pressure: $q_5 := \gamma_s \cdot h_5 \cdot k_q$ $q_5 = 30.3$ kN/m^2

$P_5 := \frac{1}{2} \cdot q_5 \cdot h_5$ $P_5 = 76.508$ kN

$M_5 := P_5 \cdot \frac{h_5}{3}$ $M_5 = 129$ kN-m

- Surcharge: $P_{25} := q_2 \cdot h_5$ $P_{25} = 16.833$ kN

$M_{25} := P_{25} \cdot \frac{h_5}{2}$ $M_{25} = 43$ kN-m

- Moment at 5 m: $M_5 := M_5 + M_{25} + H \cdot (8.05 - 5.0)$ $M_5 = 311.287$ kN-m

$M_\gamma := \gamma \cdot \beta_E \cdot M_5$

$M_\gamma = 526$ kN-m

Bar diameter : $\phi_b := 20$ mm

Cross section area : $A_b := \frac{\pi \cdot \phi_b^2}{4}$ $A_b = 314.159$ mm^2

Effective depth: $d := D - \frac{cov}{1000} - \frac{\phi_b}{2 \cdot 1000}$ $d = 0.972$ m.

Bar spacing: $s := 0.20$

Number of rebars: $No := \frac{b}{s}$ $No = 5$

Total reinforcement area: $A_s := No \cdot A_b$ $A_s = 1.571 \cdot 10^3$ mm^2

$a := \frac{A_s \cdot f_y \cdot 10^{-6}}{0.85 \cdot f_{c'} \cdot b}$ $a = 0.035$ m

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$$M_u := \left[\phi \cdot A_s \cdot f_y \cdot \left(d - \frac{a}{2}\right)\right] \cdot 10^{-3}$$

$M_u = 553$ kN-m > $M_\gamma = 526$ kN-m OK

USE DB20 @0.20 or DB16@0.10 c/c for stem (5m above top of abutment base) reinforcement:

3.1.1.3 Base Design:

- Maximum pressure: $q_{max} = 484.765$ kN/m^2
- Length L1: $L_1 = 5.395$ m

(a) Toe Design:

- Pressure at the left face of the stem: $q_{v2} := q_{max} \cdot \frac{(L_1 - w_{toe})}{L_1}$ $q_{v2} = 349.99$ kN/m^2
- Moment at the face of the stem:

$$M_E := q_{v2} \cdot \frac{w_{toe}^2}{2} + \frac{1}{2} \cdot (q_{max} - q_{v2}) \cdot w_{toe}^2 \cdot \frac{2}{3}$$ $M_E = 494.82$ kN-m

$$M_D := h_{base} \cdot \gamma_c \cdot \frac{w_{toe}^2}{2}$$ $M_D = 22.5$ kN-m

With: $\gamma = 1.3$ $\beta_D := 1$ $\beta_E = 1.3$

$$M_\gamma := \gamma \cdot (\beta_E \cdot M_E - \beta_D \cdot M_D)$$ $M_\gamma = 807$ kN-m

Bar diameter : $\phi_b := 32$ mm

Cross section area : $A_b := \frac{\pi \cdot \phi_b^2}{4}$ $A_b = 804.248$ mm^2

Concrete cover: $cov := 75$ mm

Effective depth: $d := h_{base} - \frac{cov \cdot 2}{1000} - \frac{\phi_b}{2 \cdot 1000}$ $d = 0.634$ m.

Bar spacing: $s := 0.20$

Number of rebars: $No := \frac{b}{s}$ $No = 5$

Total reinforcement area: $A_s := No \cdot A_b$ $A_s = 4.021 \cdot 10^3$ mm^2

$$a := \frac{A_s \cdot f_y \cdot 10^{-6}}{0.85 \cdot f_{c'} \cdot b}$$ $a = 0.089$ m

$$M_u := \left[\phi \cdot A_s \cdot f_y \cdot \left(d - \frac{a}{2}\right)\right] \cdot 10^{-3}$$

$M_u = 875$ kN-m > $M_\gamma = 807$ kN-m OK.

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(b) Heel Design:

- Pressure at the right face of the stem: $q_{v3} := q_{max} \cdot \frac{(L_1 - w_{toe} - w_{stem})}{L_1}$ $q_{v3} = 255.648$ kN/m^2

- Moment at the right face of the stem:

$M_D := h_{base} \cdot \gamma_c \cdot \frac{w_{heel}^2}{2} + h_{stem} \cdot \gamma_s \cdot \frac{w_{heel}^2}{2}$ $M_D = 1.196 \cdot 10^3$ kN-m

$M_L := w_{sur} \cdot \frac{w_{heel}^2}{2}$ $M_L = 59.513$ kN-m

$M_E := \frac{1}{2} \cdot q_{v3} \cdot (L_1 - w_{toe} - w_{heel})^2 \cdot \frac{1}{3}$ $M_E = 8.448$ kN-m

$\gamma = 1.3$ $\beta_D = 1$ $\beta_E = 1.3$ $\beta_L := 1.67$

$M_\gamma := \gamma \cdot (\beta_D \cdot M_D + \beta_L \cdot M_L - \beta_E \cdot M_E)$ $M_\gamma = 1669$ kN-m

Bar diameter : $\phi_b := 32$ mm

Cross section area : $A_b := \frac{\pi \cdot \phi_b^2}{4}$ $A_b = 804.248$ mm^2

Effective depth: $d := h_{base} - \frac{cov}{1000} - \frac{\phi_b}{2 \cdot 1000}$ $d = 0.709$ m.

Bar spacing: $s := 0.10$

Number of rebars: $No := \frac{b}{s}$ $No = 10$

Total reinforcement area: $A_s := No \cdot A_b$ $A_s = 8.042 \cdot 10^3$ mm^2

$a := \frac{A_s \cdot f_y \cdot 10^{-6}}{0.85 \cdot f_c' \cdot b}$ $a = 0.178$ m

$M_u := \left[\phi \cdot A_s \cdot f_y \cdot \left(d - \frac{a}{2}\right)\right] \cdot 10^{-3}$

$M_u = 1839$ kN-m $>$ $M_\gamma = 1669$ kN-m OK

USE DB32 @0.10 c/c for abutment base reinforcement:

3.1.2 Wingwalls Design:

- Design check for L-shape retaining wall

- Concrete cylinder strength (Class C25): $f_{c'} := 21.75$ MPa

- Earth pressure: $h_{wall} := h_{stem}$ $h_{wall} = 10.05$ m

$q_1 := \gamma_s \cdot h_{wall} \cdot k_q$ $q_1 = 60.3$ kN/m^2

$P_1 := \frac{1}{2} \cdot q_1 \cdot h_{wall}$ $P_1 = 303.007$ kN/m

$M_{01} := P_1 \cdot \frac{h_{wall}}{3}$ $M_{01} = 1015$ kN-m

- Surcharge: $q_2 := w_{sur} \cdot k_q$ $q_2 = 3.333$ kN/m^2

$P_2 := q_2 \cdot h_{wall}$ $P_2 = 33.5$ kN

$M_{02} := P_2 \cdot \frac{h_{wall}}{2}$ $M_{02} = 168$ kN-m

- Sum of moments:

$M_0 := M_{01} + M_{02}$ $M_0 = 1183$ kN-m

Design Moment:

Therefore, design ultimate moment is:

$M_\gamma := \gamma \cdot \beta_E \cdot M_0$

$M_\gamma = 2000$ kN-m

Reinforcement Design:

Maximum depth of wall: $D := 1.10$ m

Design width of the wall: $b := 1.00$ m

Bar diameter : $\phi_b := 32$ mm

Cross section area : $A_b := \frac{\pi \cdot \phi_b^{\,2}}{4}$ $A_b = 804.248$ mm^2

From Vol. (III) Design Drawings, General Notes: minimum concrete cover for abutment walls is 70 mm.

$cov := 70$ mm.

Effective depth: $d := D - \frac{cov}{1000} - \frac{\phi_b}{2 \cdot 1000}$ $d = 1.014$ m.

Bar spacing: $s := 0.15$

Number of rebars: $No := \frac{b}{s} \cdot 2$ $No = 13.333$

Total reinforcement area: $A_s := No \cdot A_b$ $A_s = 1.072 \cdot 10^4$ mm^2

$a := \frac{A_s \cdot f_y \cdot 10^{-6}}{0.85 \cdot f_c' \cdot b}$ $a = 0.238$ m

$M_u := \left[\phi \cdot A_s \cdot f_y \cdot \left(d - \frac{a}{2}\right)\right] \cdot 10^{-3}$

$M_u = 3542$ kN-m > $M_\gamma = 2000$ kN-m **OK.**

Consider the point at 4.00 m from top of the wall base:

- Heigth of the wall above 4 m: $h_4 := h_{wall} - 4$ $h_4 = 6.05$ m
- Depth of the wall at 4 m: $D := 0.30 + \frac{0.80}{h_{wall}} \cdot h_4$ $D = 0.782$ m
- Earth pressure: $q_4 := \gamma_s \cdot h_4 \cdot k_q$ $q_4 = 36.3$ kN/m^2

  $P_4 := \frac{1}{2} \cdot q_4 \cdot h_4$ $P_4 = 109.808$ kN

  $M_{14} := P_4 \cdot \frac{h_4}{3}$ $M_{14} = 221$ kN-m
- Surcharge: $P_{24} := q_2 \cdot h_4$ $P_{24} = 20.167$ kN

  $M_{24} := P_{24} \cdot \frac{h_4}{2}$ $M_{24} = 61$ kN-m
- Moment at 5 m: $M_4 := M_{14} + M_{24}$ $M_4 = 282.449$ kN-m

  $M_\gamma := \gamma \cdot \beta_E \cdot M_4$

  $M_\gamma = 477$ kN-m

Bar diameter : $\phi_b := 32$ mm

Cross section area : $A_b := \frac{\pi \cdot \phi_b^2}{4}$ $A_b = 804.248$ mm^2

Effective depth: $d := D - \frac{cov}{1000} - \frac{\phi_b}{2 \cdot 1000}$ $d = 0.696$ m.

Bar spacing: $s := 0.15$

Number of rebars: $No := \frac{b}{s}$ $No = 6.667$

Total reinforcement area: $A_s := No \cdot A_b$ $A_s = 5.362 \cdot 10^3$ mm^2

$a := \frac{A_s \cdot f_y \cdot 10^{-6}}{0.85 \cdot f_c' \cdot b}$ $a = 0.119$ m

$M_u := \left[\phi \cdot A_s \cdot f_y \cdot \left(d - \frac{a}{2}\right)\right] \cdot 10^{-3}$

$M_u = 1259$ kN-m > $M_\gamma = 477$ kN-m **OK.**

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3.1.3 Parapet Design:

- Concrete cylinder strength (Class C30) same as deck slab: $f_{c'} := 26.1$ MPa
- Hiegth of parapet: $h_p := 0.55$ m
- Width of parapet: $w_p := 0.30$ m
- Concrete cover: $cov := 50$ mm
- AASHTO 2.7.1.3.1: Transverse concentrated load, P=10 kips should be applied at top of parapet and distributed 5 ft (1.524 m) along the longitudinal length of the bridge (AASHTO 2.7.1.3.6).

$P := 45$ kN

$L_{dist} := 1.524$ m

- Moment: $M := P \cdot h_p$ $M = 24.75$ kN-m
- Factored moment: $M_\gamma := \gamma \cdot \beta_L \cdot M$ $M_\gamma = 53.732$ kN-m
- R/F used: DB14@0.20:

Bar diameter : $\phi_b := 14$ mm

Cross section area : $A_b := \frac{\pi \cdot \phi_b^2}{4}$ $A_b = 153.938$ mm^2

Effective depth: $d := w_p - \frac{cov}{1000} - \frac{\phi_b}{2 \cdot 1000}$ $d = 0.243$ m.

Bar spacing: $s := 0.20$

Total reinforcement area: $A_s := \frac{A_b}{s} \cdot L_{dist}$ $A_s = 1.173 \cdot 10^3$ mm^2

$a := \frac{A_s \cdot f_y \cdot 10^{-6}}{0.85 \cdot f_{c'} \cdot b}$ $a = 0.022$ m

$$M_u := \left[\phi \cdot A_s \cdot f_y \cdot \left(d - \frac{a}{2}\right)\right] \cdot 10^{-3}$$

$M_u = 100$ kN-m > $M_\gamma = 54$ kN-m **OK.**

3.1.4 Approach Cushion Slab Design:

- Concrete cylinder strength (Class C25): $f_{c'} := 21.75$ MPa
- Length of approach slab: $L_{ap} := 3.0$ m
- Thickness of approach slab: $t_{ap} := 0.30$ m
- Thickness of fill over slab: $t_{fill} := 0.25$ m

Dead loads:

- Concrete slab: $w_{slab} := t_{ap} \cdot \gamma_c$ $w_{slab} = 7.5$ kN/m^2
- Fill: $w_{fill} := t_{fill} \cdot \gamma_s$ $w_{fill} = 4.5$ kN/m^2
- Sum of dead loads: $w_{dl} := w_{slab} + w_{fill}$ $w_{dl} = 12$ kN/m^2
- Moment due to dead loads: $M_{dl} := \frac{1}{2} \cdot w_{dl} \cdot (L_{ap} - 0.25)^2$ $M_{dl} = 45.375$ kN-m/m

Live loads:

- Live load surcharge: $w_{sur} = 10$ kN/m^2
- Moment due to surcharge: $M_{ll} := \frac{1}{2} \cdot w_{sur} \cdot (L_{ap} - 0.25)^2$ $M_{ll} = 37.813$ kN-m/m

Design Moment:

Therefore, design ultimate moment is:

$$M_\gamma := \gamma \cdot \left[\beta_D \cdot (M_{dl}) + \beta_L \cdot (M_{ll})\right]$$

$M_\gamma = 141$ kN-m

Reinforcement Design:

Depth of approach slab: $D_{ap} := 0.30$ m

Design width of slab: $b := 1.0$ m

- Select reinforcement:

Bar diameter : $\phi_b := 20$ mm

Cross section area : $A_b := \frac{\pi \cdot \phi_b^{\,2}}{4}$ $A_b = 314.159$ mm^2

From Vol. (III) Design Drawings,
General Notes: minimum concrete cover for abutment walls, wingwalls and pier is 70 mm.

$cov := 70$ mm.

Effective depth: $d := D_{ap} - \frac{cov}{1000} - \frac{\phi_b}{2 \cdot 1000}$ $d = 0.223$ m.

Reinforcement spacing: $s := 0.15$ m.

Number of rebars: $No := \frac{b}{s}$ $No = 6.667$

Total reinforcement area: $A_s := No \cdot A_b$ $A_s = 2{,}094$ mm^2

For flexural design: $\phi := 0.90$

$$a := \frac{A_s \cdot f_y \cdot 10^{-6}}{0.85 \cdot f_c' \cdot b}$$ $a = 0.046$ m

$$M_u := \left[\phi \cdot A_s \cdot f_y \cdot \left(d - \frac{a}{2}\right)\right] \cdot 10^{3}$$

$M_u = 154$ kN-m/m > $M_y = 141$ kN-m/m OK

<u>USE DB20 @ 0.15 c/c for Top & Bottom reinforcement as main reinforcement:</u>

- Minimum reinforcement for shrinkage and temperature stresses required by AASHTO 8.20.1:

$A_{smin} := 264$ mm^2/m

Provide bar diameter : $\phi_b := 12$ mm

Cross section area : $A_b := \frac{\pi \cdot \phi_b^{2}}{4}$ $A_b = 113.097$ mm^2

Reinforcement spacing: $s := 0.20$ m.

Number of rebars: $No := \frac{b}{s}$ $No = 5$

Total reinforcement area: $A_s := No \cdot A_b$ $A_s = 565$ mm^2

<u>USE DB12 @ 0.20 c/c for Top & Bottom reinforcement as temperature steel:</u>

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3.2 Pier Design:

Group VII: Loading = γ(D+EQ)

Since ground acceleration (A) = 0.2g, from AASHTO Table 3.4, Seismic Performance Category (SPC) for this bridge is therefore **C**.

From Table 4.2A, minimum analysis requirements for SPC C & regular bridges with 2-6 spans, follows Procedure 1 or 2.

Select Procedure 2: ***SINGLE MODE SPECTRAL ANALYSIS METHOD***

Material Properties:

1.1 Concrete:

- Cylinder strength (Class 30): $f_{c'} := 26.1$ MPa
- Modulus of Elasticity: $E_c := 4730 \cdot \sqrt{f_{c'}}$ $E_c = 2.416 \cdot 10^4$ MPa
- Concrete Density: $\gamma_c := 25$ kN/m3

1.2 Reinforcing Steel:

- Deformed bar of high tensile strength, Grade 60, conforming to JSS/441/1986 or AASHTO M31M (ASTM, A615M) :
- Yield strength : $f_y := 410$ MPa

1.3 Asphalt wearing surface:

- Density: $\gamma_a := 18$ kN/m^3
- Thickness of asphalt overlay: $t_a := 0.05$ m
- In Final Design Report, 3.4.2 Bridge Design said that "Additional 5 cm of future wearing surface shall be considered", therefore, let say, future wearing surface thickness equals to 10 cm.

$t_a := 0.10$ m

WADI HIMARA BRIDGE:

- Width of pier in longitudinal direction: $wd_l := 1.50$ m
- Width of pier in transverse direction: $wd_t := 8.00$ m
- Hiegth of pier P2: $h_{p2} := 29.20$ m
- Hiegth of pier P3: $h_{p3} := 30.35$ m
- Hiegth of pier P4: $h_{p4} := 20.45$ m
- Total bridge length: $L := 120$ m $\quad L := 120 \cdot 10^3$ mm
- Longitudinal moment of inertia $I_l := \frac{1}{12} \cdot wd_t \cdot wd_l^{\,3} \cdot 10^{12}$ $\quad I_l = 2.25 \cdot 10^{12}$ mm^4
- Transverse moment of inertia: $I_t := \frac{1}{12} \cdot wd_l \cdot wd_t^{\,3} \cdot 10^{12}$ $\quad I_t = 6.4 \cdot 10^{13}$ mm^4

- Weight per unit length of the dead load of the bridge superstructure and tributary substructure:

$W_{DL} := 6 \cdot 2 \cdot 4 \cdot 644$ $\quad W_{DL} = 3.091 \cdot 10^4$ kN

$W_{LL} := 1.5 \cdot 4 \cdot (2 \cdot 18 + 4 \cdot 72)$ $\quad W_{LL} = 1.944 \cdot 10^3$ kN

$W_{DL\_sub} := 10031$ kN

$\Sigma W_{DL} := W_{DL} + W_{DL\_sub}$ $\quad \Sigma W_{DL} = 4.094 \cdot 10^4$ kN

$\Sigma W = \Sigma W_{DL} + W_{LL}$ $\quad \Sigma W = 4.289 \cdot 10^4$ kN

$w := \frac{\Sigma W}{L \cdot 10^{-3}}$ $\quad w = 357.392$ kN/m

- General stiffness of pier: $k := \frac{3 \cdot E \cdot I}{h^3}$

Stifness of pier P2:

- Longitudinal direction: $k_{l2} := \frac{3 \cdot E_c \cdot I_l}{\left(h_{p2} \cdot 1000\right)^3}$ $\quad k_{l2} = 6.551 \cdot 10^3$
- Transverse direction: $k_{t2} := \frac{3 \cdot E_c \cdot I_t}{\left(h_{p2} \cdot 1000\right)^3}$ $\quad k_{t2} = 1.864 \cdot 10^5$

Stifness of pier P3:

- Longitudinal direction: $k_{l3} := \frac{3 \cdot E_c \cdot I_l}{\left(h_{p3} \cdot 1000\right)^3}$ $\quad k_{l3} = 5.835 \cdot 10^3$
- Transverse direction: $k_{t3} := \frac{3 \cdot E_c \cdot I_t}{\left(h_{p3} \cdot 1000\right)^3}$ $\quad k_{t3} = 1.66 \cdot 10^5$

Stifness of pier P4:

- Longitudinal direction: $k_{l4} := \frac{3 \cdot E_c \cdot I_l}{\left(h_{p4} \cdot 1000\right)^3}$ $\quad k_{l4} = 1.907 \cdot 10^4$

- Transverse direction: $k_{t4} := \dfrac{3 \cdot E_c \cdot I_t}{\left(h_{p4} \cdot 1000\right)^3}$ $k_{t4} = 5.425 \cdot 10^5$

LONGITUDINAL EARTHQUAKE

1. Calculate the static displacements $v_s(x)$ due to an assumed uniform loading p_o

- Assumed uniform loading: $p_o := 1$ N/m

- Static displacements: $v_s := \dfrac{p_o \cdot L}{k_{12} + k_{13} + k_{14}}$ $v_s = 3.815$ mm

2. Calculate factors α, β, and γ from:

$\alpha := \int_0^L v_s(x)\,dx$ $\alpha := v_s \cdot L$ $\alpha = 4.577 \cdot 10^5$ mm^2

$\beta := \int_0^L w(x) \cdot v_s(x)\,dx$ $\beta := w \cdot v_s \cdot L$ $\beta = 1.636 \cdot 10^8$ N-mm

$\gamma := \int_0^L w(x) \cdot v_s(x)^2\,dx$ $\gamma := w \cdot v_s^{\,2} \cdot L$ $\gamma = 6.24 \cdot 10^8$ N-mm^2

3. Calculate the period of the bridge, T

$T := 2 \cdot \pi \cdot \sqrt{\dfrac{\gamma}{p_o \cdot g \cdot \alpha}}$ (Note: units must be changed from SI to American custom units)

Therefore: $\gamma_f := \dfrac{\gamma}{(25.4 \cdot 12)^2} \cdot \dfrac{600}{2669}$ $\gamma_f = 1.51 \cdot 10^3$ lb-ft^2

$p_o := 0.0685 \cdot 10^3$ lb/ft

$g := 32.2$ ft/sec^2

$\alpha_f := \dfrac{\alpha}{(25.4 \cdot 12)^2}$ $\alpha_f = 4.927$ ft^2

$T := 2 \cdot \pi \cdot \sqrt{\dfrac{\gamma_f}{p_o \cdot g \cdot \alpha_f}}$ $T = 2.342$ sec

4. Calculate the equivalent static earthquake loading $p_e(x)$ from

$$p_e(x) := \frac{\beta \cdot C_s}{\gamma} \cdot w(x) \cdot v_s(x)$$

- Elastic seismic response coefficient, C_s

$$C_s := \frac{1.2\ A \cdot S}{T^{\frac{2}{3}}}$$ Where: the acceleration coefficient: $A := 0.19$ g

Site Coeffient for soil profile type I: $S := 1.0$

C_s < $2.5 \cdot A = 0.475$

$$C_s := \frac{1.2\ A \cdot S}{T^{\frac{2}{3}}}$$ $C_s = 0.129$ < $2.5 \cdot A = 0.475$

$$p_e := \frac{\beta \cdot C_s}{\gamma} \cdot w \cdot v_s$$ $p_e = 46$ N/mm

- Longitudinal EQ per one span:

$EQ_{long} := p_e \cdot 30$ $EQ_{long} = 1386$ kN

- Displacement due to equivalent staic earthquake, p_e

$$v_s := \frac{p_e \cdot L}{k_{12} + k_{13} + k_{14}}$$ $v_s = 176.247$ mm

__Longitudinal Moment, Ml__

- AASHTO 3.7 stated that seismic design forces for individual members and connections of bridges classified as SPC B, C, or D are determined by dividing the elastic forces by the appropriate Response Modification Factor (R).

- From Table 3.7, a wall-type pier may be designed as a column in the weak direction of the pier provided all the provisions for columns in Article 6.6 (SPC B) or 7.6 (SPC C & D), as appropriate, are followed.

$R_l := 3$

__Pier 2:__ $$M_{l2} := \frac{3 \cdot E_c \cdot I_l}{\left(h_{p2} \cdot 1000\right)^2} \cdot v_s \cdot 10^{-6}$$ $M_{l2} = 33716$ kN-m

$$\frac{M_{l2}}{R_l} = 11239$$ kN-m

__Pier 3:__ $$M_{l3} := \frac{3 \cdot E_c \cdot I_l}{\left(h_{p3} \cdot 1000\right)^2} \cdot v_s \cdot 10^{-6}$$ $M_{l3} = 31210$ kN-m

$$\frac{M_{l3}}{R_l} = 10403$$ kN-m

__Pier 4:__ $$M_{l4} := \frac{3 \cdot E_c \cdot I_l}{\left(h_{p4} \cdot 1000\right)^2} \cdot v_s \cdot 10^{-6}$$ $M_{l4} = 68742$ kN-m

$$\frac{M_{l4}}{R_l} = 22914$$ kN-m

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TRANSVERSE EARTHQUAKE

1. Calculate the static displacements $v_s(x)$ due to an assumed uniform loading p_o

- Assumed uniform loading: $p_o := 1$ N/m

- Static displacements: $v_s := \dfrac{p_o \cdot L}{k_{t2} + k_{t3} + k_{t4}}$ $v_s = 0.134$ mm

2. Calculate factors α, β, and γ from:

$\alpha := v_s \cdot L$ $\alpha = 1.609 \cdot 10^4$ mm^2

$\beta := w \cdot v_s \cdot L$ $\beta = 5.751 \cdot 10^6$ N-mm

$\gamma := w \cdot v_s^2 \cdot L$ $\gamma = 7.713 \cdot 10^5$ N-mm^2

3. Calculate the period of the bridge, T

$\gamma_f := \dfrac{\gamma}{(25.4 \cdot 12)^2} \cdot \dfrac{600}{2669}$ $\gamma_f = 1.866$ lb-ft^2

$p_o := 0.0685 \cdot 10^3$ lb/ft

$\alpha_f := \dfrac{\alpha}{(25.4 \cdot 12)^2}$ $\alpha_f = 0.173$ ft^2

$T := 2 \cdot \pi \cdot \sqrt{\dfrac{\gamma_f}{p_o \cdot g \cdot \alpha_f}}$ $T = 0.439$ sec

4. Calculate the equivalent static earthquake loading $p_e(x)$ from

$C_s := \dfrac{1.2\ A \cdot S}{T^{\frac{2}{3}}}$ $C_s = 0.395$ < $2.5 \cdot A = 0.475$ OK

$p_e := \dfrac{\beta \cdot C_s}{\gamma} \cdot w \cdot v_s$ $p_e = 141$ N/mm

- Transverse EQ per one span:

$EQ_{trans} := p_e \cdot 30$ $EQ_{trans} = 4231$ kN

- Displacement due to equivalent staic earthquake, p_e

$v_s := \dfrac{p_e \cdot L}{k_{t2} + k_{t3} + k_{t4}}$ $v_s = 18.914$ mm

Transverse Moment, Mt

- From Table 3.7, a wall-type pier, for strong direction

$$R_t := 2$$

Pier 2: $M_{t2} := \frac{3 \cdot E_c \cdot I_t}{(h_{p2} \cdot 1000)^2} \cdot v_s \cdot 10^{-6}$ $\quad M_{t2} = 102922$ kN-m

$\frac{M_{t2}}{R_t} = 51461$ kN-m

Pier 3: $M_{t3} := \frac{3 \cdot E_c \cdot I_t}{(h_{p3} \cdot 1000)^2} \cdot v_s \cdot 10^{-6}$ $\quad M_{t3} = 95270$ kN-m

$\frac{M_{t3}}{R_t} = 47635$ kN-m

Pier 4: $M_{t4} := \frac{3 \cdot E_c \cdot I_t}{(h_{p4} \cdot 1000)^2} \cdot v_s \cdot 10^{-6}$ $\quad M_{t4} = 209839$ kN-m

$\frac{M_{t4}}{R_t} = 104920$ kN-m

PROJECT : Design Bridges in Jordan

SUBJECT : Pier Design

SHEET NO.........OF......

JOB NO.......................

DRAWING NO..............

DATE....JULY...2000....

Design Moment: M_{γ}

AASHTO 3.9 Division IA: Combination of Orthogonal Seismic Forces

LOAD CASE 1: $M_{design} := \gamma\cdot(100\,\% M_{long} + 30\,\% M_{trans})$ $\gamma := 1.3$

Pier 2: $M_{\gamma l2} := 100\,\% \dfrac{M_{l2}}{R_l}\cdot\gamma$ $M_{\gamma l2} = 14610$ kN-m

$M_{\gamma t2} := 30\,\%\cdot\dfrac{M_{t2}}{R_t}\cdot\gamma$ $M_{\gamma t2} = 20070$ kN-m

Pier 3: $M_{\gamma l3} := 100\,\% \dfrac{M_{l3}}{R_l}\cdot\gamma$ $M_{\gamma l3} = 13524$ kN-m

$M_{\gamma t3} := 30\,\%\cdot\dfrac{M_{t3}}{R_t}\cdot\gamma$ $M_{\gamma t3} = 18578$ kN-m

Pier 4: $M_{\gamma l4} := 100\,\% \dfrac{M_{l4}}{R_l}\cdot\gamma$ $M_{\gamma l4} = 29788$ kN-m

$M_{\gamma t4} := 30\,\%\cdot\dfrac{M_{t4}}{R_t}\cdot\gamma$ $M_{\gamma t4} = 40919$ kN-m

LOAD CASE 2: $M_{design} := \gamma\cdot(30\,\% M_{long} + 100\,\% M_{trans})$

Pier 2: $M_{\gamma l2} := 30\,\% \dfrac{M_{l2}}{R_l}\cdot\gamma$ $M_{\gamma l2} = 4383$ kN-m

$M_{\gamma t2} := 100\,\%\cdot\dfrac{M_{t2}}{R_t}\cdot\gamma$ $M_{\gamma t2} = 66899$ kN-m

Pier 3: $M_{\gamma l3} := 30\,\% \dfrac{M_{l3}}{R_l}\cdot\gamma$ $M_{\gamma l3} = 4057$ kN-m

$M_{\gamma t3} := 100\,\%\cdot\dfrac{M_{t3}}{R_t}\cdot\gamma$ $M_{\gamma t3} = 61925$ kN-m

Pier 4: $M_{\gamma l4} := 30\,\% \dfrac{M_{l4}}{R_l}\cdot\gamma$ $M_{\gamma l4} = 8936$ kN-m

$M_{\gamma t4} := 100\,\%\cdot\dfrac{M_{t4}}{R_t}\cdot\gamma$ $M_{\gamma t4} = 136395$ kN-m

Maximum biaxial moments of Pier 4 is critical both for Load case 1 & 2, therefore, capacity of piers are checked by using computer program NovaDesign. Moment capacity (see Fig.3.2) is OK for the given cross section.

PROJECT : Design Bridges in Jordan

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WADI ABU EL-ASAL BRIDGE:

- Width of pier in longitudinal direction: $wd_l := 1.50$ m
- Width of pier in transverse direction: $wd_t := 8.00$ m
- Hiegth of pier P2: $h_{p2} := 14.95$ m
- Hiegth of pier P3: $h_{p3} := 24.40$ m
- Total bridge length: $L := 90$ m $\quad L := 90\cdot 10^3$ mm
- Longitudinal moment of inertia $I_l := \frac{1}{12}\cdot wd_t \cdot wd_l^3 \cdot 10^{12}$ $\quad I_l = 2.25\cdot 10^{12}$ mm^4
- Transverse moment of inertia: $I_t := \frac{1}{12}\cdot wd_l \cdot wd_t^3 \cdot 10^{12}$ $\quad I_t = 6.4\cdot 10^{13}$ mm^4
- Weight per unit length of the dead load of the bridge superstructure and tributary substructure:

$W_{DL} := 6\cdot 2\cdot 3\cdot 644$ $\quad W_{DL} = 2.318\cdot 10^4$ kN

$W_{LL} := 1.5\cdot 4\cdot (2\cdot 18 + 4\cdot 72)$ $\quad W_{LL} = 1.944\cdot 10^3$ kN

$W_{DL\_sub} := 4167$ kN

$\Sigma W_{DL} := W_{DL} + W_{DL\_sub}$ $\quad \Sigma W_{DL} = 2.735\cdot 10^4$ kN

$\Sigma W := \Sigma W_{DL} + W_{LL}$ $\quad \Sigma W = 2.93\cdot 10^4$ kN

$w := \frac{\Sigma W}{L\cdot 10^{-3}}$ $\quad w = 325.5$ kN/m

- General stiffness of pier: $k := \frac{3\cdot E\cdot I}{h^3}$

Stifness of pier P2:

- Longitudinal direction: $k_{l2} := \frac{3\cdot E_c\cdot I_l}{(h_{p2}\cdot 1000)^3}$ $\quad k_{l2} = 4.882\cdot 10^4$
- Transverse direction: $k_{t2} := \frac{3\cdot E_c\cdot I_t}{(h_{p2}\cdot 1000)^3}$ $\quad k_{t2} = 1.389\cdot 10^6$

Stifness of pier P3:

- Longitudinal direction: $k_{l3} := \frac{3\cdot E_c\cdot I_l}{(h_{p3}\cdot 1000)^3}$ $\quad k_{l3} = 1.123\cdot 10^4$
- Transverse direction: $k_{t3} := \frac{3\cdot E_c\cdot I_t}{(h_{p3}\cdot 1000)^3}$ $\quad k_{t3} = 3.194\cdot 10^5$

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LONGITUDINAL EARTHQUAKE

1. Calculate the static displacements $v_s(x)$ due to an assumed uniform loading p_o

- Assumed uniform loading: $p_o := 1$ N/m

- Static displacements: $v_s := \frac{p_o \cdot L}{k_{12} + k_{13}}$ $v_s = 1.499$ mm

2. Calculate factors α, β, and γ from:

$\alpha := \int_0^L v_s(x)\,dx$ $\alpha := v_s \cdot L$ $\alpha = 1.349 \cdot 10^5$ mm^2

$\beta := \int_0^L w(x) \cdot v_s(x)\,dx$ $\beta := w \cdot v_s \cdot L$ $\beta = 4.391 \cdot 10^7$ N-mm

$\gamma := \int_0^L w(x) \cdot v_s(x)^2\,dx$ $\gamma := w \cdot v_s^2 \cdot L$ $\gamma = 6.582 \cdot 10^7$ N-mm^2

3. Calculate the period of the bridge, T

$T := 2 \cdot \pi \cdot \sqrt{\frac{\gamma}{p_o \cdot g \cdot \alpha}}$ (Note: units must be changed from SI to American custom units)

Therefore: $\gamma_f := \frac{\gamma}{(25.4 \cdot 12)^2} \cdot \frac{600}{2669}$ $\gamma_f = 159.26$ lb-ft^2

$p_o := 0.0685 \cdot 10^3$ lb/ft

$g := 32.2$ ft/sec^2

$\alpha_f := \frac{\alpha}{(25.4 \cdot 12)^2}$ $\alpha_f = 1.452$ ft^2

$T := 2 \cdot \pi \cdot \sqrt{\frac{\gamma_f}{p_o \cdot g \cdot \alpha_f}}$ $T = 1.401$ sec

4. Calculate the equivalent static earthquake loading $p_e(x)$ from

$$p_e(x) := \frac{\beta \cdot C_s}{\gamma} \cdot w(x) \cdot v_s(x)$$

- Elastic seismic response coefficient, C_s

$C_s := \frac{1.2\ A \cdot S}{T^{\frac{2}{3}}}$ $C_s = 0.182$ < $2.5 \cdot A = 0.475$ OK

$p_e := \frac{\beta \cdot C_s}{\gamma} \cdot w \cdot v_s$ $\quad p_e = 59$ N/mm

- Longitudinal EQ per one span:

$EQ_{long} := p_e \cdot 30$ $\quad EQ_{long} = 1778$ kN

- Displacement due to equivalent staic earthquake, p_e

$v_s := \frac{p_e \cdot L}{k_{12} + k_{13}}$ $\quad v_s = 88.84$ mm

Longitudinal Moment, Ml

Pier 2: $M_{12} := \frac{3 \cdot E_c \cdot I_1}{\left(h_{p2} \cdot 1000\right)^2} \cdot v_s \cdot 10^{-6}$ $\quad M_{12} = 64836$ kN-m

$\frac{M_{12}}{R_1} = 21612$ kN-m

Pier 3: $M_{13} := \frac{3 \cdot E_c \cdot I_1}{\left(h_{p3} \cdot 1000\right)^2} \cdot v_s \cdot 10^{-6}$ $\quad M_{13} = 24340$ kN-m

$\frac{M_{13}}{R_1} = 8113$ kN-m

TRANSVERSE EARTHQUAKE

1. Calculate the static displacements $v_s(x)$ due to an assumed uniform loading p_o

- Assumed uniform loading: $p_o := 1$ N/m

- Static displacements: $v_s := \frac{p_o \cdot L}{k_{t2} + k_{t3}}$ $\quad v_s = 0.053$ mm

2. Calculate factors α, β, and γ from:

$\alpha := v_s \cdot L$ $\quad \alpha = 4.743 \cdot 10^3$ mm^2

$\beta := w \cdot v_s \cdot L$ $\quad \beta = 1.544 \cdot 10^6$ N-mm

$\gamma := w \cdot v_s^{\,2} \cdot L$ $\quad \gamma = 8.135 \cdot 10^4$ N-mm^2

3. Calculate the period of the bridge, T

$\gamma_f := \frac{\gamma}{(25.4 \cdot 12)^2} \cdot \frac{600}{2669}$ $\quad \gamma_f = 0.197$ lb-ft^2

$p_o := 0.0685 \cdot 10^3$ lb/ft

$\alpha_f := \dfrac{\alpha}{(25.4 \cdot 12)^2}$ $\qquad \alpha_f = 0.051 \qquad ft^2$

$T := 2 \cdot \pi \cdot \sqrt{\dfrac{\gamma_f}{p_o \cdot g \cdot \alpha_f}}$ $\qquad T = 0.263 \qquad$ sec

4. Calculate the equivalent static earthquake loading $p_e(x)$ from

$C_s := \dfrac{1.2\ A \cdot S}{T^{\frac{2}{3}}}$ $\qquad C_s = 0.556 \quad > \quad 2.5 \cdot A = 0.475$

Therefore, $\qquad C_s := 2.5\ A \qquad C_s = 0.475$

$p_e := \dfrac{\beta \cdot C_s}{\gamma} \cdot w \cdot v_s$ $\qquad p_e = 155 \qquad$ N/mm

- Transverse EQ per one span:

$EQ_{trans} := p_e \cdot 30$ $\qquad EQ_{trans} = 4638 \qquad$ kN

- Displacement due to equivalent staic earthquake, p_e

$v_s := \dfrac{p_e \cdot L}{k_{t2} + k_{t3}}$ $\qquad v_s = 8.147 \qquad$ mm

Transverse Moment, Mt

- From Table 3.7, a wall-type pier, for strong direction

$R_t := 2$

Pier 2: $\quad M_{t2} := \dfrac{3 \cdot E_c \cdot I_t}{\left(h_{p2} \cdot 1000\right)^2} \cdot v_s \cdot 10^{-6}$ $\qquad M_{t2} = 169129 \qquad$ kN-m

$\dfrac{M_{t2}}{R_t} = 84565$ $\qquad$ kN-m

Pier 3: $\quad M_{t3} := \dfrac{3 \cdot E_c \cdot I_t}{\left(h_{p3} \cdot 1000\right)^2} \cdot v_s \cdot 10^{-6}$ $\qquad M_{t3} = 63492 \qquad$ kN-m

$\dfrac{M_{t3}}{R_t} = 31746$ $\qquad$ kN-m

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Design Moment: M_γ

AASHTO 3.9 Division IA: Combination of Orthogonal Seismic Forces

LOAD CASE 1: $M_{design} := (100\,\% M_{long} + 30\,\% M_{trans})\cdot\gamma$ $\gamma := 1.3$

Pier 2:

$M_{\gamma l2} := 100\,\%\dfrac{M_{l2}}{R_l}\cdot\gamma$ $M_{\gamma l2} = 28095$ kN-m

$M_{\gamma t2} := 30\,\%\cdot\dfrac{M_{t2}}{R_t}\cdot\gamma$ $M_{\gamma t2} = 32980$ kN-m

Pier 3:

$M_{\gamma l3} := 100\,\%\dfrac{M_{l3}}{R_l}\cdot\gamma$ $M_{\gamma l3} = 10547$ kN-m

$M_{\gamma t3} := 30\,\%\cdot\dfrac{M_{t3}}{R_t}\cdot\gamma$ $M_{\gamma t3} = 12381$ kN-m

LOAD CASE 2: $M_{design} := (30\,\% M_{long} + 100\,\% M_{trans})\cdot\gamma$

Pier 2:

$M_{\gamma l2} := 30\,\%\dfrac{M_{l2}}{R_l}\cdot\gamma$ $M_{\gamma l2} = 8429$ kN-m

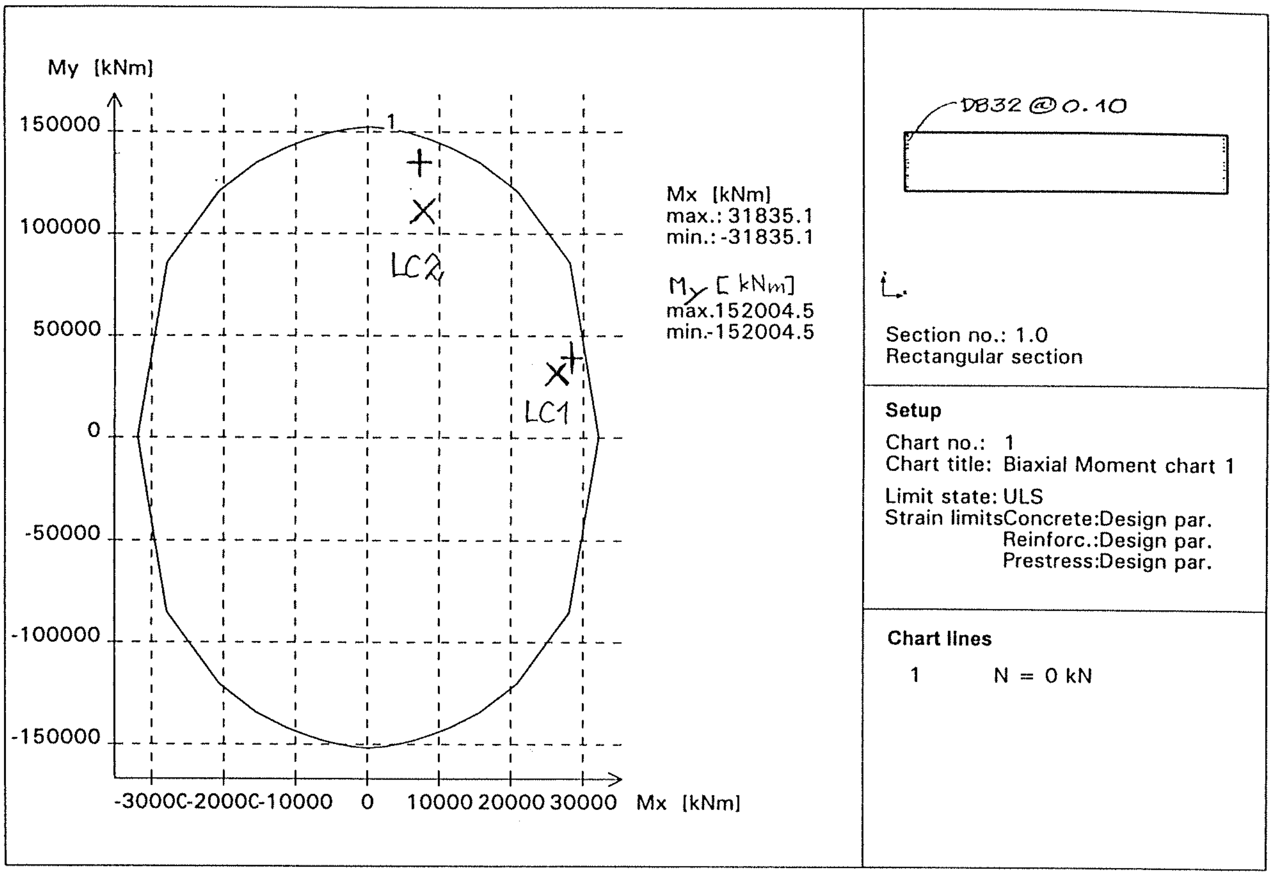
$M_{\gamma t2} := 100\,\%\cdot\dfrac{M_{t2}}{R_t}\cdot\gamma$ $M_{\gamma t2} = 109934$ kN-m

Pier 3:

$M_{\gamma l3} := 30\,\%\dfrac{M_{l3}}{R_l}$ $M_{\gamma l3} = 2434$ kN-m

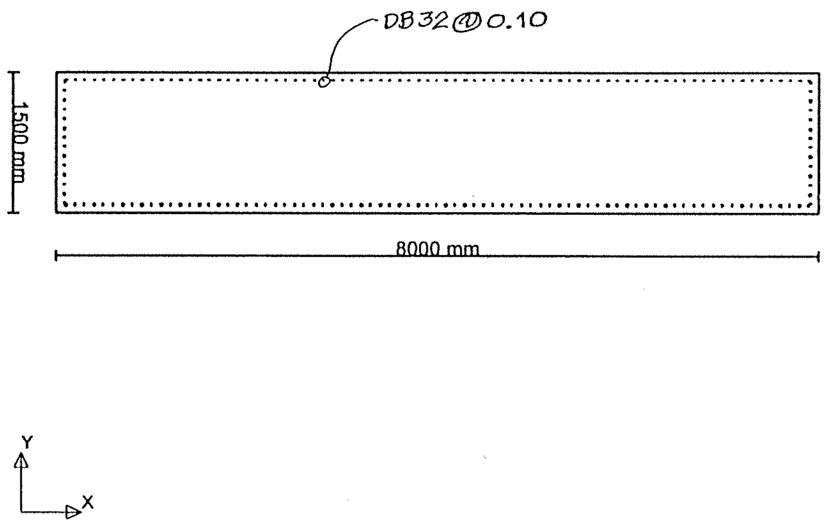
$M_{\gamma t3} := 100\,\%\cdot\dfrac{M_{t3}}{R_t}$ $M_{\gamma t3} = 31746$ kN-m

Maximum biaxial moments of Pier 2 is critical both for Load case 1 & 2, therefore, capacity of piers are checked by using computer program NovaDesign. Moment capacity (see Fig.) is OK for the given cross section.



+ Wadi Himara Bridge
× Wadi Abu EL-Asal Bridge

Fig. 3.2 Pier biaxial moment capacity



Section:

No. Name
1.0 Rectangular section

Concrete:
fck = 30 MPa
Rho = 2500 kg/m3

Section properties:
Ac = 12.000000 m2
Ix = 2.250000 m4
Iy = 64.000000 m4
It = 7.948125 m4

Center of gravity:
CoGx = 0.000 m
CoGy = 0.000 m

Total reinforcement area:
As = 149590 mm2

Total prestressing area:
Asp = 0 mm2

Fig. 3.3 Pier cross-section and reinforcement

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AASHTO 7.3.1 Minimum Support Length Requirements for Seismic Perfomance Category C:

$N := (305 + 2.5 \cdot L + 10\,H) \cdot \left(1 + 0.000125\,S^2\right)$ mm

Where: L = length of the bridge deck to the adjacent expansion joint, m

H : for abutments, H is the average height of columns supporting the bridge deck to the next expansion joint.
for piers, H is the pier height, m.

S = angle of skew of support in degrees measured from a line normal to the span.

..ADI HIMARA BRIDGE:

$L := 30$ m $S := 0$

For Abuments:

$H := 29.20$ m

$N := (305 + 2.5 \cdot L + 10\,H) \cdot \left(1 + 0.000125\,S^2\right)$ $N = 672$ mm < 750 mm OK

For Piers:

$H := 30.35$ m

$N := (305 + 2.5 \cdot L + 10\,H) \cdot \left(1 + 0.000125\,S^2\right)$ $N = 683.5$ mm < $750 - 2.75 = 747.25$ mm OK

WADI ABU EL-ASAL BRIDGE:

For Abuments:

$H := 23.70$ m

$N := (305 + 2.5 \cdot L + 10\,H) \cdot \left(1 + 0.000125\,S^2\right)$ $N = 617$ mm < 750 mm OK

For Piers:

$H := 23.70$ m

$N := (305 + 2.5 \cdot L + 10\,H) \cdot \left(1 + 0.000125\,S^2\right)$ $N = 617$ mm < $750 - 2.75 = 747.25$ mm OK

Conclusion: support lengths are OK for both bridges.

PROJECT : Design Bridges in Jordan

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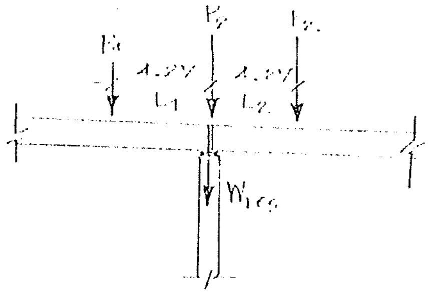
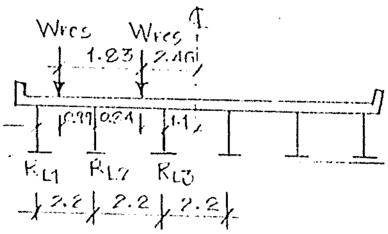
CROSS BEAM DESIGN

- Concrete cylinder strength (Class 30): $f_{c'} := 26.1$ MPa
- Concrete Density: $\gamma_c := 25$ kN/m³
- Reinforcement yield strength : $f_y := 410$ MPa
- Span length: $L_{span} := 30$ m
- Girder spacing: $G_{sp} := 2.20$ m



- Calculate resultant wheel loads on the cross beam:
- HS20-44 Truck loading: $P_1 := 18$ kN $P_2 := 72$ kN
  $L_1 := 4.27$ m $L_2 := 4.27$ m
- Impact factor: $I := 0.226$
- Percent incease in LL: $P_{inc} := 50\ \%$
- Wheel load: $W_{wheel} := P_2 + \left(\dfrac{L_{span} - L_1}{L_{span}}\right)\cdot(P_1 + P_2)$ $W_{wheel} = 149.19$ kN
- Resultant wheel load: $W_{res} := (1 + I)\cdot(1 + P_{inc})\cdot W_{wheel}$ $W_{res} = 274.36$ kN
- Reactions due to LL on cross beam:

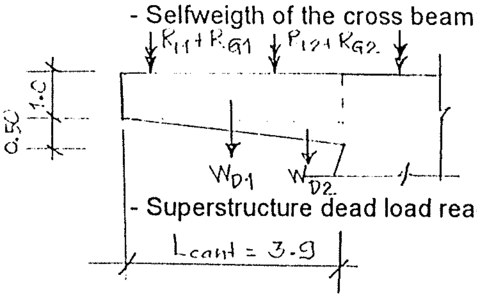


$R_{L1} := \dfrac{0.99}{2.2}\cdot W_{res}$ $R_{L1} = 123.462$ kN

$R_{L2} := \dfrac{1.21 + 1.36}{2.2}\cdot W_{res}$ $R_{L2} = 320.503$ kN

$R_{L3} := \dfrac{0.84}{2.2}\cdot W_{res}$ $R_{L3} = 104.756$ kN

- Selfweigth of the cross beam:



$W_{D1} := 1.0\cdot 1.5\cdot 3.9\cdot\gamma_c$ $W_{D1} = 146.25$ kN

$W_{D2} := \left(\dfrac{1}{2}\cdot 0.5\cdot 3.9\right)\cdot 1.5\cdot\gamma_c$ $W_{D2} = 36.563$ kN

- Superstructure dead load reactions:

$R_g := 644$ kN

$R_{G1} := 2\cdot R_g$ $R_{G1} = 1.288\cdot 10^3$ kN

$R_{G2} := 2\cdot R_g$ $R_{G2} = 1.288\cdot 10^3$ kN

- Length of cantilever part of the cross beam: $L_{cant} := 3.90$ m
- Distance from c/c of Girder# 2 to assumed fixed end: $D_{g2} := 0.80$ m

- Moment due to dead loads: $M_{DL} := R_{G1} \cdot (G_{sp} + D_{g2}) + R_{G2} \cdot D_{g2} + W_{D1} \cdot \frac{L_{cant}}{2} + W_{D2} \cdot \frac{L_{cant}}{3}$

$M_{DL} = 5227$ kN-m

- Moment due to live loads: $M_{LL} := R_{L1} \cdot (G_{sp} + D_{g2}) + R_{L2} \cdot D_{g2}$

$M_{LL} = 627$ kN-m

Load Combinations:

Group I: $Factored\_Load := \gamma \cdot [\beta_D \cdot D + \beta_L \cdot (L + I)]$

Where : $\gamma := 1.3$ $\beta_D := 1.0$ $\beta_L := 1.67$

Design Moment:

Therefore, design ultimate moment is:

$$M_\gamma := \gamma \cdot \left[\beta_D \cdot (M_{DL}) + \beta_L \cdot (M_{LL})\right]$$

$M_\gamma = 8156$ kN-m

Reinforcement Design:

Maximum depth of cross beam: $D_{cb} := 1.50$ m

Width of the cross beam: $b := 1.50$ m

- Select reinforcement:

Bar diameter : $\phi_b := 32$ mm

Cross section area : $A_b := \frac{\pi \cdot \phi_b^2}{4}$ $A_b = 804.248$ mm^2

From Vol. (III) Design Drawings,
General Notes: minimum concrete cover for pier is 70 mm.

$cov := 70$ mm.

Effective depth: $d := D_{cb} - \frac{cov}{1000} - \frac{(1.4 + 3.2 + 6)}{100}$ $d = 1.324$ m.

Number of rebars: $No := 24$

Total reinforcement area: $A_s := No \cdot A_b$ $A_s = 1.930 \cdot 10^4$ mm^2

For flexural design: $\phi := 0.90$

$a := \frac{A_s \cdot f_y \cdot 10^{-6}}{0.85 \cdot f_{c'} \cdot b}$ $a = 0.238$ m

$$M_u := \left[\phi \cdot A_s \cdot f_y \cdot \left(d - \frac{a}{2}\right)\right] \cdot 10^{-3}$$

$M_u = 8583$ kN-m > $M_\gamma = 8156$ kN-m OK

USE 24-DB32 (3 layers: 8 in each layer) for cross beam reinforcement:

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STABILITY CHECK (Group I Loading)

- Allowable bearing stress of the bed rock: $q_{all} := 500$ kN/m^2
- Width of footing: $B_{ft} := 10$ m
- Length of footing: $L_{ft} := 10$ m
- Distance of first wheel from CL of bridge: $D_{w1} := 2.46$ m
- Distance between wheels: $D_{ww} := 1.83$ m
- Centroid of resultant wheel loads from CL bridge: $D_w := \dfrac{\left[D_{w1} + \left(D_{w1} + D_{ww}\right)\right] \cdot W_{res}}{2 \cdot W_{res}}$

  $D_w = 3.375$ m
- Total live loads: $W_{LL} := 2 \cdot W_{res}$ $W_{LL} = 548.721$ kN
- Total dead loads from superstructure: $W_{DL} := 2 \cdot 6 \cdot R_g$ $W_{DL} = 7.728 \cdot 10^3$ kN
- Pier weight:

$W_1 := 1.2 \cdot 12.8 \cdot 1.5 \cdot \gamma_c$ $W_1 = 576$ kN

$W_2 := \left(\dfrac{12.8 + 5}{2}\right) \cdot 1.0 \cdot 1.5 \cdot \gamma_c$ $W_2 = 333.75$ kN

$W_3 := 8 \cdot 28.15 \cdot 1.5 \cdot \gamma_c$ $W_3 = 8445$ kN

$W_4 := 10 \cdot 10 \cdot 1.3 \cdot \gamma_c$ $W_4 = 3250$ kN

- Total pier & footing weight: $W_p := W_1 + W_2 + W_3 + W_4$ $W_p = 12605$ kN
- Total vertical loads: $W_{vert} := W_{DL} + W_{LL} + W_p$ $W_{vert} = 20882$ kN
- Overturning moment: $M_o := W_{LL} \cdot D_w$ $M_o = 1852$ kN-m
- Righting moment: $M_R := W_{vert} \cdot \dfrac{B_{ft}}{2}$ $M_R = 104410$ kN-m
- Since $M_R >>> M_o$ OK
- Checking stresses at footing base:

$$q_{max} := \frac{W_{vert}}{B_{ft} \cdot L_{ft}} + \frac{M_o \cdot L_{ft}}{\frac{1}{12} \cdot B_{ft} \cdot L_{ft}^3} \qquad q_{max} = 231 \text{ kN/m^2} < q_{all} = 500 \text{ kN/m^2} \quad \text{OK}$$

4. Rock Bearing Capacity Estimation

PROJECT : Design Bridges in Jordan

SUBJECT : Rock Bearing Capacity

4. Rock Bearing Capacity Estimation:

Service Load Design Method:

AASHTO 4.4.8.1.2 Footings on Broken or Jointed Rock

- The ultimate bearing capacity of footings on broken or jointed rock may be estimated using:

$$q_{ult} := N_{ms} \cdot C_o$$

- Where: N_{ms} = Coefficient obtained from Table 4.4.8.1.2A

C_o = Uniaxial compressive strength of intact rock (ksf)

- Assumption:

  - Rock Mass Quality is Good (Fresh to slightly weathered rock, slightly disturbed with joints spaced 3 to 10 feet apart).

  - Rock Category C (Arenaceous rocks with strong crystals and poor cleavage).

  - Rock Type is Sandstone.

- From Table 4.4.8.1.2B, we have

C_o = 1400-3600 ksf

$C_{o\_min} := 1400$ ksf

$C_{o\_max} := 3600$ ksf

- From Table 4.4.8.1.2A, we have

$N_{ms} := 0.38$

- Therefore:

$q_{ult\_min} := C_{o\_min} \cdot N_{ms}$ $\qquad q_{ult\_min} = 532$ ksf

$q_{ult\_min} := q_{ult\_min} \cdot \dfrac{1000 \cdot 3.28^2}{2.204 \cdot 1000}$ $\qquad q_{ult\_min} = 2597$ $\dfrac{\text{ton}}{\text{m}^2}$

$q_{ult\_max} := C_{o\_max} \cdot N_{ms}$ $\qquad q_{ult\_max} = 1368$ ksf

$q_{ult\_max} := q_{ult\_max} \cdot \dfrac{1000 \cdot 3.28^2}{2.204 \cdot 1000}$ $\qquad q_{ult\_max} = 6678$ $\dfrac{\text{ton}}{\text{m}^2}$

- AASHTO 4.4.8.1.3: specifying minimum factor of safety (FS) of 3.0 against a bearing capacity failure

$FS := 3.0$

- Allowable rock bearing capacity is therefore:

$q_{a\_min} := \frac{q_{ult\_min}}{FS}$ $\quad q_{a\_min} = 866$ t/m^2 >>> 50 t/m^2 OK

$q_{a\_max} := \frac{q_{ult\_max}}{FS}$ $\quad q_{a\_max} = 2226$ t/m^2 >>> 50 t/m^2 OK

- Allowable bearing stress of concrete footing:

$f_{c'} := 20$ Mpa

$f_b := 0.3 \cdot f_{c'}$ $\quad f_b = 6$ MPa or $f_b := 600$ t/m^2 >>> 50 t/m^2 OK

Load Factor Design Method:

- AASHTO 4.11.4.2 Bearing Capacity of Foundations on Rock

- Assumption:

  - Type of bearing material: Sedimentary rock - hard cemented shales, siltstone, **sandstone**, limestone without cavities.

  - Consistency in Place: Hard sound rock

- From Table 4.11.4.1.4-1 Presumptive Allowable Bearing Pressures for Spread Footing Foundations

- Recommended allowable bearing pressure is

$q_a := 20$ tsf or

$q_a := 20 \cdot \frac{2000}{2.204 \cdot 1000} \cdot 3.28^2$ $\quad q_a = 195$ t/m^2 or approximate 1950 kN/m^2

Or approximate q_a = 1950 kN/m^2 >>> 500 kN/m^2 OK

- In conclusion, the value of bearing capacity used in design which equals to 500 kN/m^2 is OK and conservative.

should be used to determine q_{ult}. Alternatively, Table 4.4.8.1.2B may be used as a guide to estimate C_o. For rocks defined by very poor quality, the value of q_{ult} should be determined as the value of q_{ult} for an equivalent soil mass.

4.4.8.1.3 Factors of Safety

Spread footings on rock shall be designed for Group 1 loadings using a minimum factor of safety (FS) of 3.0 against a bearing capacity failure.

4.4.8.2 Settlement

4.4.8.2.1 Footings on Competent Rock

For footings on competent rock, elastic settlements will generally be less than ½ inch when footings are designed in accordance with Article 4.4.8.1.1. When elastic settlements of this magnitude are unacceptable or when the rock is not competent, an analysis of settlement based on rock mass characteristics must be made. For rock masses which have time-dependent settlement characteristics, the procedure in Article 4.4.7.2.3 may be followed to determine the time-dependent component of settlement.

4.4.8.2.2 Footings on Broken or Jointed Rock

Where the criteria for competent rock are not met, the influence of rock type, condition of discontinuities and degree of weathering shall be considered in the settlement analysis.

The elastic settlement of footings on broken or jointed rock may be determined using the following:

- For circular (or square) footings;

$$\rho = q_o(1 - \nu^2)rI_\rho/E_m, \text{ with } I_\rho = (\sqrt{\pi})/\beta_z \quad (4.4.8.2.2\text{-}1)$$

- For rectangular footings;

TABLE 4.4.8.1.2A Values of Coefficient N_{ms} for Estimation of the Ultimate Bearing Capacity of Footings on Broken or Jointed Rock (Modified after Hoek, (1983))

Rock Mass Quality	General Description	RMR[1] Rating	NGI[2] Rating	RQD[3] (%)	N_{ms}[4] A	B	C	D	E
Excellent	Intact rock with joints spaced > 10 feet apart	100	500	95-100	3.8	4.3	5.0	5.2	6.1
Very good	Tightly interlocking, undisturbed rock with rough unweathered joints spaced 3 to 10 feet apart	85	100	90-95	1.4	1.6	1.9	2.0	2.3
Good	Fresh to slightly weathered rock, slightly disturbed with joints spaced 3 to 10 feet apart	65	10	75-90	0.28	0.32	0.38	0.40	0.46
Fair	Rock with several sets of moderately weathered joints spaced 1 to 3 feet apart	44	1	50-75	0.049	0.056	0.066	0.069	0.081
Poor	Rock with numerous weathered joints spaced 1 to 20 inches apart with some gouge	23	0.1	25-50	0.015	0.016	0.019	0.020	0.024
Very poor	Rock with numerous highly weathered joints spaced < 2 inches apart	3	0.01	<25	Use q_{ult} for an equivalent soil mass				

[1] Geomechanics Rock Mass Rating (RMQ) System—Bieniawski, 1988.
[2] Norwegian Geotechnical Institute (NGI) Rock Mass Classification System, Barton, et al., 1974.
[3] Range of RQD values provided for general guidance only; actual determination of rock mass quality should be based on RMR or NGI rating systems.
[4] Value of N_{ms} as a function of rock type; refer to Table 4.4.8.1.2B for typical range of values of C_o for different rock type in each category.

TABLE 4.4.8.1.2B Typical Range of Uniaxial Compressive Strength (C_o) as a Function of Rock Category and Rock Type

Rock Category	General Description	Rock Type	C_o[(1)] (ksf)	C_o[(1)] (psi)
A	Carbonate rocks with well-developed crystal cleavage	Dolostone	700- 6,500	4,800-45,000
		Limestone	500- 6,000	3,500-42,000
		Carbonatite	800- 1,500	5,500-10,000
		Marble	800- 5,000	5,500-35,000
		Tactite-Skarn	2,700- 7,000	19,000-49,000
B	Lithified argillaceous rock	Argillite	600- 3,000	4,200-21,000
		Claystone	30- 170	200- 1,200
		Marlstone	1,000- 4,000	7,600-28,000
		Phyllite	500- 5,000	3,500-35,000
		Siltstone	200- 2,500	1,400-17,000
		Shale[(2)]	150- 740	1,000- 5,100
		Slate	3,000- 4,400	21,000-30,000
C	Arenaceous rocks with strong crystals and poor cleavage	Conglomerate	700- 4,600	4,800-32,000
		Sandstone	1,400- 3,600	9,700-25,000
		Quartzite	1,300- 8,000	9,000-55,000
D	Fine-grained igneous crystalline rock	Andesite	2,100- 3,800	14,000-26,000
		Diabase	450-12,000	3,100-83,000
E	Coarse-grained igneous and metamorphic crystalline rock	Amphibolite	2,500- 5,800	17,000-40,000
		Gabbro	2,600- 6,500	18,000-45,000
		Gneiss	500- 6,500	3,500-45,000
		Granite	300- 7,000	2,100-49,000
		Quartzdiorite	200- 2,100	1,400-14,000
		Quartzmonzonite	2,700- 3,300	19,000-23,000
		Schist	200- 3,000	1,400-21,000
		Syenite	3,800- 9,000	26,000-62,000

(1)Range of Uniaxial Compressive Strength values reported by various investigations.
(2)Not including oil shale.

$$\rho = q_o(1 - \nu^2)BI_p/E_m, \text{ with } I_p = (L/B)^{1/2}/\beta_z \quad (4.4.8.2.2\text{-}2)$$

Values of I_p may be computed using the β_z values presented in Table 4.4.7.2.2B from Article 4.4.7.2.2 for rigid footings. Values of Poisson's ratio (υ) for typical rock types are presented in Table 4.4.8.2.2A. Determination of the rock mass modulus (E_m) should be based on the results of in-situ and laboratory tests. Alternatively, values of E_m may be estimated by multiplying the intact rock modulus (E_o) obtained from uniaxial compression tests by a reduction factor (α_E) which accounts for frequency of discontinuities by the rock quality designation (RQD), using the following relationships (Gardner, 1987):

$$E_m = \alpha_E E_o \quad (4.4.8.2.2\text{-}3)$$

$$\alpha_E = 0.0231(RQD) - 1.32 \geq 0.15 \quad (4.4.8.2.2\text{-}4)$$

For preliminary design or when site-specific test data cannot be obtained, guidelines for estimating values of E_o (such as presented in Table 4.4.8.2.2B or Figure 4.4.8.2.2A) may be used. For preliminary analyses or for final design when in-situ test results are not available, a value of $\alpha_E = 0.15$ should be used to estimate E_m.

4.4.8.2.3 Tolerable Movement

Refer to Article 4.4.7.2.3.

4.4.9 Overall Stability

The overall stability of footings, slopes, and foundation soil or rock shall be evaluated for footings located on

TABLE 4.11.4.1.4-1 Presumptive Allowable Bearing Pressures for Spread Footing Foundations (Modified after U.S. Department of the Navy, 1982)

1 t = 2000 lbs.

Type of Bearing Material	Consistency in Place	Allowable Bearing Pressure (tsf): Ordinary Range	Allowable Bearing Pressure (tsf): Recommended Value for Use
Massive crystalline igneous and metamorphic rock: graphite, diorite, basalt, gneiss, thoroughly cemented conglomerate (sound condition allows minor cracks)	Very hard, sound rock	60 to 100	80
Foliated metamorphic rock: slate, schist (sound condition allows minor cracks)	Hard sound rock	30 to 40	35
Sedimentary rock: hard cemented shales, siltstone, sandstone, limestone without cavities	Hard sound rock	15 to 25	20 ≈ 195 t/m²
Weathered or broken bedrock of any kind except highly argillacous rock (shale)	Medium hard rock	8 to 12	10 ≈ 98 t/m²
Compaction shale or other highly argillacous rock in sound condition	Medium hard rock	8 to 12	10
Well-graded mixture of fine- and coarse-grained soil: glacial till, hardpan, boulder clay (GW-GC, GC, SC)	Very dense	8 to 12	10
Gravel, gravel-sand mixtures, boulder-gravel mixtures (GW, GP, SW, SP)	Very dense	6 to 10	7
	Medium dense to dense	4 to 7	5
	Loose	2 to 6	3
Coarse to medium sand, sand with little gravel (SW, SP)	Very dense	4 to 6	4
	Medium dense to dense	2 to 4	3
	Loose	1 to 3	1.5
Fine to medium sand, silty or clayey medium to coarse sand (SW, SM, SC)	Very dense	3 to 5	3
	Medium dense to dense	2 to 4	2.5
	Loose	1 to 2	1.5
Find sand, silty or clayey medium to fine sand (SP, SM, SC)	Very dense	3 to 5	3
	Medium dense to dense	2 to 4	2.5
	Loose	1 to 2	1.5
Homogeneous inorganic clay, sandy or silty clay (CL, CH)	Very stiff to hard	3 to 6	4
	Medium stiff to stiff	1 to 3	2
	Soft	0.5 to 1	0.5
Inorganic silt, sandy or clayey silt, varved silt-clay-fine sand (ML, MH)	Very stiff to hard	2 to 4	3
	Medium stiff to stiff	1 to 3	1.5
	Soft	0.5 to 1	0.5

STRUCTURAL CALCULATIONS

RETAINING WALLS

DEAD SEA PARKWAY

C14

Retaining Wall Design : Height = 5.0 m

Input Data

Seepage allowed

Theory : Coulomb
Wall type : Cantilever

SEISMIC ANALYSIS SETTINGS:

Seismic Analysis ON/OFF:ON

VALUES OF PRESSURE COEFFICIENTS:

Active Pressure coefficient Ka :0.32
Passive Pressure coefficient Kp :4.98
Seismic Active Pressure coefficient Kas :0.45
Seismic Passive Pressure coefficient Kps :2.04
Base frictional constant μ :0.58

FORCES ACTING ON THE WALL:

Description	FORCES (kN) and their LEVER ARMS (m) F Horizontal left (+)	Lever arm	F Vertical down (+)	Lever arm
Destabilizing forces:				
Total Active pressure Pa	141.305	2.431	43.001	0.947
Siesmic component of Pa	40.049	3.600	12.453	0.900
As a result of surcharge w	39.414	3.000	12.255	0.955
Siesmic wall inertia	22.260	3.450		
Stabilizing forces:				
Passive pressure on base Pp	-14.891	0.300		
Siesmic component of Pp	21.388	0.540		
Weight of the wall + base			123.322	1.342
Weight of soil on the base			243.309	2.225
UDL of 16.2 kN/m2			42.322	2.175

EQUILIBRIUM CALCULATIONS AT SLS

1.Moment Equilibrium

Point of rotation: bottom front corner of base.

For Overturning moment Mo calculate as follows:
Mo = Sum(hor. forces x l.a.) - Sum(vert. forces x l.a.)
For Stabilizing moment Mr calculate as follows:
Mr = -Sum(hor. forces x l.a.) + Sum(vert. forces x l.a.)
where l.a. = lever arm of each force

```
        Stabilizing moment Mr : 803.2 kNm
      Destabilizing moment Mo : 486.1 kNm

Safety factor against overturning = Mr/Mo = 1.65

2.Force Equilibrium

    Sum of Vertical forces Pv : 464.2 kN
  Frictional resistance Pfric : 268.0 kN
Passive Pressure on shear key :  32.2 kN
     Passive pressure on base :  14.9 kN
  => Horizontal resistance Fr : 315.1 kN
  Horizontal sliding force Fh : 203.0 kN

Safety factor against overall sliding = Fr/Fh = 1.55
```

SOIL PRESSURES UNDER BASE

Maximum pressure :453.0 kPa
Minimum pressure : 0 kPa at 1.5 m from right hand side of base.
Maximum pressure occurs at left hand side of base

***** MAXIMUM PRESSURE EXCEEDS SPECIFIED ALLOWABLE PRESSURE BY 53.0 kPa *****

WALL MOMENTS (ULS) AND REINFORCEMENT TO BS8110 - 1997

Position from base top (m)	Moment (kNm)	Front Reinforcing (mm²/m)	Back Reinforcing (mm²/m)	Nominal (0.13%) (mm²/m)
0.00	623.62	0.00	3323.65	721.50
0.13	596.27	0.00	3210.34	715.00
0.25	566.05	0.00	3079.03	708.50
0.38	536.71	0.00	2949.85	702.00
0.50	508.28	0.00	2823.00	695.50
0.63	480.75	0.00	2698.52	689.00
0.76	454.11	0.00	2576.40	682.50
0.88	428.36	0.00	2456.69	676.00
1.01	403.47	0.00	2339.39	669.50
1.13	379.44	0.00	2224.53	663.00
1.26	356.27	0.00	2112.12	656.50
1.39	333.93	0.00	2002.20	650.00
1.51	312.43	0.00	1894.79	643.50
1.64	291.74	0.00	1789.91	637.00
1.76	271.86	0.00	1687.59	630.50
1.89	252.79	0.00	1587.85	624.00
2.02	234.50	0.00	1490.73	617.50
2.14	216.99	0.00	1396.25	611.00
2.27	200.25	0.00	1304.45	604.50
2.39	184.27	0.00	1215.37	598.00
2.52	169.04	0.00	1129.03	591.50
2.65	154.55	0.00	1045.47	585.00
2.77	140.79	0.00	964.74	578.50
2.90	127.74	0.00	886.87	572.00
3.02	115.41	0.00	811.91	565.50
3.15	103.77	0.00	739.90	559.00
3.28	92.82	0.00	670.89	552.50
3.40	82.55	0.00	604.94	546.00
3.53	72.94	0.00	542.09	539.50
3.65	64.00	0.00	482.40	533.00
3.78	55.70	0.00	425.93	526.50
3.91	48.04	0.00	372.75	520.00
4.03	41.01	0.00	322.93	513.50
4.16	34.59	0.00	276.53	507.00
4.28	28.78	0.00	233.62	500.50
4.41	23.57	0.00	194.30	494.00
4.54	18.94	0.00	158.65	487.50
4.66	14.89	0.00	126.76	481.00

4.91	8.49	0.00	74.64	468.00
5.04	6.11	0.00	54.62	461.50
5.17	4.27	0.00	38.80	455.00
5.29	2.95	0.00	27.29	448.50
5.42	2.14	0.00	20.19	442.00
5.54	1.58	0.00	15.15	435.50
5.67	1.10	0.00	10.76	429.00
5.80	0.71	0.00	7.06	422.50
5.92	0.40	0.00	4.10	416.00
6.05	0.18	0.00	1.90	409.50
6.17	0.05	0.00	0.53	403.00
6.30	0.00	0.00	0.02	396.50

BASE MOMENTS (ULS) AND REINFORCEMENT TO BS8110 - 1997

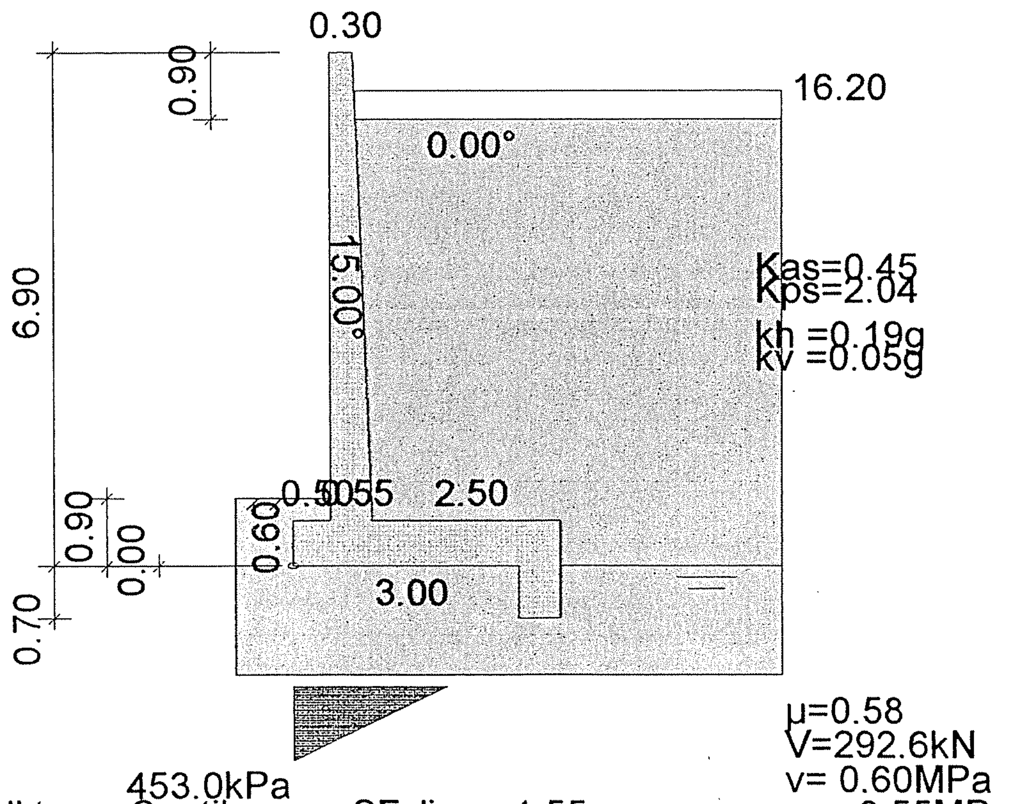
Position from left (m)	Moment (kNm)	Top Reinforcing (mm^2/m)	Bot Reinforcing (mm^2/m)	Nominal (0.13%) (mm^2/m)
0.07	-0.00	0.00	0.00	780.00
0.14	-1.87	0.00	9.13	780.00
0.21	-7.48	0.00	36.53	780.00
0.28	-16.83	0.00	82.20	780.00
0.35	-29.91	0.00	146.14	780.00
0.43	-46.74	0.00	228.34	780.00
0.50	-67.30	0.00	328.81	780.00
0.57	-91.61	0.00	447.55	780.00
0.64	-119.65	0.00	584.55	780.00
0.71	-151.43	0.00	739.82	780.00
0.78	-186.95	0.00	913.36	780.00
0.85	-222.75	1088.25	0.00	780.00
0.92	560.43	2738.00	0.00	780.00
0.99	558.01	2726.17	0.00	780.00
1.06	529.76	2588.16	0.00	780.00
1.14	502.25	2453.73	0.00	780.00
1.21	475.47	2322.89	0.00	780.00
1.28	449.42	2195.63	0.00	780.00
1.35	424.10	2071.96	0.00	780.00
1.42	399.52	1951.87	0.00	780.00
1.49	375.68	1835.37	0.00	780.00
1.56	352.56	1722.45	0.00	780.00
1.63	330.18	1613.12	0.00	780.00
1.70	308.54	1507.37	0.00	780.00
1.77	287.63	1405.21	0.00	780.00
1.85	267.45	1306.63	0.00	780.00
1.92	248.01	1211.63	0.00	780.00
1.99	229.29	1120.22	0.00	780.00
2.06	211.32	1032.40	0.00	780.00
2.13	194.07	948.15	0.00	780.00
2.20	177.57	867.50	0.00	780.00
2.27	161.79	790.43	0.00	780.00
2.34	146.75	716.94	0.00	780.00
2.41	132.44	647.04	0.00	780.00
2.48	118.87	580.72	0.00	780.00
2.56	106.03	517.99	0.00	780.00
2.63	93.92	458.84	0.00	780.00
2.70	82.55	403.28	0.00	780.00
2.77	71.91	351.30	0.00	780.00
2.84	62.00	302.91	0.00	780.00
2.91	52.83	258.10	0.00	780.00
2.98	44.39	216.87	0.00	780.00
3.05	36.69	179.24	0.00	780.00
3.12	29.72	145.18	0.00	780.00
3.19	23.48	114.71	0.00	780.00
3.27	17.98	87.83	0.00	780.00
3.34	13.21	64.52	0.00	780.00
3.41	9.17	44.81	0.00	780.00
3.48	5.87	28.68	0.00	780.00
3.55	3.30	16.13	0.00	780.00

SHEAR CHECK AT WALL-BASE JUNCTION TO BS8110 - 1997

```
 Shear force at bottom of wall V = 292.6 kN
Shear stress at bottom of wall v = 0.60 MPa INADEQUATE
       Allowable shear stress vc = 0.55 MPa (based on Wall tensile reinf.)
```

Sketch of Wall

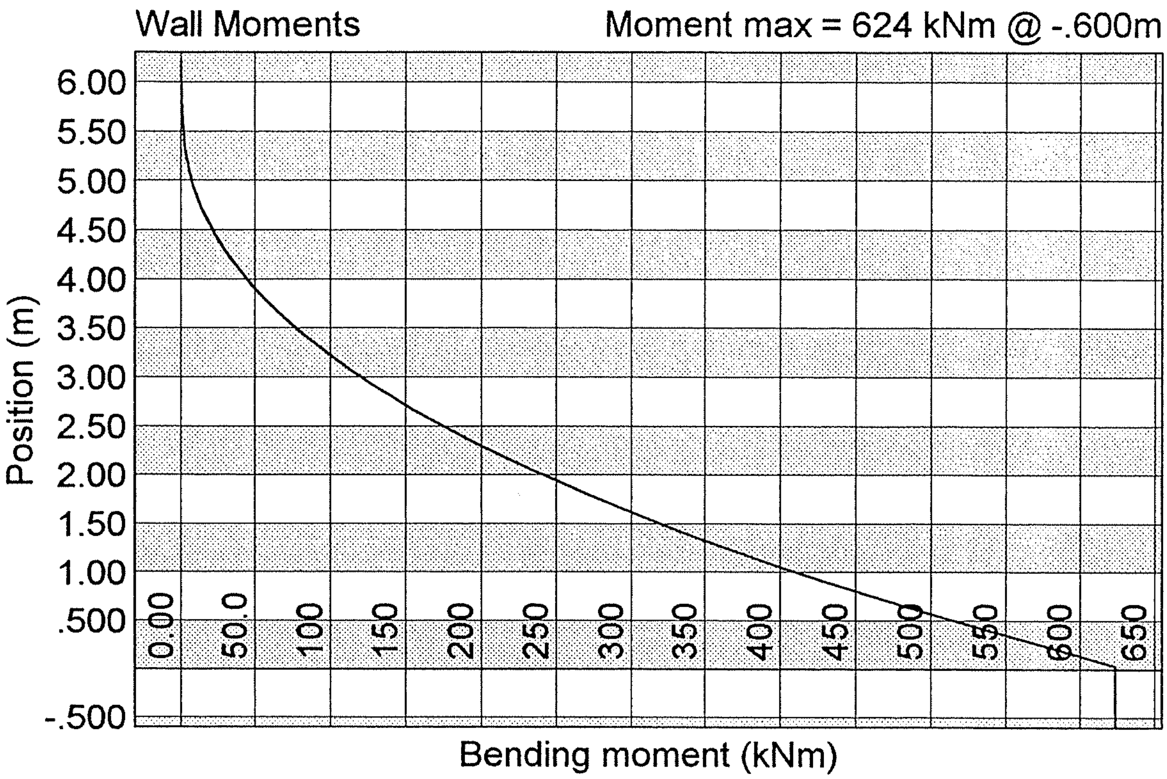
Design code: BS8110 - 1997



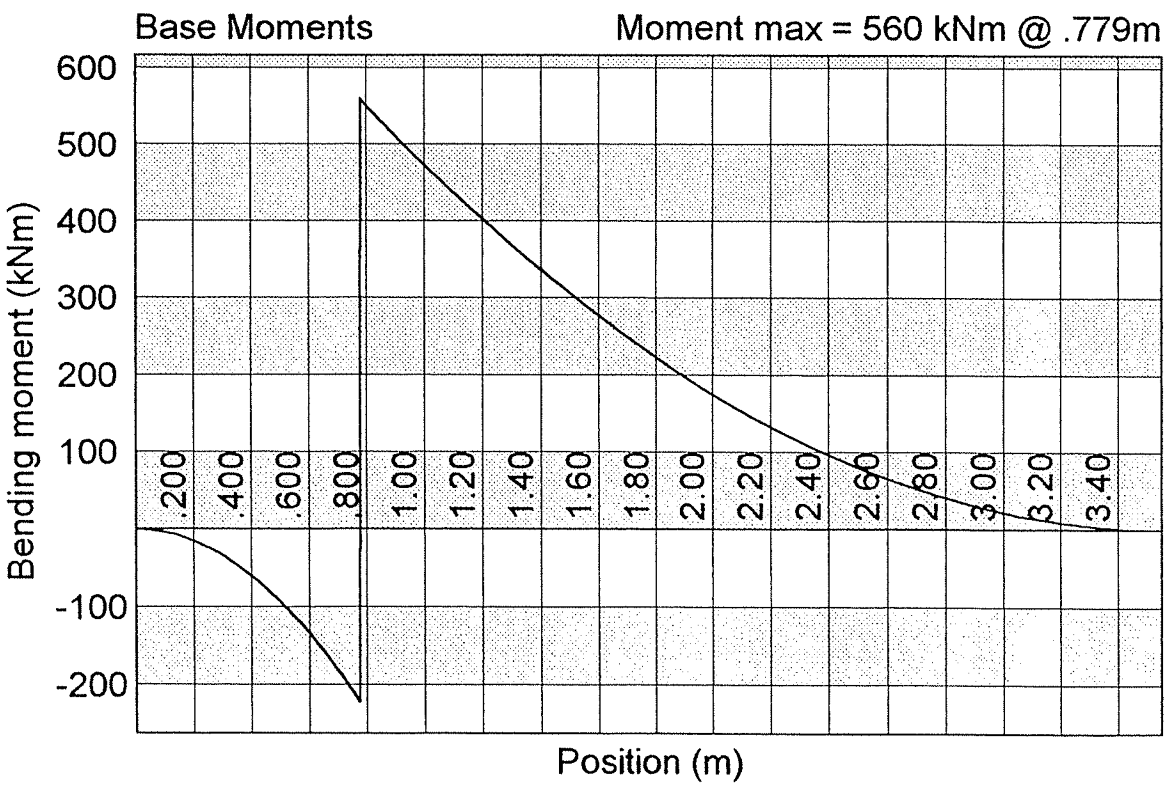
Wall type: Cantilever SFslip = 1.55
Theory: Coulomb SFovt = 1.65

PROKON Software Consultants (Pty) Ltd Internet: http://www.prokon.com E-Mail : mail@prokon.com	Job Number 12132		Sheet 11
	Job Title Dead Sea Parkway - JICA		
	Client Ministry of Tourism		
	Calcs by N.O	Checked by	Date FEB,2000

Wall Bending Moments



Base Bending Moments



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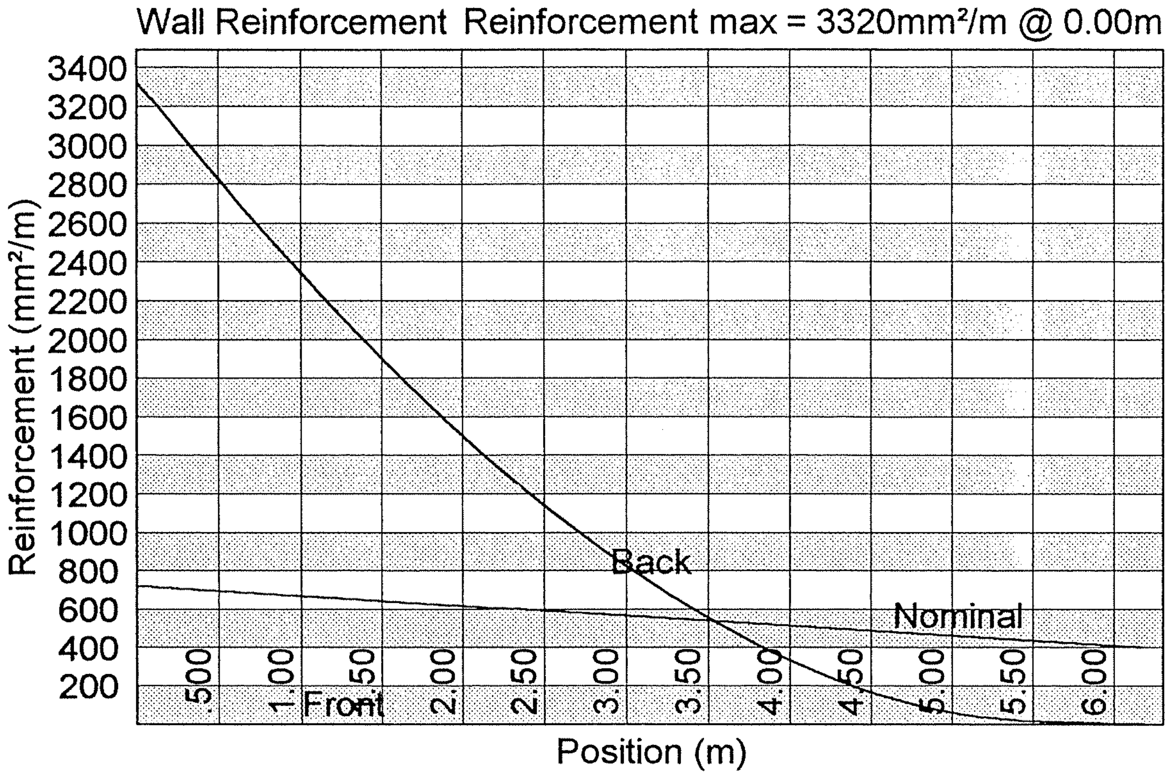
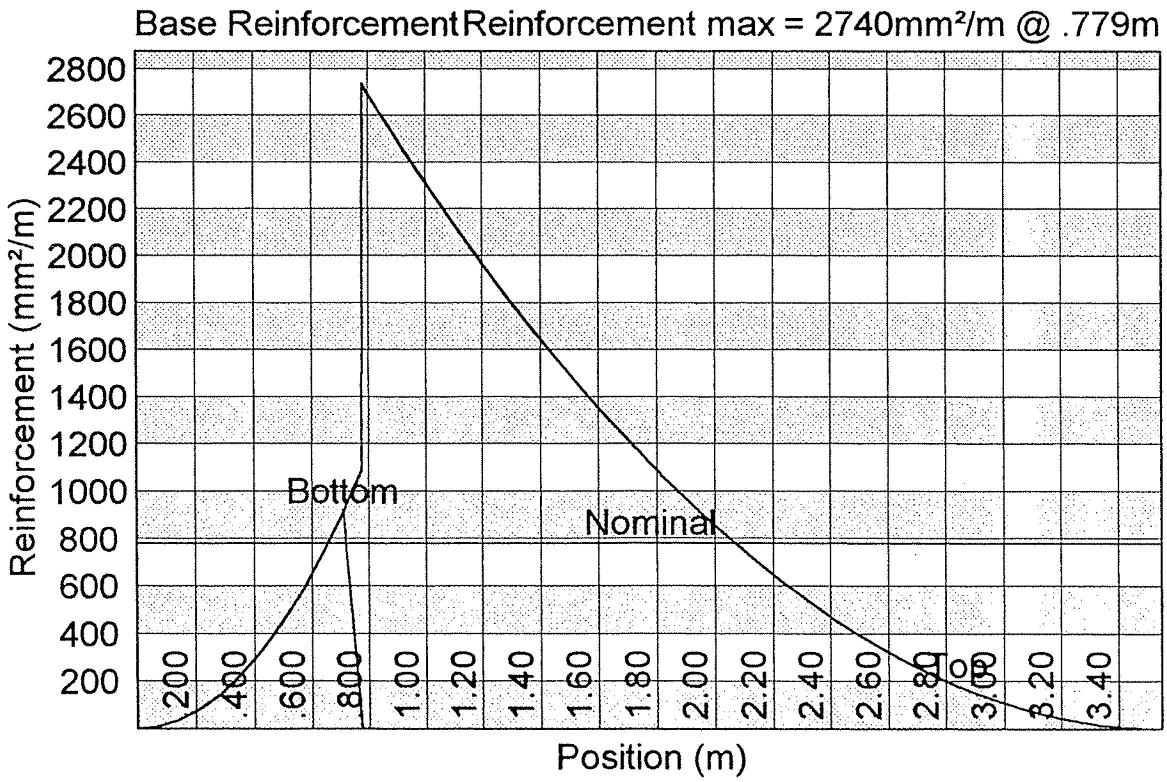
	Job Title Dead Sea Parkway - JICA		
	Client Ministry of Tourism		
	Calcs by N.O	Checked by	Date FEB,2000

Wall Reinforcement



Base Reinforcement



Retaining Wall Design : Height = 5.5 m

C14

Input Data

Seepage allowed

Theory : Coulomb
Wall type : Cantilever

SEISMIC ANALYSIS SETTINGS:

Seismic Analysis ON/OFF:ON

VALUES OF PRESSURE COEFFICIENTS:

Active Pressure coefficient Ka :0.31
Passive Pressure coefficient Kp :4.98
Seismic Active Pressure coefficient Kas :0.44
Seismic Passive Pressure coefficient Kps :2.04
Base frictional constant μ :0.58

FORCES ACTING ON THE WALL:

FORCES (kN) and their LEVER ARMS (m)

Description	F Horizontal left (+)	Lever arm	F Vertical down (+)	Lever arm
Destabilizing forces:				
Total Active pressure Pa	171.695	2.701	50.455	0.918
Siesmic component of Pa	49.202	3.990	14.724	0.880
As a result of surcharge w	43.354	3.325	12.974	0.922
Siesmic wall inertia	24.478	3.775		
Stabilizing forces:				
Passive pressure on base Pp	-18.384	0.333		
Siesmic component of Pp	26.405	0.600		
Weight of the wall + base			135.612	1.481
Weight of soil on the base			319.714	2.435
UDL of 16.2 kN/m2			49.248	2.400

EQUILIBRIUM CALCULATIONS AT SLS

1.Moment Equilibrium

Point of rotation: bottom front corner of base.

For Overturning moment Mo calculate as follows:
  Mo = Sum(hor. forces x l.a.) - Sum(vert. forces x l.a.)
For Stabilizing moment Mr calculate as follows:
  Mr = -Sum(hor. forces x l.a.) + Sum(vert. forces x l.a.)
where l.a. = lever arm of each force

Stabilizing moment Mr : 1103.8 kNm
Destabilizing moment Mo : 642.1 kNm

Safety factor against overturning = Mr/Mo = 1.72

2.Force Equilibrium

Sum of Vertical forces Pv : 568.0 kN
Frictional resistance Pfric : 327.9 kN
Passive Pressure on shear key : 34.7 kN
Passive pressure on base : 18.4 kN
=> Horizontal resistance Fr : 381.1 kN
Horizontal sliding force Fh : 239.5 kN

Safety factor against overall sliding = Fr/Fh = 1.59

SOIL PRESSURES UNDER BASE

Maximum pressure :465.9 kPa
Minimum pressure : 0 kPa at 1.6 m from right hand side of base.
Maximum pressure occurs at left hand side of base

***** MAXIMUM PRESSURE EXCEEDS SPECIFIED ALLOWABLE PRESSURE BY 65.9 kPa *****

WALL MOMENTS (ULS) AND REINFORCEMENT TO BS8110 - 1997

Position from base top (m)	Moment (kNm)	Front Reinforcing (mm^2/m)	Back Reinforcing (mm^2/m)	Nominal (0.13%) (mm^2/m)
0.00	814.51	0.00	4839.68	655.20
0.14	777.45	0.00	4661.50	650.00
0.28	738.33	0.00	4467.55	644.80
0.41	700.34	0.00	4276.87	639.60
0.55	663.51	0.00	4089.84	634.40
0.69	627.84	0.00	3906.51	629.20
0.83	593.32	0.00	3726.88	624.00
0.97	559.94	0.00	3551.00	618.80
1.10	527.67	0.00	3378.88	613.60
1.24	496.52	0.00	3210.55	608.40
1.38	466.46	0.00	3046.03	603.20
1.52	437.48	0.00	2885.36	598.00
1.66	409.57	0.00	2728.56	592.80
1.79	382.71	0.00	2575.66	587.60
1.93	356.90	0.00	2426.70	582.40
2.07	332.11	0.00	2281.70	577.20
2.21	308.35	0.00	2140.71	572.00
2.35	285.58	0.00	2003.75	566.80
2.48	263.80	0.00	1870.86	561.60
2.62	243.00	0.00	1742.09	556.40
2.76	223.17	0.00	1617.46	551.20
2.90	204.28	0.00	1497.03	546.00
3.04	186.33	0.00	1380.84	540.80
3.17	169.31	0.00	1268.94	535.60
3.31	153.19	0.00	1161.36	530.40
3.45	137.98	0.00	1058.17	525.20
3.59	123.65	0.00	959.41	520.00
3.73	110.19	0.00	865.15	514.80
3.86	97.58	0.00	775.43	509.60
4.00	85.83	0.00	690.32	504.40
4.14	74.90	0.00	609.88	499.20
4.28	64.79	0.00	534.18	494.00
4.42	55.49	0.00	463.30	488.80
4.55	46.99	0.00	397.29	483.60
4.69	39.26	0.00	336.25	478.40
4.83	32.29	0.00	280.25	473.20
4.97	26.08	0.00	229.38	468.00
5.11	20.61	0.00	183.73	462.80

5.38	11.84	0.00	108.48	452.40
5.52	8.51	0.00	79.09	447.20
5.66	5.87	0.00	55.34	442.00
5.80	3.91	0.00	37.34	436.80
5.93	2.60	0.00	25.22	431.60
6.07	1.87	0.00	18.40	426.40
6.21	1.30	0.00	13.01	421.20
6.35	0.84	0.00	8.50	416.00
6.49	0.48	0.00	4.90	410.80
6.62	0.21	0.00	2.25	405.60
6.76	0.06	0.00	0.61	400.40
6.90	0.00	0.00	0.02	395.20

BASE MOMENTS (ULS) AND REINFORCEMENT TO BS8110 - 1997

Position from left (m)	Moment (kNm)	Top Reinforcing (mm²/m)	Bot Reinforcing (mm²/m)	Nominal (0.13%) (mm²/m)
0.08	-0.00	0.00	0.00	845.00
0.16	-2.48	0.00	11.08	845.00
0.24	-9.91	0.00	44.32	845.00
0.32	-22.30	0.00	99.71	845.00
0.40	-39.64	0.00	177.26	845.00
0.48	-61.94	0.00	276.98	845.00
0.56	-89.20	0.00	398.84	845.00
0.64	-121.41	0.00	542.87	845.00
0.72	-158.57	0.00	709.06	845.00
0.80	-200.69	0.00	897.40	845.00
0.88	-217.77	973.74	0.00	845.00
0.96	799.10	3573.15	0.00	845.00
1.04	774.70	3464.05	0.00	845.00
1.12	736.45	3293.02	0.00	845.00
1.20	699.17	3126.31	0.00	845.00
1.28	662.85	2963.93	0.00	845.00
1.36	627.51	2805.88	0.00	845.00
1.44	593.13	2652.17	0.00	845.00
1.52	559.72	2502.78	0.00	845.00
1.60	527.28	2357.72	0.00	845.00
1.68	495.81	2216.99	0.00	845.00
1.76	465.30	2080.60	0.00	845.00
1.84	435.77	1948.53	0.00	845.00
1.92	407.20	1820.79	0.00	845.00
2.00	379.60	1697.39	0.00	845.00
2.08	352.97	1578.31	0.00	845.00
2.16	327.31	1463.56	0.00	845.00
2.24	302.62	1353.15	0.00	845.00
2.32	278.89	1247.06	0.00	845.00
2.40	256.13	1145.30	0.00	845.00
2.48	234.35	1047.88	0.00	845.00
2.56	213.53	954.78	0.00	845.00
2.64	193.67	866.01	0.00	845.00
2.72	174.79	781.58	0.00	845.00
2.80	156.88	701.47	0.00	845.00
2.88	139.93	625.69	0.00	845.00
2.96	123.95	554.25	0.00	845.00
3.04	108.94	487.13	0.00	845.00
3.12	94.90	424.35	0.00	845.00
3.20	81.83	365.89	0.00	845.00
3.28	69.72	311.76	0.00	845.00
3.36	58.59	261.97	0.00	845.00
3.44	48.42	216.50	0.00	845.00
3.52	39.22	175.37	0.00	845.00
3.60	30.99	138.56	0.00	845.00
3.68	23.73	106.09	0.00	845.00
3.76	17.43	77.94	0.00	845.00
3.84	12.10	54.13	0.00	845.00
3.92	7.75	34.64	0.00	845.00
4.00	4.36	19.49	0.00	845.00

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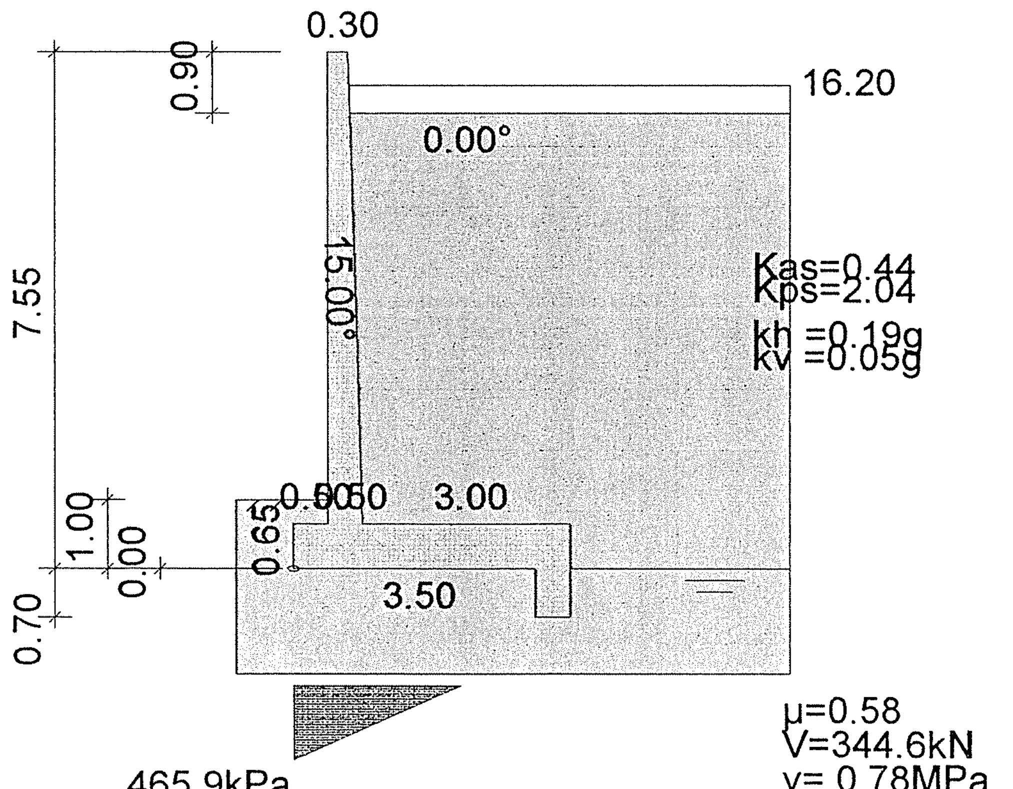
	Calcs by N.O	Checked by	Date FEB,2000

SHEAR CHECK AT WALL-BASE JUNCTION TO BS8110 - 1997

```
 Shear force at bottom of wall V = 344.6 kN
Shear stress at bottom of wall v = 0.78 MPa INADEQUATE
       Allowable shear stress vc = 0.65 MPa (based on Wall tensile reinf.)
```

Sketch of Wall

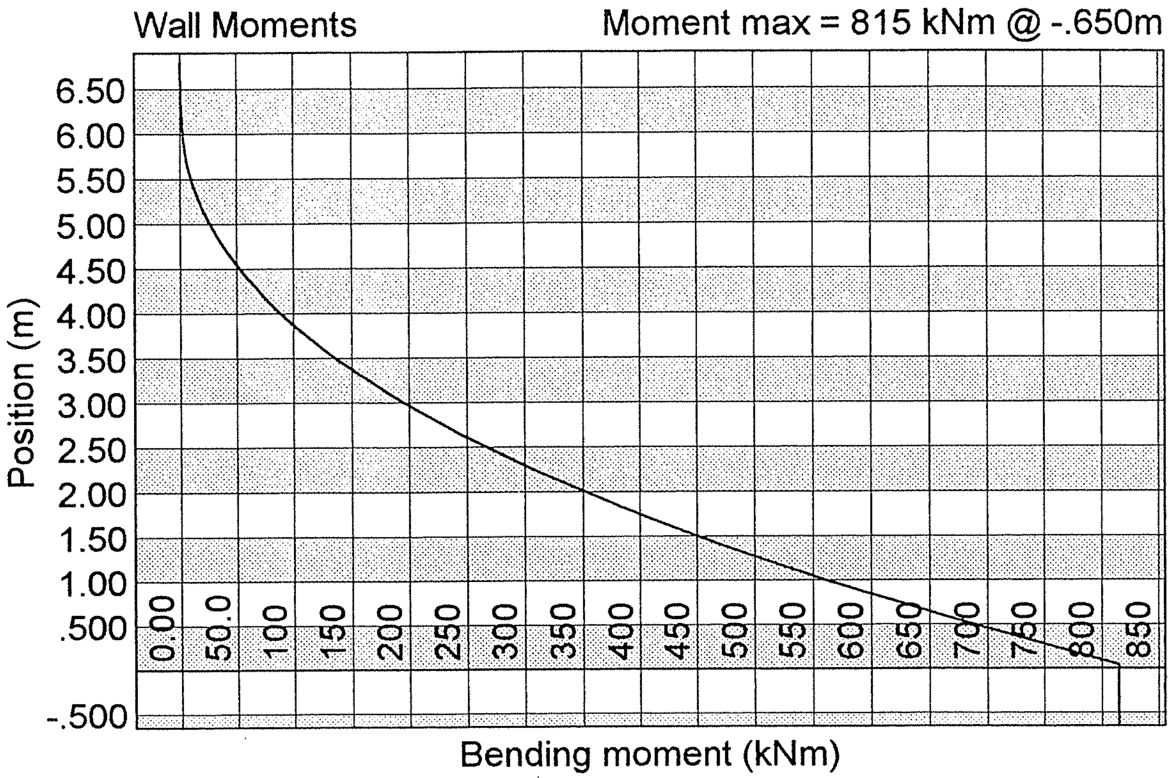
Design code: BS8110 - 1997



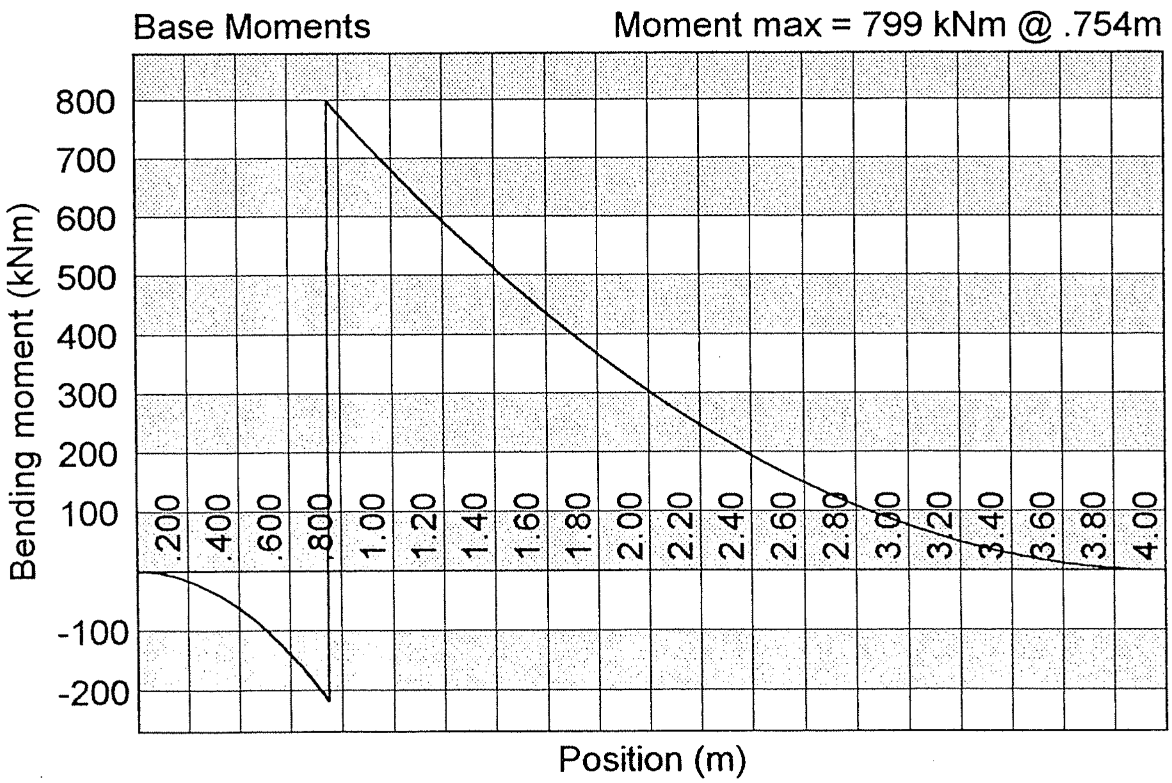
Wall type: Cantilever SFslip = 1.59
Theory: Coulomb SFovt = 1.72

PROKON	Job Number 12132		Sheet 17
Software Consultants (Pty) Ltd Internet: http://www.prokon.com E-Mail : mail@prokon.com	Job Title Dead Sea Parkway - JICA		
	Client Ministry of Tourism		
	Calcs by N.O	Checked by	Date FEB,2000

Wall Bending Moments



Base Bending Moments



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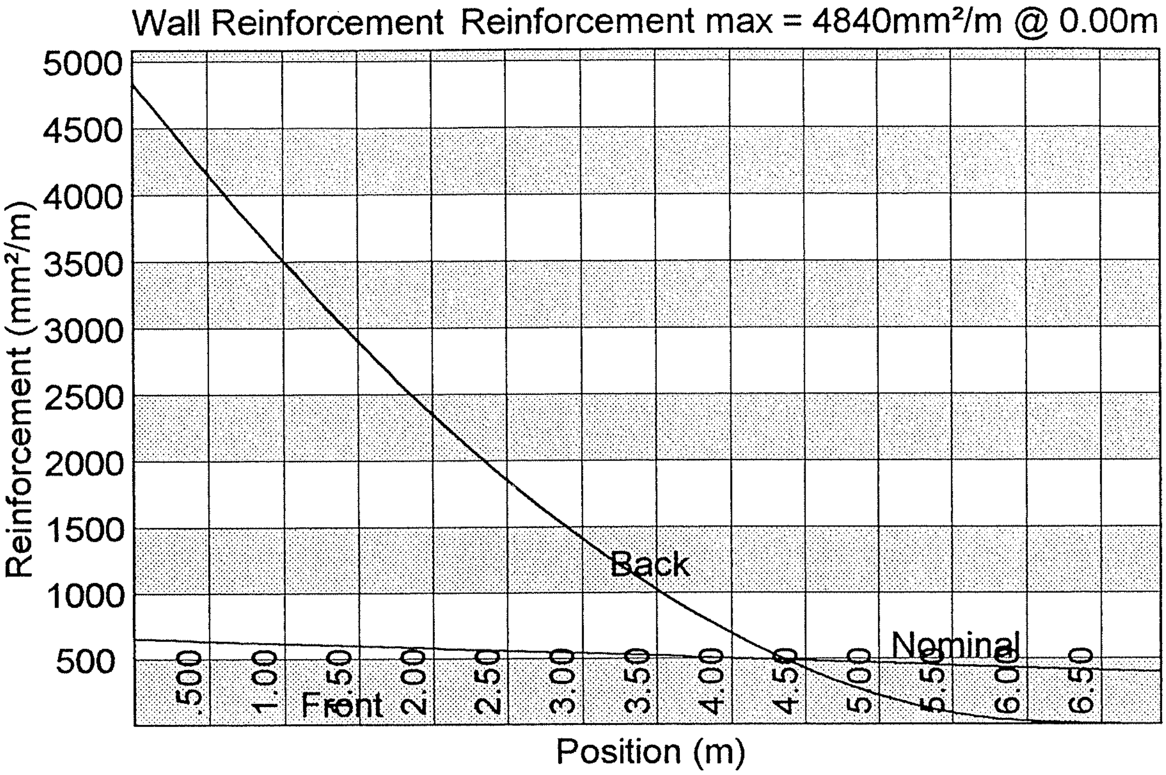
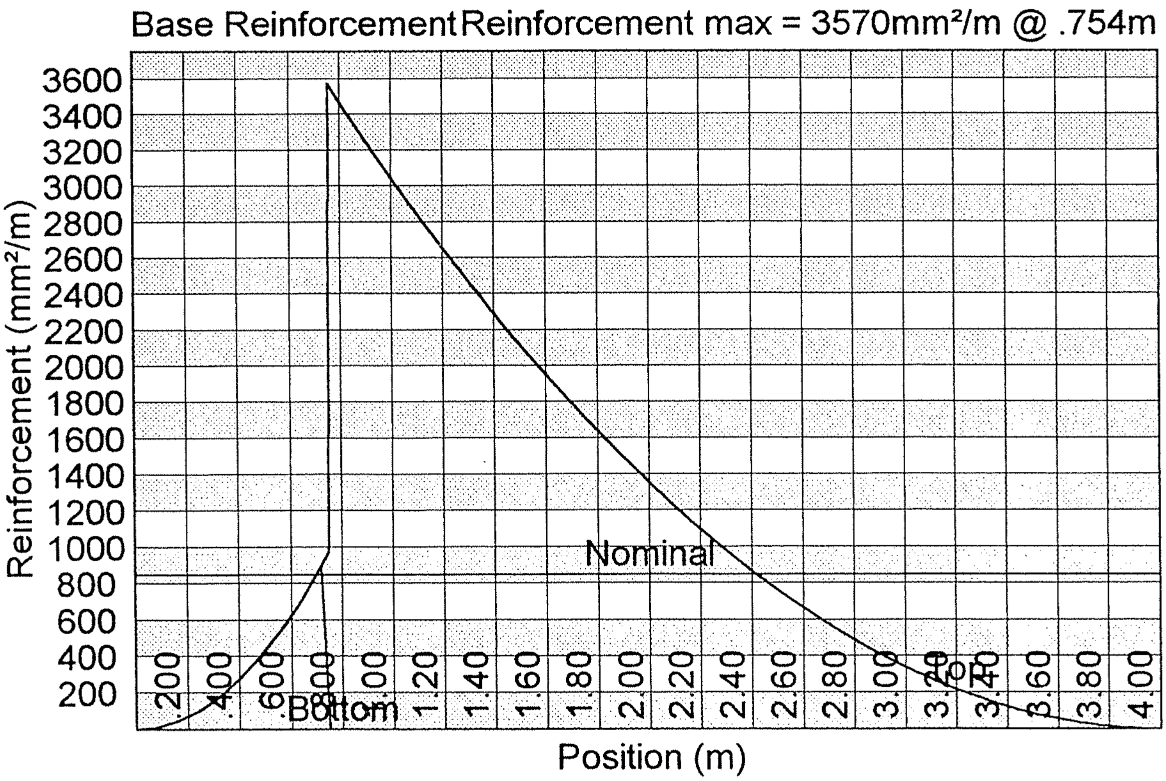
	Job Title Dead Sea Parkway - JICA			
	Client Ministry of Tourism			
	Calcs by N.O	Checked by	Date FEB,2000	

Wall Reinforcement



Base Reinforcement



C14

Retaining Wall Design : Height = 6.0 m

Input Data

Seepage allowed

Theory : Coulomb
Wall type : Cantilever

SEISMIC ANALYSIS SETTINGS:

Seismic Analysis ON/OFF:ON

VALUES OF PRESSURE COEFFICIENTS:

Active Pressure coefficient Ka :0.32
Passive Pressure coefficient Kp :4.98
Seismic Active Pressure coefficient Kas :0.44
Seismic Passive Pressure coefficient Kps :2.04
Base frictional constant μ :0.58

FORCES ACTING ON THE WALL:

FORCES (kN) and their LEVER ARMS (m)

Description	F Horizontal left (+)	Lever arm	F Vertical down (+)	Lever arm
Destabilizing forces:				
Total Active pressure Pa	201.798	2.929	60.327	1.247
Siesmic component of Pa	57.674	4.320	17.547	1.200
As a result of surcharge w	47.093	3.600	14.328	1.251
Siesmic wall inertia	27.624	4.050		
Stabilizing forces:				
Passive pressure on base Pp	-14.891	0.300		
Siesmic component of Pp	21.388	0.540		
Weight of the wall + base			153.039	1.749
Weight of soil on the base			362.945	2.821
UDL of 16.2 kN/m2			51.556	2.775

EQUILIBRIUM CALCULATIONS AT SLS

1.Moment Equilibrium

Point of rotation: bottom front corner of base.

For Overturning moment Mo calculate as follows:
Mo = Sum(hor. forces x l.a.) - Sum(vert. forces x l.a.)
For Stabilizing moment Mr calculate as follows:
Mr = -Sum(hor. forces x l.a.) + Sum(vert. forces x l.a.)
where l.a. = lever arm of each force

Stabilizing moment Mr : 1439.2 kNm
Destabilizing moment Mo : 779.2 kNm

Safety factor against overturning = Mr/Mo = 1.85

2.Force Equilibrium

Sum of Vertical forces Pv : 642.2 kN
Frictional resistance Pfric : 370.8 kN
Passive Pressure on shear key : 32.2 kN
Passive pressure on base : 14.9 kN
=> Horizontal resistance Fr : 417.8 kN
Horizontal sliding force Fh : 276.5 kN

Safety factor against overall sliding = Fr/Fh = 1.51

SOIL PRESSURES UNDER BASE

Maximum pressure :416.6 kPa
Minimum pressure : 0 kPa at 1.4 m from right hand side of base.
Maximum pressure occurs at left hand side of base

***** MAXIMUM PRESSURE EXCEEDS SPECIFIED ALLOWABLE PRESSURE BY 16.6 kPa *****

WALL MOMENTS (ULS) AND REINFORCEMENT TO BS8110 - 1997

Position from base top (m)	Moment (kNm)	Front Reinforcing (mm^2/m)	Back Reinforcing (mm^2/m)	Nominal (0.13%) (mm^2/m)
0.00	1031.66	0.00	5498.37	721.50
0.15	985.30	0.00	5304.87	715.00
0.30	935.98	0.00	5091.28	708.50
0.45	888.14	0.00	4881.38	702.00
0.60	841.77	0.00	4675.21	695.50
0.74	796.85	0.00	4472.82	689.00
0.89	753.37	0.00	4274.22	682.50
1.04	711.31	0.00	4079.45	676.00
1.19	670.65	0.00	3888.53	669.50
1.34	631.38	0.00	3701.51	663.00
1.49	593.47	0.00	3518.40	656.50
1.64	556.93	0.00	3339.25	650.00
1.79	521.72	0.00	3164.09	643.50
1.94	487.83	0.00	2992.96	637.00
2.09	455.24	0.00	2825.90	630.50
2.23	423.95	0.00	2662.96	624.00
2.38	393.92	0.00	2504.17	617.50
2.53	365.15	0.00	2349.58	611.00
2.68	337.62	0.00	2199.23	604.50
2.83	311.30	0.00	2053.19	598.00
2.98	286.20	0.00	1911.50	591.50
3.13	262.28	0.00	1774.22	585.00
3.28	239.54	0.00	1641.41	578.50
3.43	217.95	0.00	1513.12	572.00
3.58	197.50	0.00	1389.43	565.50
3.72	178.17	0.00	1270.40	559.00
3.87	159.95	0.00	1156.10	552.50
4.02	142.82	0.00	1046.61	546.00
4.17	126.76	0.00	942.02	539.50
4.32	111.76	0.00	842.40	533.00
4.47	97.80	0.00	747.85	526.50
4.62	84.86	0.00	658.45	520.00
4.77	72.93	0.00	574.33	513.50
4.92	61.99	0.00	495.57	507.00
5.07	52.02	0.00	422.29	500.50
5.21	43.01	0.00	354.62	494.00
5.36	34.95	0.00	292.69	487.50
5.51	27.80	0.00	236.62	481.00

5.81	16.23	0.00	142.69	468.00
5.96	11.76	0.00	105.15	461.50
6.11	8.15	0.00	74.12	455.00
6.26	5.38	0.00	49.80	448.50
6.41	3.44	0.00	32.39	442.00
6.56	2.30	0.00	22.10	435.50
6.70	1.61	0.00	15.69	429.00
6.85	1.03	0.00	10.28	422.50
7.00	0.59	0.00	5.95	416.00
7.15	0.27	0.00	2.75	409.50
7.30	0.07	0.00	0.75	403.00
7.45	0.00	0.00	0.03	396.50

BASE MOMENTS (ULS) AND REINFORCEMENT TO BS8110 - 1997

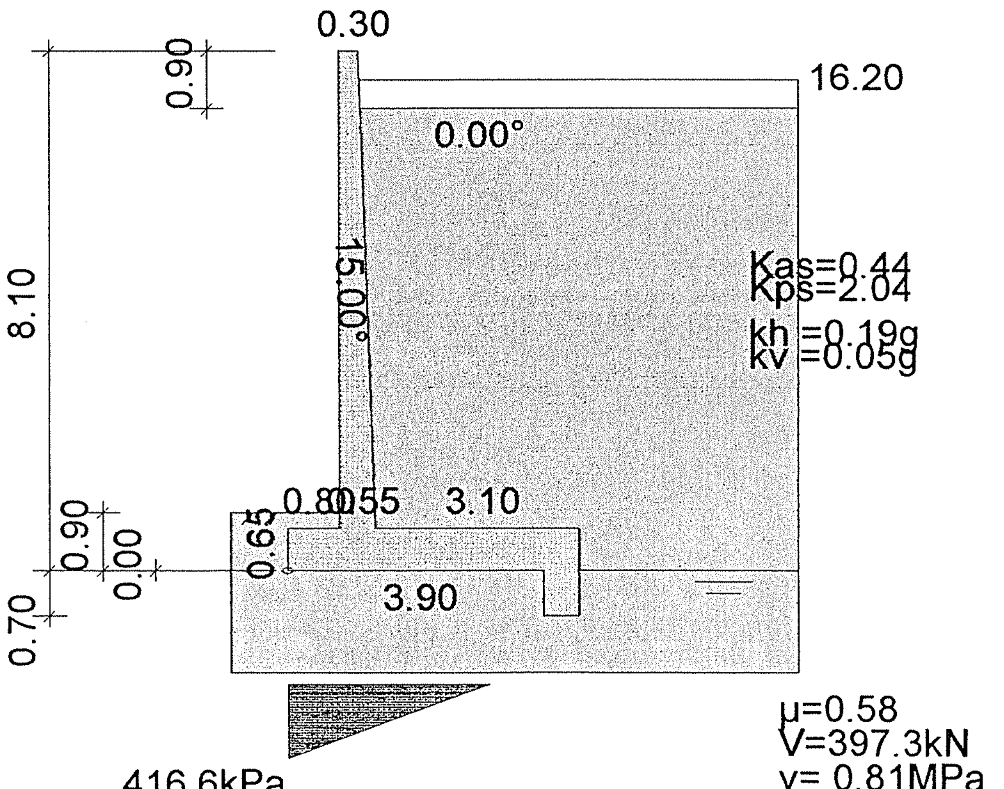
Position from left (m)	Moment (kNm)	Top Reinforcing (mm^2/m)	Bot Reinforcing (mm^2/m)	Nominal (0.13%) (mm^2/m)
0.09	-0.00	0.00	0.00	845.00
0.18	-2.44	0.00	10.93	845.00
0.27	-9.77	0.00	43.70	845.00
0.36	-21.99	0.00	98.33	845.00
0.45	-39.10	0.00	174.81	845.00
0.53	-61.09	0.00	273.15	845.00
0.62	-87.96	0.00	393.33	845.00
0.71	-119.73	0.00	535.37	845.00
0.80	-156.38	0.00	699.26	845.00
0.89	-197.92	0.00	885.00	845.00
0.98	-244.35	0.00	1092.59	845.00
1.07	-295.66	0.00	1322.03	845.00
1.16	-351.86	0.00	1573.33	845.00
1.25	-356.49	1594.02	0.00	845.00
1.33	913.36	4084.08	0.00	845.00
1.42	869.52	3888.04	0.00	845.00
1.51	823.15	3680.71	0.00	845.00
1.60	778.06	3479.07	0.00	845.00
1.69	734.23	3283.11	0.00	845.00
1.78	691.68	3092.82	0.00	845.00
1.87	650.39	2908.22	0.00	845.00
1.96	610.38	2729.29	0.00	845.00
2.05	571.63	2556.05	0.00	845.00
2.14	534.16	2388.49	0.00	845.00
2.23	497.96	2226.60	0.00	845.00
2.31	463.02	2070.40	0.00	845.00
2.40	429.36	1919.88	0.00	845.00
2.49	396.97	1775.04	0.00	845.00
2.58	365.85	1635.87	0.00	845.00
2.67	335.99	1502.39	0.00	845.00
2.76	307.41	1374.59	0.00	845.00
2.85	280.10	1252.46	0.00	845.00
2.94	254.06	1136.02	0.00	845.00
3.03	229.29	1025.26	0.00	845.00
3.12	205.79	920.18	0.00	845.00
3.20	183.56	820.78	0.00	845.00
3.29	162.60	727.05	0.00	845.00
3.38	142.91	639.01	0.00	845.00
3.47	124.49	556.65	0.00	845.00
3.56	107.34	479.97	0.00	845.00
3.65	91.46	408.97	0.00	845.00
3.74	76.85	343.65	0.00	845.00
3.83	63.51	284.01	0.00	845.00
3.92	51.45	230.04	0.00	845.00
4.00	40.65	181.76	0.00	845.00
4.09	31.12	139.16	0.00	845.00
4.18	22.87	102.24	0.00	845.00
4.27	15.88	71.00	0.00	845.00
4.36	10.16	45.44	0.00	845.00
4.45	5.72	25.56	0.00	845.00

SHEAR CHECK AT WALL-BASE JUNCTION TO BS8110 - 1997

```
 Shear force at bottom of wall V = 397.3 kN
Shear stress at bottom of wall v = 0.81 MPa INADEQUATE
       Allowable shear stress vc = 0.65 MPa (based on Wall tensile reinf.)
```

Sketch of Wall

Design code: BS8110 - 1997



Wall type: Cantilever
Theory: Coulomb
SFslip = 1.51
SFovt = 1.85

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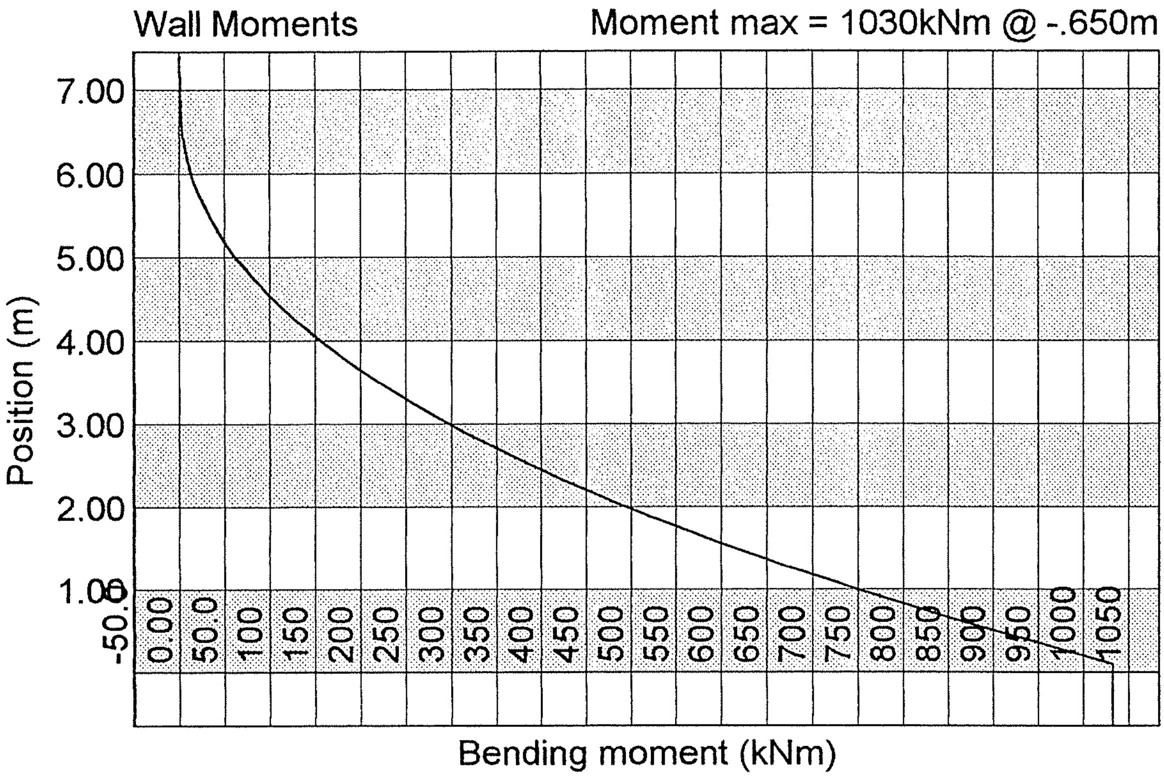
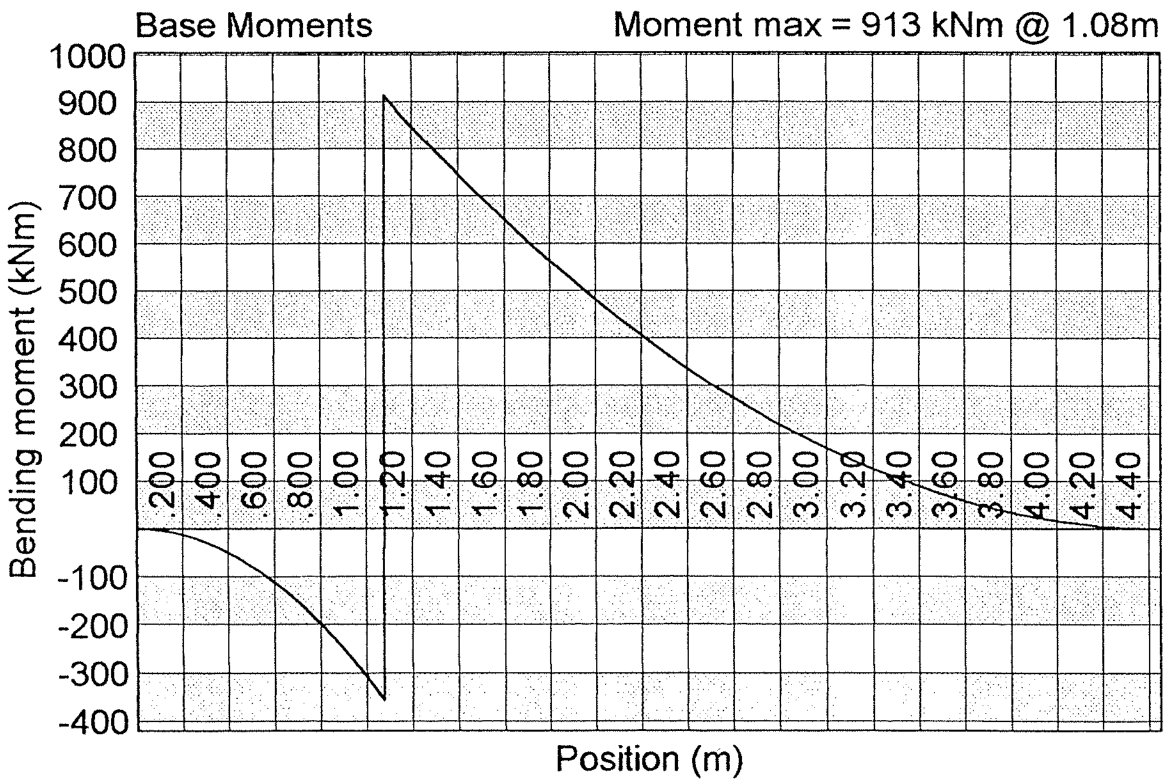
	Job Title Dead Sea Parkway - JICA		
	Client Ministry of Tourism		
	Calcs by N.O	Checked by	Date FEB,2000

Wall Bending Moments



Base Bending Moments



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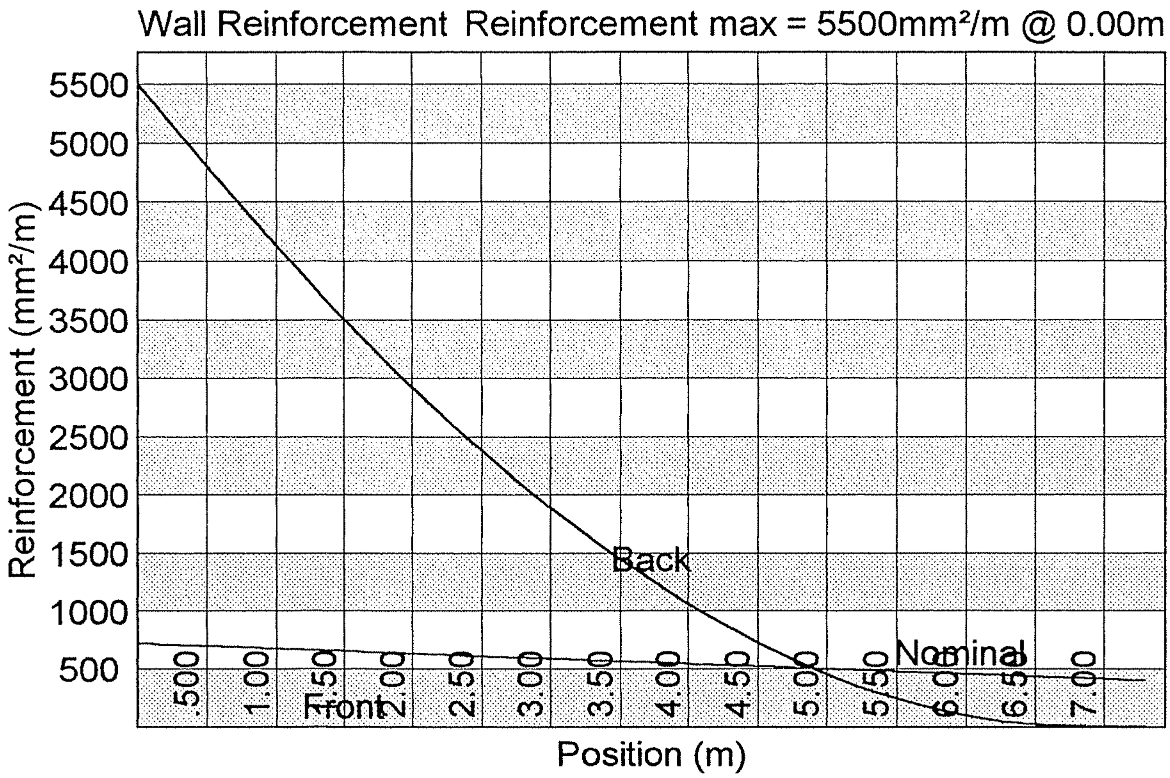
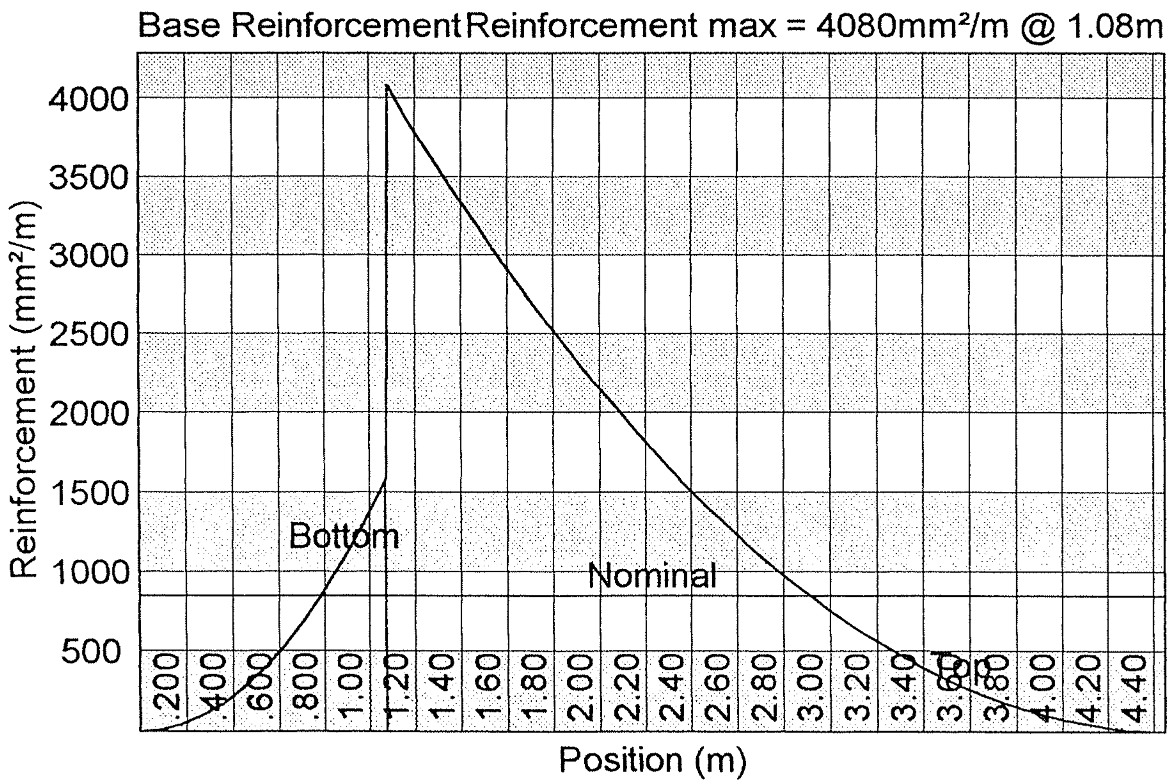
	Job Title Dead Sea Parkway - JICA		
	Client Ministry of Tourism		
	Calcs by N.O	Checked by	Date FEB,2000

Wall Reinforcement



Base Reinforcement



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Job Number	12132		Sheet	25
Job Title	Dead Sea Parkway - JICA			
Client	Ministry of Tourism			
Calcs by	N.O	Checked by	Date	FEB,2000

C14

Retaining Wall Design : Height = 6.5 m

Input Data

Seepage allowed

Theory : Coulomb
Wall type : Cantilever

SEISMIC ANALYSIS SETTINGS:

Seismic Analysis ON/OFF:ON

VALUES OF PRESSURE COEFFICIENTS:

Active Pressure coefficient Ka :0.32
Passive Pressure coefficient Kp :4.98
Seismic Active Pressure coefficient Kas :0.45
Seismic Passive Pressure coefficient Kps :2.04
Base frictional constant μ :0.58

FORCES ACTING ON THE WALL:

Description	FORCES (kN) and their LEVER ARMS (m) F Horizontal left (+)	Lever arm	F Vertical down (+)	Lever arm
Destabilizing forces:				
Total Active pressure Pa	234.412	3.150	71.227	1.277
Siesmic component of Pa	66.818	4.650	20.651	1.220
As a result of surcharge w	50.849	3.875	15.716	1.281
Siesmic wall inertia	32.473	4.325		
Stabilizing forces:				
Passive pressure on base Pp	-14.891	0.300		
Siesmic component of Pp	21.388	0.540		
Weight of the wall + base			179.906	1.875
Weight of soil on the base			424.941	3.020
UDL of 16.2 kN/m2			56.943	2.950

EQUILIBRIUM CALCULATIONS AT SLS

1.Moment Equilibrium

Point of rotation: bottom front corner of base.

For Overturning moment Mo calculate as follows:
Mo = Sum(hor. forces x l.a.) - Sum(vert. forces x l.a.)
For Stabilizing moment Mr calculate as follows:
Mr = -Sum(hor. forces x l.a.) + Sum(vert. forces x l.a.)
where l.a. = lever arm of each force

Stabilizing moment Mr : 1793.1 kNm
Destabilizing moment Mo : 964.9 kNm

Safety factor against overturning = Mr/Mo = 1.86

2.Force Equilibrium

Sum of Vertical forces Pv : 748.7 kN
Frictional resistance Pfric : 432.3 kN
Passive Pressure on shear key : 32.2 kN
Passive pressure on base : 14.9 kN
=> Horizontal resistance Fr : 479.3 kN
Horizontal sliding force Fh : 317.7 kN

Safety factor against overall sliding = Fr/Fh = 1.51

SOIL PRESSURES UNDER BASE

Maximum pressure :451.3 kPa
Minimum pressure : 0 kPa at 1.5 m from right hand side of base.
Maximum pressure occurs at left hand side of base

***** MAXIMUM PRESSURE EXCEEDS SPECIFIED ALLOWABLE PRESSURE BY 51.3 kPa *****

WALL MOMENTS (ULS) AND REINFORCEMENT TO BS8110 - 1997

Position from base top (m)	Moment (kNm)	Front Reinforcing (mm^2/m)	Back Reinforcing (mm^2/m)	Nominal (0.13%) (mm^2/m)
0.00	1250.66	0.00	6042.96	787.80
0.16	1195.20	0.00	5839.14	780.00
0.32	1135.71	0.00	5610.88	772.20
0.47	1078.01	0.00	5386.34	764.40
0.63	1022.08	0.00	5165.55	756.60
0.79	967.88	0.00	4948.54	748.80
0.95	915.42	0.00	4735.34	741.00
1.11	864.65	0.00	4525.99	733.20
1.26	815.57	0.00	4320.51	725.40
1.42	768.15	0.00	4118.94	717.60
1.58	722.38	0.00	3921.31	709.80
1.74	678.23	0.00	3727.68	702.00
1.90	635.68	0.00	3538.08	694.20
2.05	594.73	0.00	3352.54	686.40
2.21	555.33	0.00	3171.13	678.60
2.37	517.48	0.00	2993.88	670.80
2.53	481.16	0.00	2820.84	663.00
2.69	446.34	0.00	2652.08	655.20
2.84	413.01	0.00	2487.64	647.40
3.00	381.14	0.00	2327.58	639.60
3.16	350.72	0.00	2171.97	631.80
3.32	321.73	0.00	2020.87	624.00
3.48	294.14	0.00	1874.36	616.20
3.63	267.94	0.00	1732.50	608.40
3.79	243.10	0.00	1595.38	600.60
3.95	219.61	0.00	1463.07	592.80
4.11	197.45	0.00	1335.67	585.00
4.27	176.60	0.00	1213.27	577.20
4.42	157.03	0.00	1095.97	569.40
4.58	138.73	0.00	983.88	561.60
4.74	121.68	0.00	877.11	553.80
4.90	105.86	0.00	775.77	546.00
5.06	91.25	0.00	680.01	538.20
5.21	77.82	0.00	589.95	530.40
5.37	65.56	0.00	505.74	522.60
5.53	54.45	0.00	427.55	514.80
5.69	44.47	0.00	355.54	507.00
5.85	35.60	0.00	289.89	499.20

6.16	21.11	0.00	178.48	483.60
6.32	15.45	0.00	133.16	475.80
6.48	10.81	0.00	95.08	468.00
6.64	7.19	0.00	64.51	460.20
6.79	4.56	0.00	41.73	452.40
6.95	2.89	0.00	27.07	444.60
7.11	1.99	0.00	19.03	436.80
7.27	1.28	0.00	12.52	429.00
7.43	0.73	0.00	7.27	421.20
7.58	0.33	0.00	3.37	413.40
7.74	0.09	0.00	0.92	405.60
7.90	0.00	0.00	0.04	397.80

BASE MOMENTS (ULS) AND REINFORCEMENT TO BS8110 - 1997

Position from left (m)	Moment (kNm)	Top Reinforcing (mm^2/m)	Bot Reinforcing (mm^2/m)	Nominal (0.13%) (mm^2/m)
0.10	-0.00	0.00	0.00	975.00
0.19	-3.11	0.00	11.89	975.00
0.29	-12.44	0.00	47.58	975.00
0.38	-28.00	0.00	107.05	975.00
0.48	-49.78	0.00	190.32	975.00
0.58	-77.78	0.00	297.37	975.00
0.67	-112.00	0.00	428.21	975.00
0.77	-152.44	0.00	582.84	975.00
0.86	-199.10	0.00	761.26	975.00
0.96	-251.99	0.00	963.47	975.00
1.06	-311.10	0.00	1189.47	975.00
1.15	-376.43	0.00	1439.26	975.00
1.25	-408.45	1561.70	0.00	975.00
1.34	1157.81	4426.81	0.00	975.00
1.44	1125.49	4303.26	0.00	975.00
1.54	1067.04	4079.75	0.00	975.00
1.63	1010.14	3862.20	0.00	975.00
1.73	954.80	3650.62	0.00	975.00
1.82	901.02	3444.99	0.00	975.00
1.92	848.80	3245.32	0.00	975.00
2.02	798.13	3051.62	0.00	975.00
2.11	749.03	2863.87	0.00	975.00
2.21	701.48	2682.09	0.00	975.00
2.30	655.50	2506.26	0.00	975.00
2.40	611.07	2336.39	0.00	975.00
2.50	568.20	2172.49	0.00	975.00
2.59	526.89	2014.54	0.00	975.00
2.69	487.14	1862.56	0.00	975.00
2.78	448.95	1716.54	0.00	975.00
2.88	412.32	1576.47	0.00	975.00
2.98	377.24	1442.37	0.00	975.00
3.07	343.73	1314.22	0.00	975.00
3.17	311.77	1192.04	0.00	975.00
3.26	281.37	1075.81	0.00	975.00
3.36	252.53	965.55	0.00	975.00
3.46	225.25	861.25	0.00	975.00
3.55	199.53	762.90	0.00	975.00
3.65	175.37	670.52	0.00	975.00
3.74	152.77	584.10	0.00	975.00
3.84	131.72	503.64	0.00	975.00
3.94	112.24	429.13	0.00	975.00
4.03	94.31	360.59	0.00	975.00
4.13	77.94	298.01	0.00	975.00
4.22	63.13	241.39	0.00	975.00
4.32	49.88	190.73	0.00	975.00
4.42	38.19	146.02	0.00	975.00
4.51	28.06	107.28	0.00	975.00
4.61	19.49	74.50	0.00	975.00
4.70	12.47	47.68	0.00	975.00
4.80	7.01	26.82	0.00	975.00

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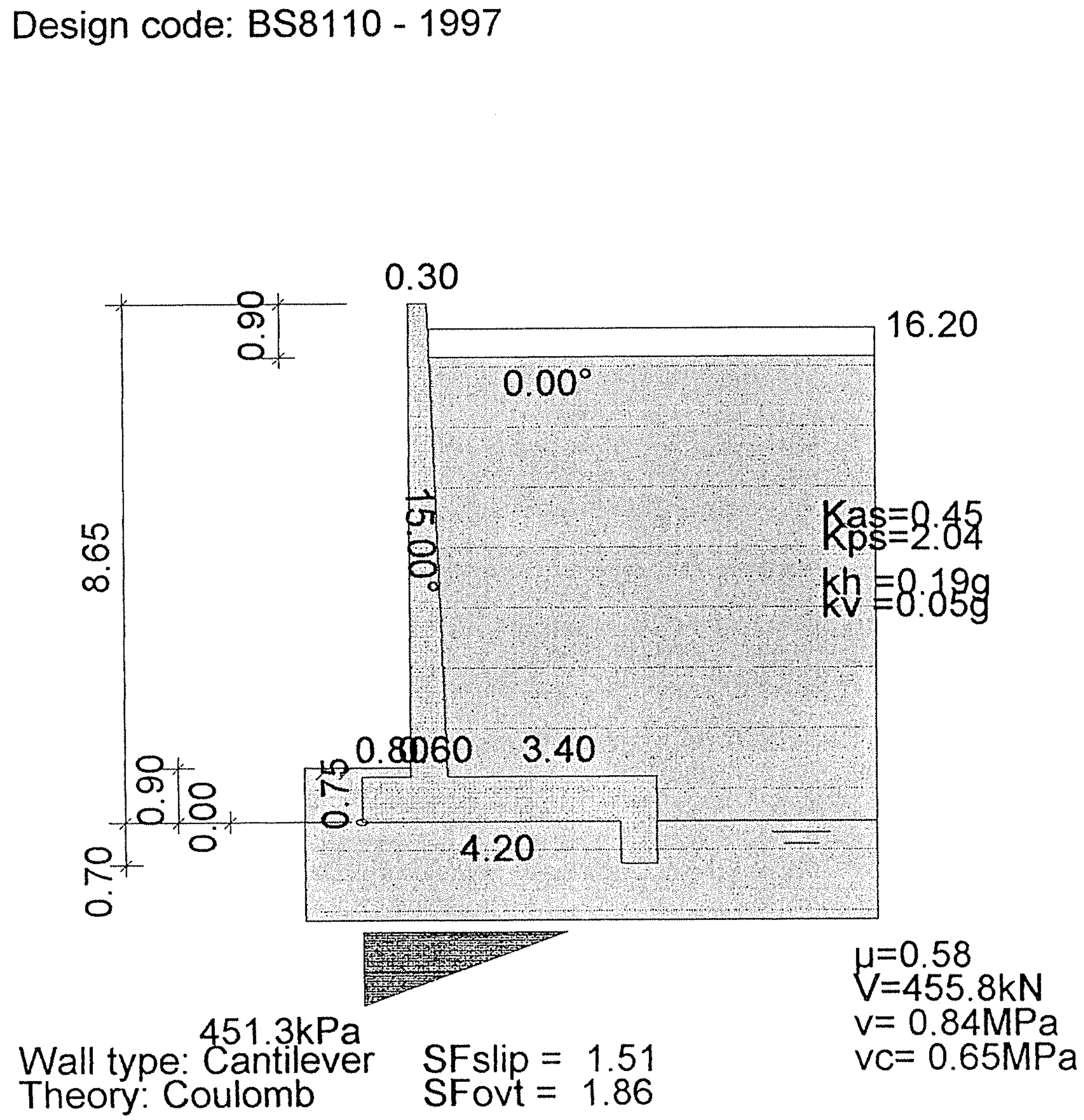
	Calcs by N.O	Checked by	Date FEB,2000

```
SHEAR CHECK AT WALL-BASE JUNCTION TO BS8110 - 1997

 Shear force at bottom of wall V = 455.8 kN
Shear stress at bottom of wall v = 0.84 MPa INADEQUATE
       Allowable shear stress vc = 0.65 MPa (based on Wall tensile reinf.)
```

Sketch of Wall



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	Client Ministry of Tourism		
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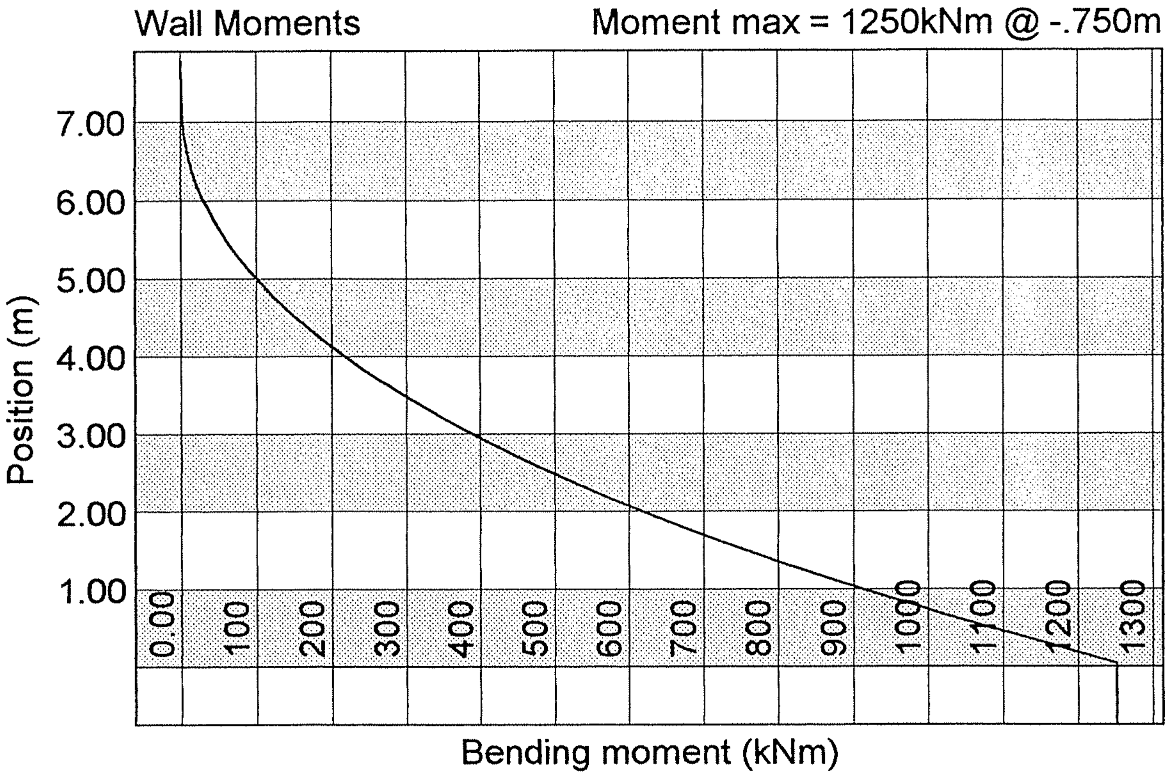
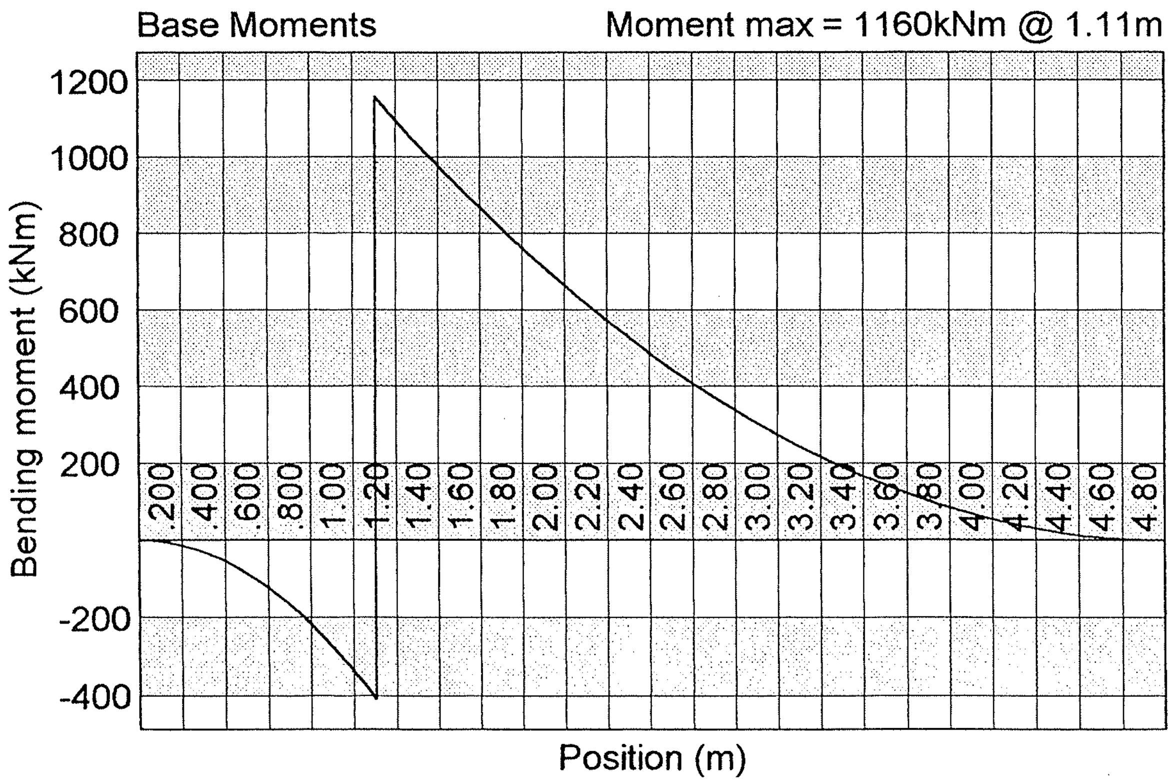
Wall Bending Moments



Base Bending Moments



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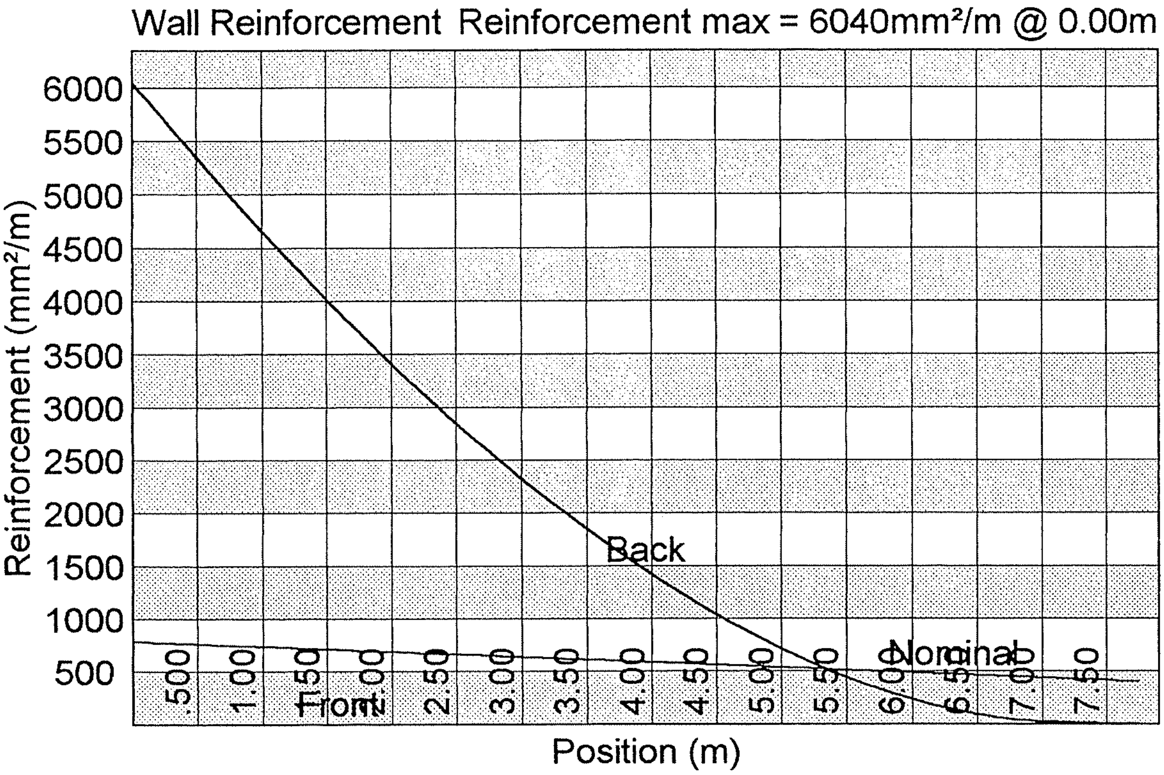
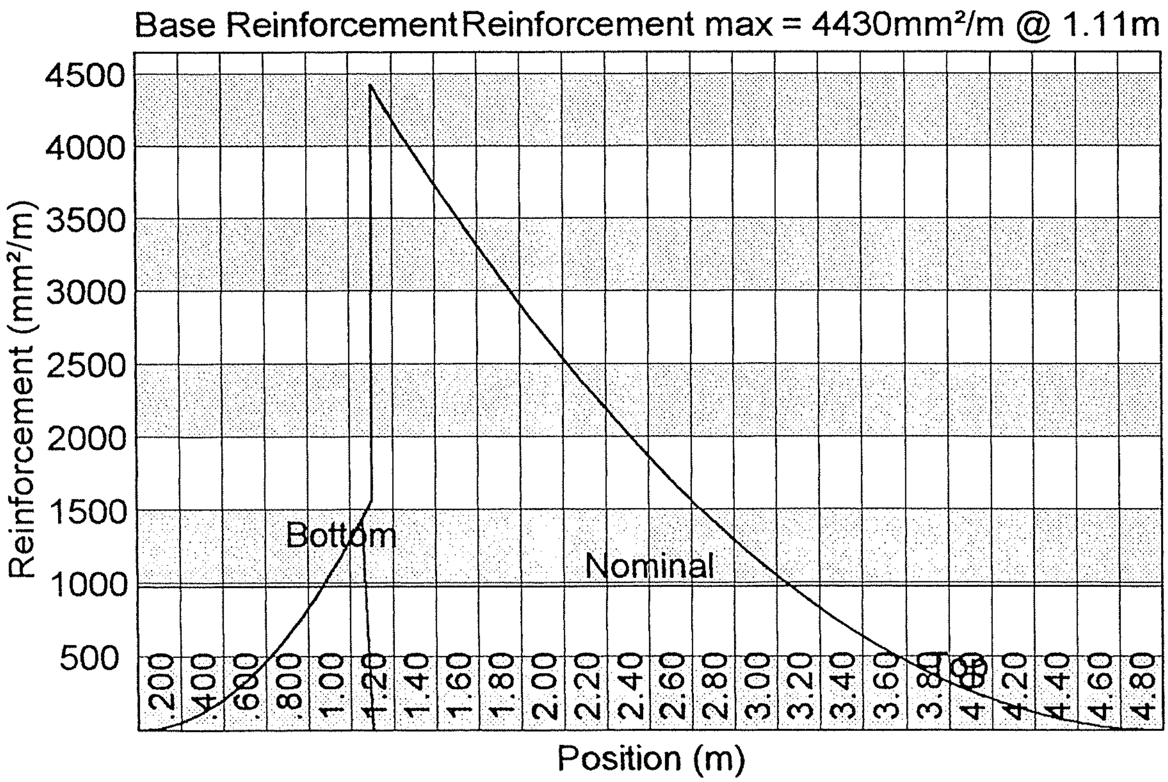
	Job Title Dead Sea Parkway - JICA		
	Client Ministry of Tourism		
	Calcs by N.O	Checked by	Date FEB,2000

Wall Reinforcement



Base Reinforcement



C14

Retaining Wall Design : Height = 7.0 m

Input Data

Seepage allowed

Theory : Coulomb
Wall type : Cantilever

SEISMIC ANALYSIS SETTINGS:

Seismic Analysis ON/OFF:ON

VALUES OF PRESSURE COEFFICIENTS:

Active Pressure coefficient Ka :0.32
Passive Pressure coefficient Kp :4.98
Seismic Active Pressure coefficient Kas :0.45
Seismic Passive Pressure coefficient Kps :2.04
Base frictional constant μ :0.58

FORCES ACTING ON THE WALL:

FORCES (kN) and their LEVER ARMS (m)

Description	F Horizontal left (+)	Lever arm	F Vertical down (+)	Lever arm
Destabilizing forces:				
Total Active pressure Pa	266.272	3.357	82.068	1.606
Siesmic component of Pa	75.715	4.950	23.728	1.540
As a result of surcharge w	54.280	4.125	17.011	1.611
Siesmic wall inertia	37.644	4.575		
Stabilizing forces:				
Passive pressure on base Pp	-14.891	0.300		
Siesmic component of Pp	21.388	0.540		
Weight of the wall + base			208.555	2.228
Weight of soil on the base			493.134	3.510
UDL of 16.2 kN/m2			62.329	3.425

EQUILIBRIUM CALCULATIONS AT SLS

1.Moment Equilibrium

Point of rotation: bottom front corner of base.

For Overturning moment Mo calculate as follows:
Mo = Sum(hor. forces x l.a.) - Sum(vert. forces x l.a.)
For Stabilizing moment Mr calculate as follows:
Mr = -Sum(hor. forces x l.a.) + Sum(vert. forces x l.a.)
where l.a. = lever arm of each force

```
        Stabilizing moment Mr : 2413.2 kNm
      Destabilizing moment Mo : 1130.7 kNm

Safety factor against overturning = Mr/Mo = 2.13

2.Force Equilibrium

    Sum of Vertical forces Pv : 863.1 kN
  Frictional resistance Pfric : 498.3 kN
Passive Pressure on shear key :  32.2 kN
     Passive pressure on base :  14.9 kN
  => Horizontal resistance Fr : 545.4 kN
  Horizontal sliding force Fh : 358.2 kN

Safety factor against overall sliding = Fr/Fh = 1.52
```

SOIL PRESSURES UNDER BASE

Maximum pressure :387.2 kPa
Minimum pressure : 0 kPa at 1.0 m from right hand side of base.
Maximum pressure occurs at left hand side of base

WALL MOMENTS (ULS) AND REINFORCEMENT TO BS8110 - 1997

Position from base top (m)	Moment (kNm)	Front Reinforcing (mm^2/m)	Back Reinforcing (mm^2/m)	Nominal (0.13%) (mm^2/m)
0.00	1498.22	0.00	6620.71	854.10
0.17	1432.66	0.00	6406.13	845.00
0.33	1361.78	0.00	6162.28	835.90
0.50	1293.01	0.00	5922.19	826.80
0.67	1226.33	0.00	5685.87	817.70
0.84	1161.71	0.00	5453.36	808.60
1.00	1099.13	0.00	5224.70	799.50
1.17	1038.58	0.00	4999.92	790.40
1.34	980.02	0.00	4779.05	781.30
1.50	923.43	0.00	4562.13	772.20
1.67	868.80	0.00	4349.21	763.10
1.84	816.08	0.00	4140.33	754.00
2.00	765.27	0.00	3935.52	744.90
2.17	716.34	0.00	3734.85	735.80
2.34	669.27	0.00	3538.35	726.70
2.51	624.02	0.00	3346.09	717.60
2.67	580.59	0.00	3158.11	708.50
2.84	538.93	0.00	2974.48	699.40
3.01	499.05	0.00	2795.26	690.30
3.17	460.89	0.00	2620.52	681.20
3.34	424.46	0.00	2450.32	672.10
3.51	389.72	0.00	2284.75	663.00
3.67	356.64	0.00	2123.88	653.90
3.84	325.21	0.00	1967.79	644.80
4.01	295.40	0.00	1816.58	635.70
4.18	267.19	0.00	1670.35	626.60
4.34	240.55	0.00	1529.19	617.50
4.51	215.46	0.00	1393.22	608.40
4.68	191.91	0.00	1262.55	599.30
4.84	169.85	0.00	1137.31	590.20
5.01	149.28	0.00	1017.64	581.10
5.18	130.17	0.00	903.68	572.00
5.34	112.48	0.00	795.59	562.90
5.51	96.21	0.00	693.53	553.80
5.68	81.33	0.00	597.69	544.70
5.85	67.81	0.00	508.26	535.60
6.01	55.64	0.00	425.45	526.50
6.18	44.78	0.00	349.49	517.40
6.35	35.21	0.00	280.63	508.30
6.51	26.91	0.00	219.15	499.20

6.85	14.04	0.00	119.49	481.00
7.01	9.42	0.00	81.98	471.90
7.18	5.97	0.00	53.19	462.80
7.35	3.67	0.00	33.53	453.70
7.52	2.44	0.00	22.82	444.60
7.68	1.57	0.00	15.06	435.50
7.85	0.89	0.00	8.77	426.40
8.02	0.40	0.00	4.08	417.30
8.18	0.11	0.00	1.12	408.20
8.35	0.00	0.00	0.04	399.10

BASE MOMENTS (ULS) AND REINFORCEMENT TO BS8110 - 1997

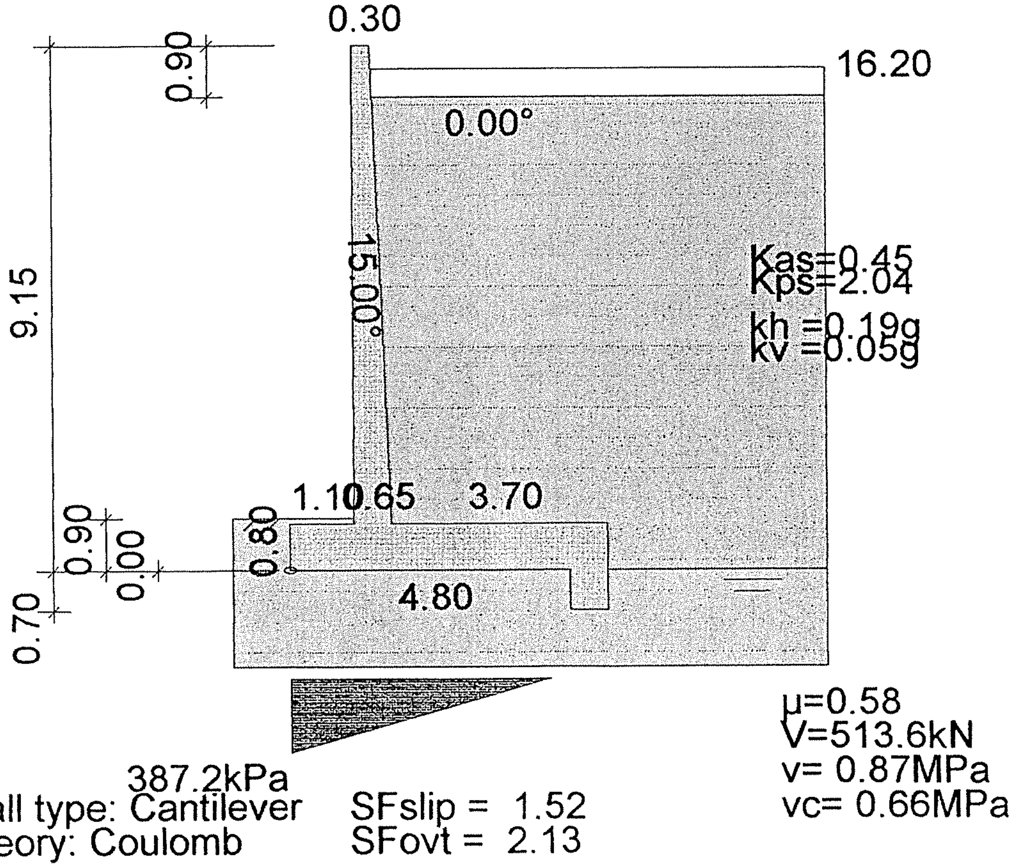
Position from left (m)	Moment (kNm)	Top Reinforcing (mm^2/m)	Bot Reinforcing (mm^2/m)	Nominal (0.13%) (mm^2/m)
0.11	-0.00	0.00	0.00	1040.00
0.22	-3.19	0.00	11.36	1040.00
0.33	-12.75	0.00	45.46	1040.00
0.44	-28.69	0.00	102.28	1040.00
0.55	-51.00	0.00	181.82	1040.00
0.65	-79.69	0.00	284.10	1040.00
0.76	-114.75	0.00	409.10	1040.00
0.87	-156.19	0.00	556.84	1040.00
0.98	-204.00	0.00	727.30	1040.00
1.09	-258.19	0.00	920.48	1040.00
1.20	-318.76	0.00	1136.40	1040.00
1.31	-385.70	0.00	1375.04	1040.00
1.42	-459.01	0.00	1636.42	1040.00
1.53	-538.70	0.00	1920.52	1040.00
1.64	-544.80	0.00	1942.26	1040.00
1.74	1334.37	4757.16	0.00	1040.00
1.85	1268.24	4521.41	0.00	1040.00
1.96	1198.76	4273.71	0.00	1040.00
2.07	1131.24	4032.98	0.00	1040.00
2.18	1065.68	3799.24	0.00	1040.00
2.29	1002.07	3572.47	0.00	1040.00
2.40	940.42	3352.68	0.00	1040.00
2.51	880.72	3139.87	0.00	1040.00
2.62	822.99	2934.03	0.00	1040.00
2.73	767.21	2735.17	0.00	1040.00
2.83	713.39	2543.29	0.00	1040.00
2.94	661.52	2358.39	0.00	1040.00
3.05	611.61	2180.46	0.00	1040.00
3.16	563.66	2009.51	0.00	1040.00
3.27	517.67	1845.54	0.00	1040.00
3.38	473.63	1688.55	0.00	1040.00
3.49	431.55	1538.53	0.00	1040.00
3.60	391.43	1395.50	0.00	1040.00
3.71	353.27	1259.44	0.00	1040.00
3.81	317.06	1130.35	0.00	1040.00
3.92	282.81	1008.25	0.00	1040.00
4.03	250.52	893.12	0.00	1040.00
4.14	220.18	784.97	0.00	1040.00
4.25	191.80	683.79	0.00	1040.00
4.36	165.38	589.60	0.00	1040.00
4.47	140.92	502.38	0.00	1040.00
4.58	118.41	422.14	0.00	1040.00
4.69	97.86	348.87	0.00	1040.00
4.80	79.27	282.59	0.00	1040.00
4.91	62.63	223.28	0.00	1040.00
5.01	47.95	170.95	0.00	1040.00
5.12	35.23	125.59	0.00	1040.00
5.23	24.46	87.22	0.00	1040.00
5.34	15.66	55.82	0.00	1040.00
5.45	8.81	31.40	0.00	1040.00

SHEAR CHECK AT WALL-BASE JUNCTION TO BS8110 - 1997

```
  Shear force at bottom of wall V = 513.6 kN
Shear stress at bottom of wall v = 0.87 MPa INADEQUATE
      Allowable shear stress vc = 0.66 MPa (based on Wall tensile reinf.)
```

Sketch of Wall

Design code: BS8110 - 1997



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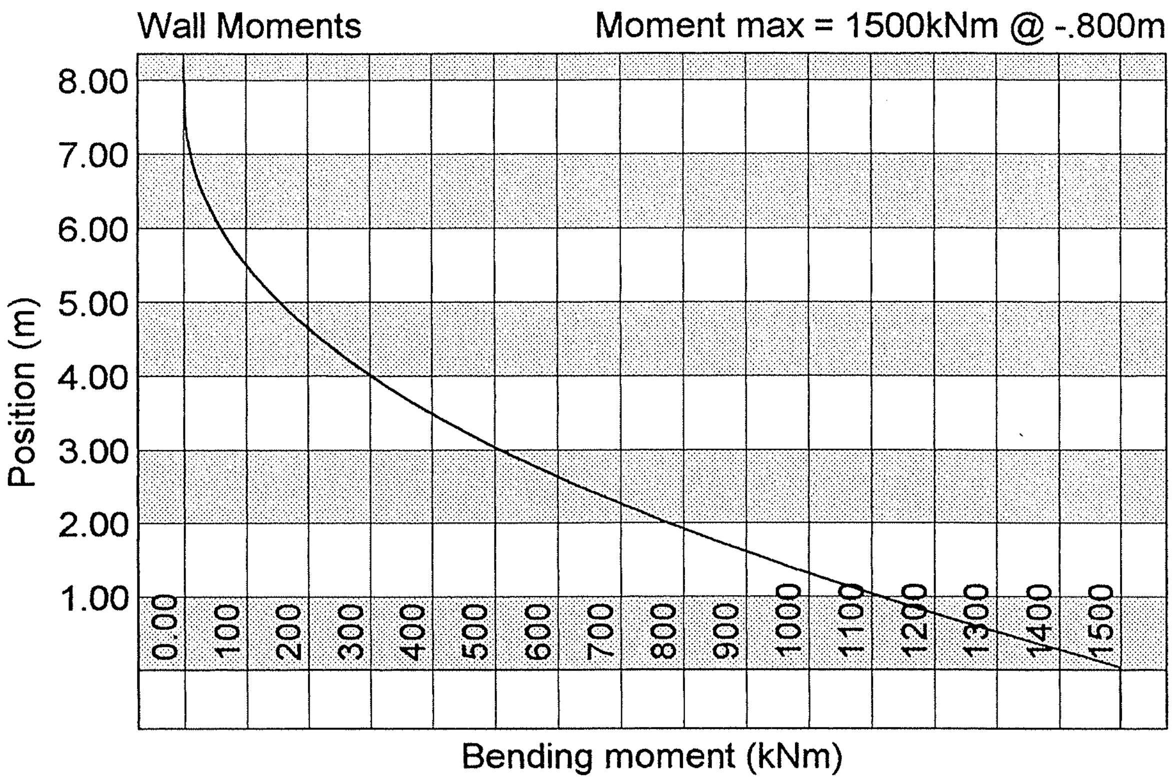
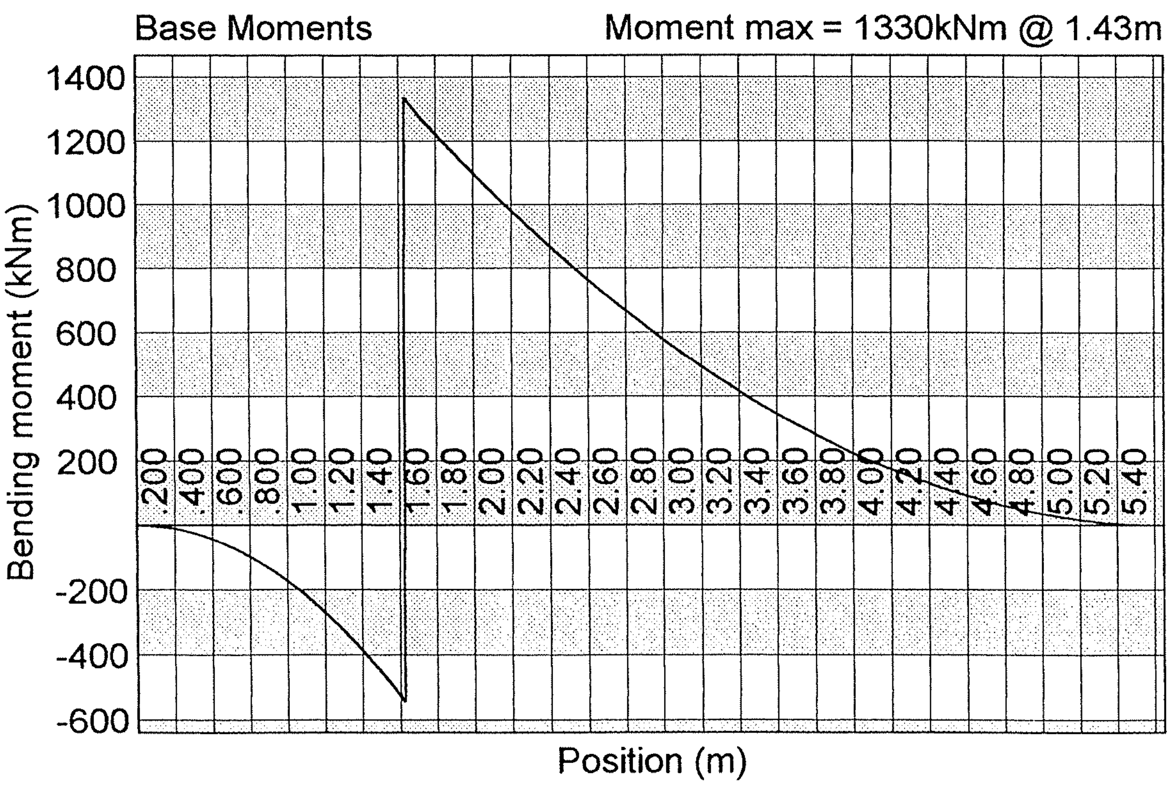
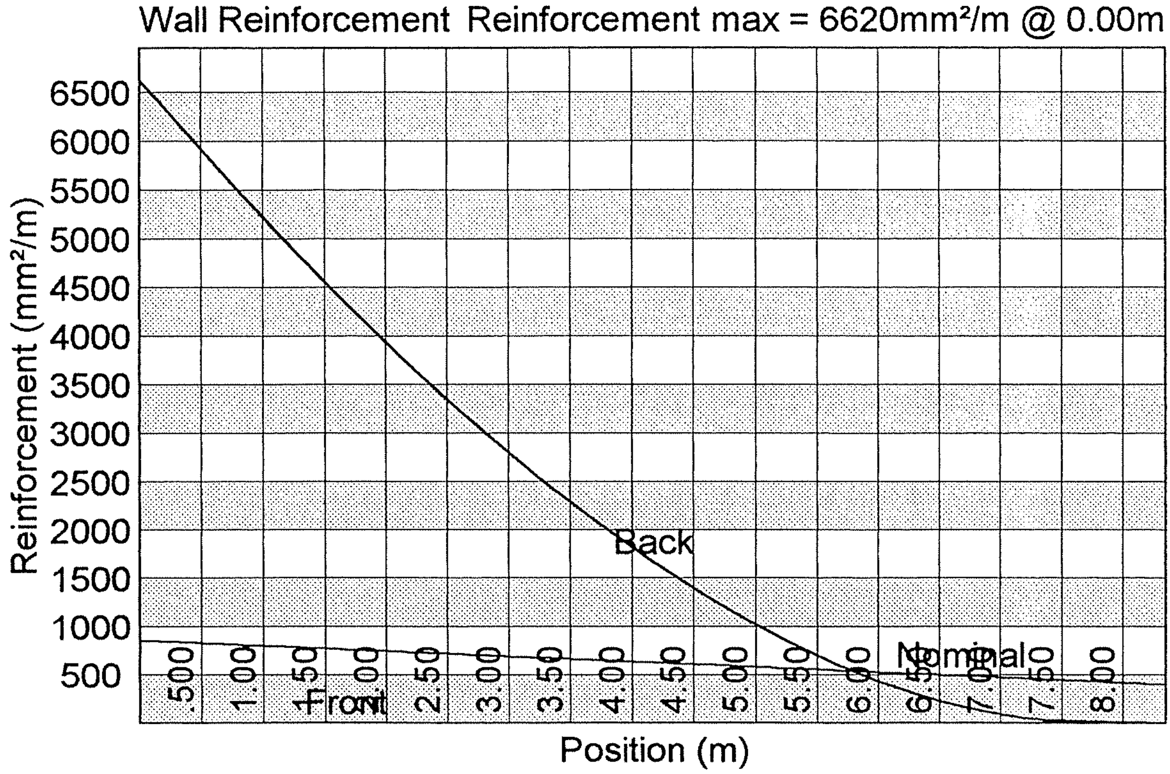
Wall Bending Moments



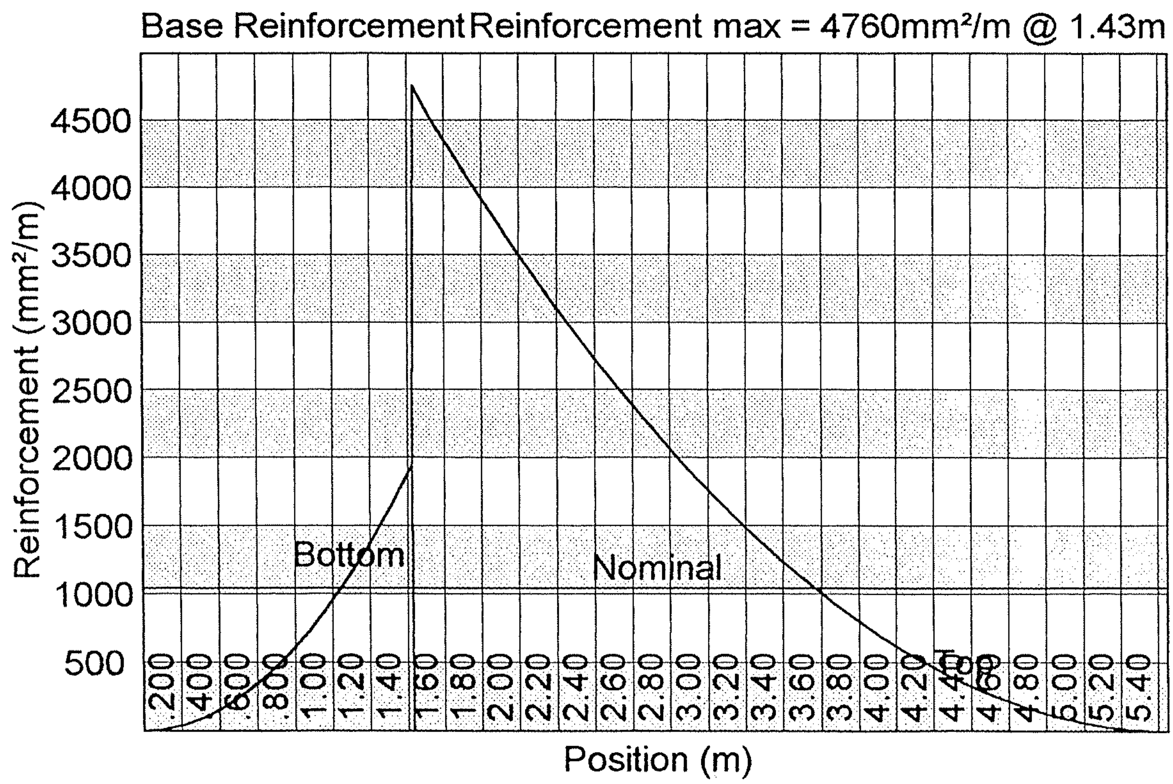
Base Bending Moments



Wall Reinforcement



Base Reinforcement



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	Job Title Dead Sea Parkway - JICA		
	Client Ministry of Tourism		
	Calcs by N.O	Checked by	Date FEB,2000

C14

Retaining Wall Design : Height = 7.5 m

Input Data

Seepage allowed

Theory : Coulomb
Wall type : Cantilever

SEISMIC ANALYSIS SETTINGS:

Seismic Analysis ON/OFF:ON

VALUES OF PRESSURE COEFFICIENTS:

Active Pressure coefficient Ka :0.32
Passive Pressure coefficient Kp :4.98
Seismic Active Pressure coefficient Kas :0.45
Seismic Passive Pressure coefficient Kps :2.04
Base frictional constant μ :0.58

FORCES ACTING ON THE WALL:

FORCES (kN) and their LEVER ARMS (m)

Description	F Horizontal left (+)	Lever arm	F Vertical down (+)	Lever arm
Destabilizing forces:				
Total Active pressure Pa	306.890	3.596	95.643	1.436
Siesmic component of Pa	87.125	5.310	27.593	1.360
As a result of surcharge w	58.351	4.425	18.480	1.439
Siesmic wall inertia	41.583	4.875		
Stabilizing forces:				
Passive pressure on base Pp	-14.891	0.300		
Siesmic component of Pp	21.388	0.540		
Weight of the wall + base			230.375	2.173
Weight of soil on the base			572.563	3.505
UDL of 16.2 kN/m2			67.716	3.400

EQUILIBRIUM CALCULATIONS AT SLS

1.Moment Equilibrium

Point of rotation: bottom front corner of base.

For Overturning moment Mo calculate as follows:
Mo = Sum(hor. forces x l.a.) - Sum(vert. forces x l.a.)
For Stabilizing moment Mr calculate as follows:
Mr = -Sum(hor. forces x l.a.) + Sum(vert. forces x l.a.)
where l.a. = lever arm of each force

```
        Stabilizing moment Mr : 2742.0 kNm
      Destabilizing moment Mo : 1400.5 kNm

Safety factor against overturning = Mr/Mo = 1.96

2.Force Equilibrium

    Sum of Vertical forces Pv : 984.8 kN
  Frictional resistance Pfric : 568.6 kN
Passive Pressure on shear key :  32.2 kN
     Passive pressure on base :  14.9 kN
  => Horizontal resistance Fr : 615.6 kN
  Horizontal sliding force Fh : 406.8 kN

Safety factor against overall sliding = Fr/Fh = 1.51
```

SOIL PRESSURES UNDER BASE

Maximum pressure :482.0 kPa
Minimum pressure : 0 kPa at 1.5 m from right hand side of base.
Maximum pressure occurs at left hand side of base

***** MAXIMUM PRESSURE EXCEEDS SPECIFIED ALLOWABLE PRESSURE BY 82.0 kPa *****

WALL MOMENTS (ULS) AND REINFORCEMENT TO BS8110 - 1997

Position from base top (m)	Moment (kNm)	Front Reinforcing (mm^2/m)	Back Reinforcing (mm^2/m)	Nominal (0.13%) (mm^2/m)
0.00	1816.00	0.00	7393.40	920.40
0.18	1737.18	0.00	7160.90	910.00
0.36	1651.48	0.00	6893.81	899.60
0.53	1568.33	0.00	6630.67	889.20
0.71	1487.71	0.00	6371.51	878.80
0.89	1409.59	0.00	6116.36	868.40
1.07	1333.93	0.00	5865.25	858.00
1.25	1260.71	0.00	5618.22	847.60
1.42	1189.91	0.00	5375.31	837.20
1.60	1121.48	0.00	5136.56	826.80
1.78	1055.41	0.00	4902.02	816.40
1.96	991.66	0.00	4671.72	806.00
2.14	930.20	0.00	4445.72	795.60
2.31	871.02	0.00	4224.07	785.20
2.49	814.07	0.00	4006.81	774.80
2.67	759.33	0.00	3794.02	764.40
2.85	706.77	0.00	3585.74	754.00
3.03	656.37	0.00	3382.05	743.60
3.20	608.09	0.00	3183.01	733.20
3.38	561.90	0.00	2988.70	722.80
3.56	517.78	0.00	2799.18	712.40
3.74	475.70	0.00	2614.55	702.00
3.92	435.63	0.00	2434.90	691.60
4.09	397.54	0.00	2260.32	681.20
4.27	361.41	0.00	2090.90	670.80
4.45	327.19	0.00	1926.76	660.40
4.63	294.87	0.00	1768.02	650.00
4.81	264.42	0.00	1614.80	639.60
4.98	235.81	0.00	1467.23	629.20
5.16	209.01	0.00	1325.46	618.80
5.34	183.98	0.00	1189.65	608.40
5.52	160.71	0.00	1059.96	598.00
5.70	139.16	0.00	936.58	587.60
5.87	119.31	0.00	819.70	577.20
6.05	101.13	0.00	709.54	566.80
6.23	84.58	0.00	606.33	556.40
6.41	69.64	0.00	510.33	546.00
6.59	56.28	0.00	421.81	535.60

6.94	34.19	0.00	268.45	514.80
7.12	25.40	0.00	204.31	504.40
7.30	18.08	0.00	149.05	494.00
7.48	12.20	0.00	103.12	483.60
7.65	7.72	0.00	66.99	473.20
7.83	4.62	0.00	41.22	462.80
8.01	2.88	0.00	26.37	452.40
8.19	1.85	0.00	17.46	442.00
8.37	1.05	0.00	10.21	431.60
8.54	0.48	0.00	4.77	421.20
8.72	0.13	0.00	1.32	410.80
8.90	0.01	0.00	0.05	400.40

BASE MOMENTS (ULS) AND REINFORCEMENT TO BS8110 - 1997

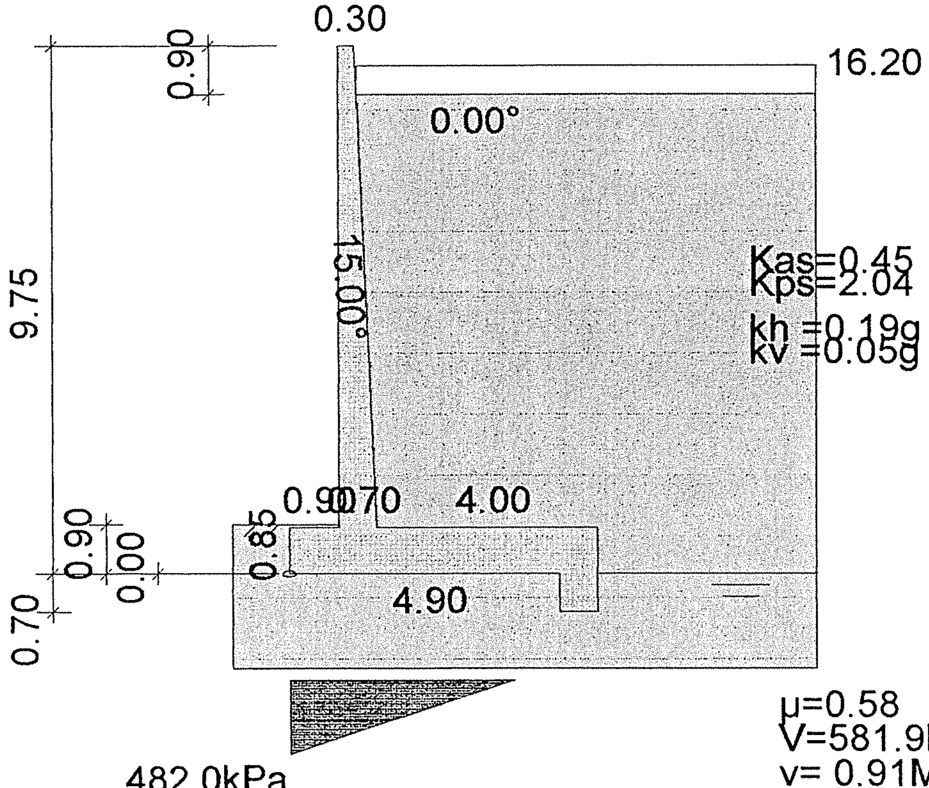
Position from left (m)	Moment (kNm)	Top Reinforcing (mm^2/m)	Bot Reinforcing (mm^2/m)	Nominal (0.13%) (mm^2/m)
0.11	-0.00	0.00	0.00	1105.00
0.22	-4.46	0.00	14.88	1105.00
0.34	-17.83	0.00	59.53	1105.00
0.45	-40.11	0.00	133.94	1105.00
0.56	-71.31	0.00	238.12	1105.00
0.67	-111.41	0.00	372.06	1105.00
0.78	-160.44	0.00	535.77	1105.00
0.90	-218.37	0.00	729.25	1105.00
1.01	-285.22	0.00	952.48	1105.00
1.12	-360.98	0.00	1205.49	1105.00
1.23	-445.66	0.00	1488.26	1105.00
1.34	-539.25	0.00	1800.79	1105.00
1.46	-555.12	1853.79	0.00	1105.00
1.57	1721.29	5748.20	0.00	1105.00
1.68	1647.71	5502.45	0.00	1105.00
1.79	1562.13	5216.66	0.00	1105.00
1.90	1478.83	4938.49	0.00	1105.00
2.02	1397.81	4667.94	0.00	1105.00
2.13	1319.08	4405.01	0.00	1105.00
2.24	1242.63	4149.70	0.00	1105.00
2.35	1168.46	3902.02	0.00	1105.00
2.46	1096.57	3661.95	0.00	1105.00
2.58	1026.96	3429.51	0.00	1105.00
2.69	959.64	3204.68	0.00	1105.00
2.80	894.60	2987.48	0.00	1105.00
2.91	831.84	2777.90	0.00	1105.00
3.02	771.36	2575.94	0.00	1105.00
3.14	713.17	2381.60	0.00	1105.00
3.25	657.26	2194.88	0.00	1105.00
3.36	603.63	2015.79	0.00	1105.00
3.47	552.28	1844.31	0.00	1105.00
3.58	503.21	1680.46	0.00	1105.00
3.70	456.43	1524.22	0.00	1105.00
3.81	411.93	1375.61	0.00	1105.00
3.92	369.71	1234.62	0.00	1105.00
4.03	329.77	1101.25	0.00	1105.00
4.14	292.11	975.50	0.00	1105.00
4.26	256.74	857.38	0.00	1105.00
4.37	223.65	746.87	0.00	1105.00
4.48	192.84	643.99	0.00	1105.00
4.59	164.31	548.72	0.00	1105.00
4.70	138.07	461.08	0.00	1105.00
4.82	114.11	381.06	0.00	1105.00
4.93	92.43	308.66	0.00	1105.00
5.04	73.03	243.88	0.00	1105.00
5.15	55.91	186.72	0.00	1105.00
5.26	41.08	137.18	0.00	1105.00
5.38	28.53	95.26	0.00	1105.00
5.49	18.26	60.97	0.00	1105.00
5.60	10.27	34.30	0.00	1105.00

```
SHEAR CHECK AT WALL-BASE JUNCTION TO BS8110 - 1997

 Shear force at bottom of wall V = 581.9 kN
Shear stress at bottom of wall v = 0.91 MPa INADEQUATE
       Allowable shear stress vc = 0.66 MPa (based on Wall tensile reinf.)
```

Sketch of Wall

Design code: BS8110 - 1997



Wall type: Cantilever
Theory: Coulomb

SFslip = 1.51
SFovt = 1.96

μ=0.58
V=581.9kN
v= 0.91MPa
vc= 0.66MPa

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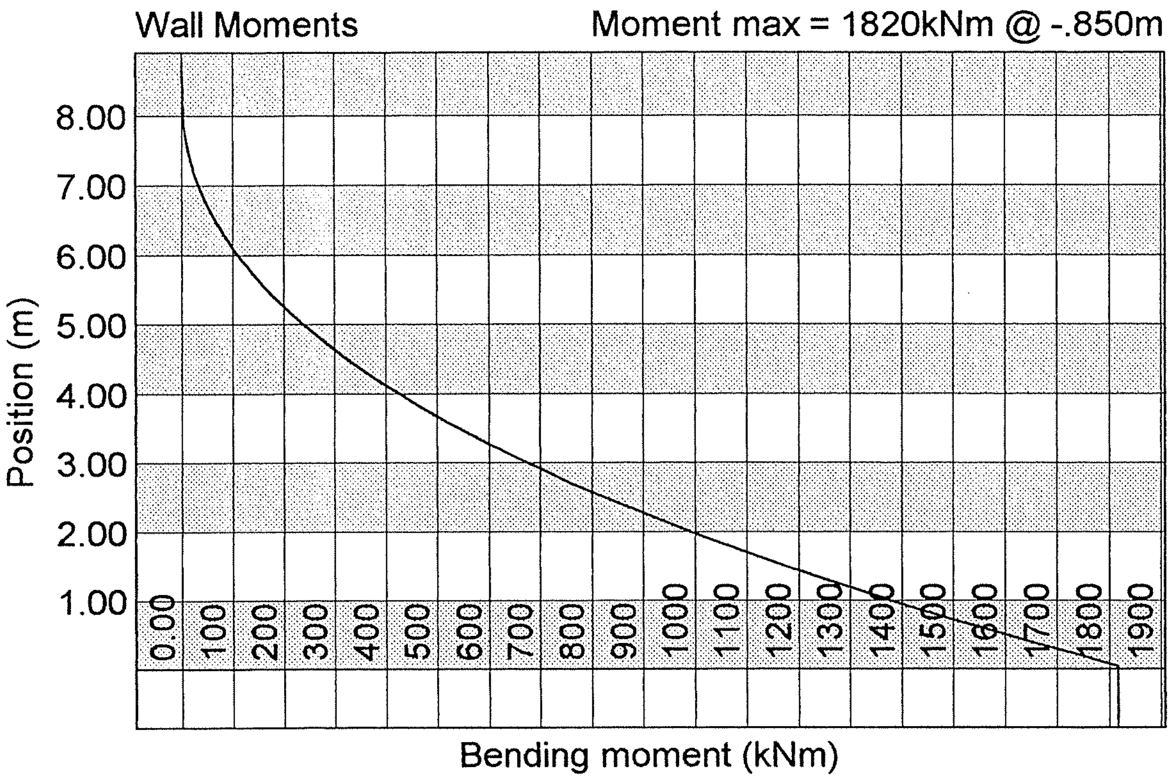
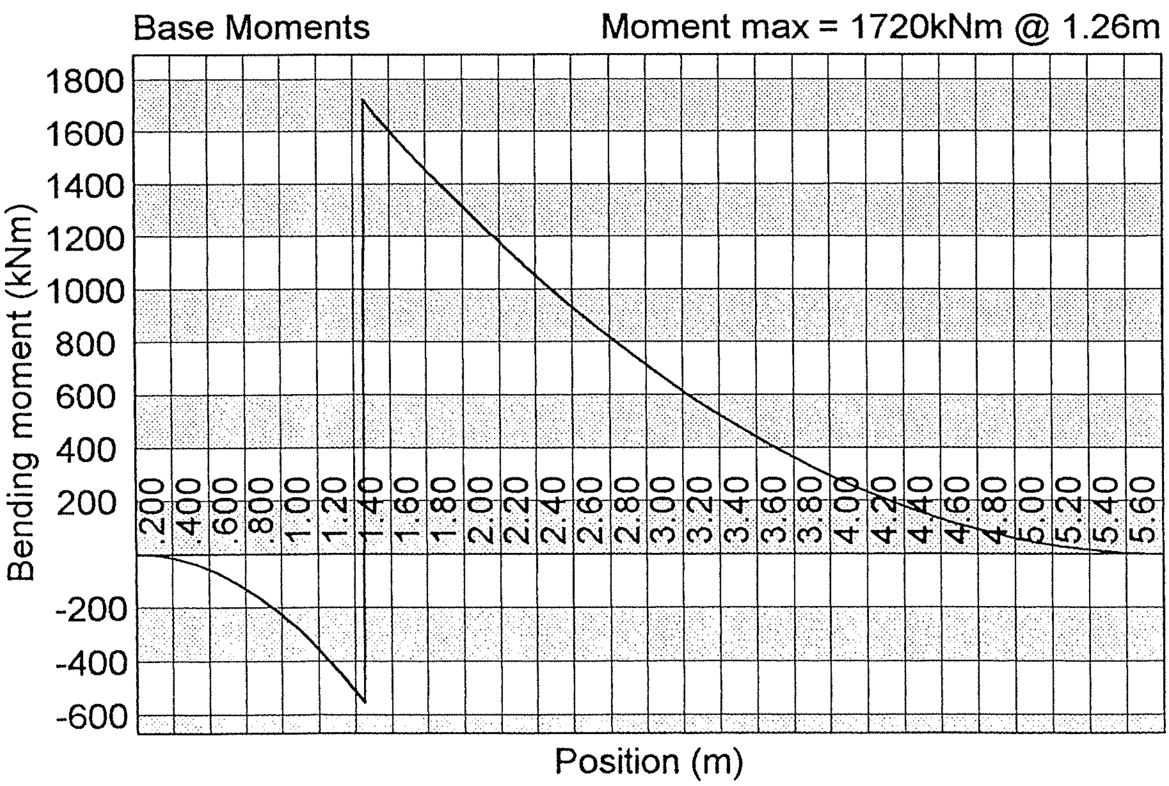
Wall Bending Moments



Base Bending Moments



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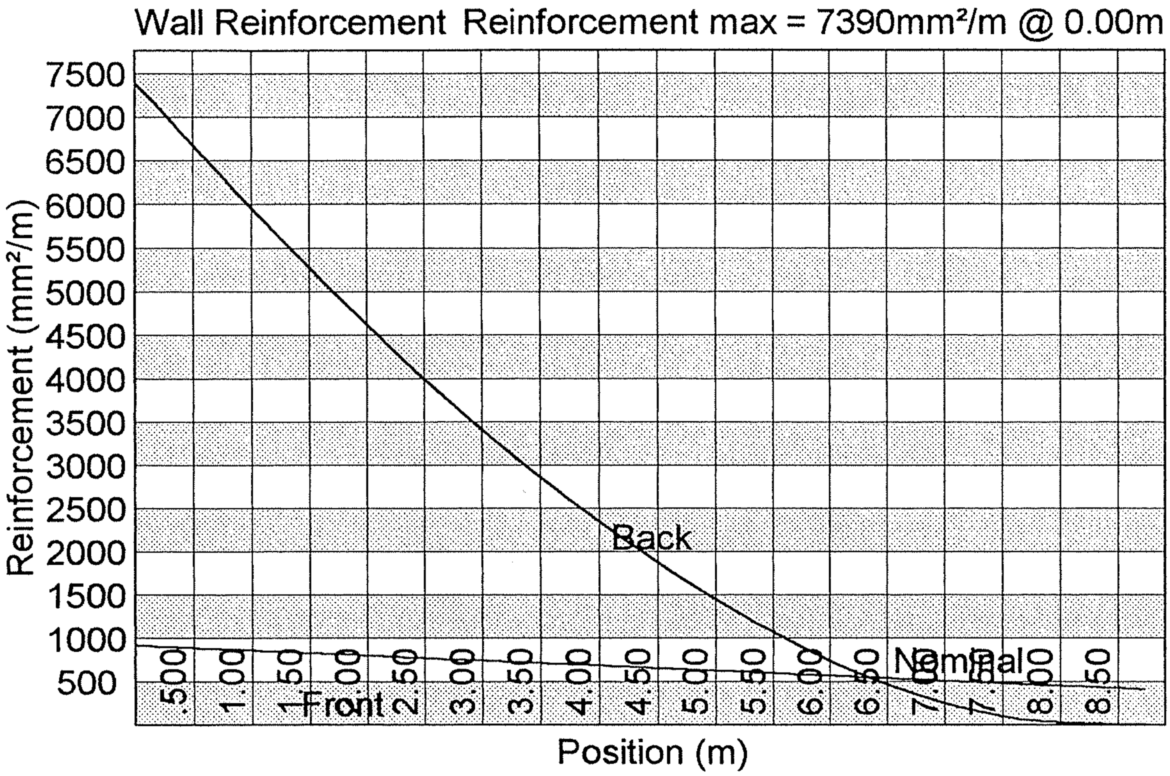
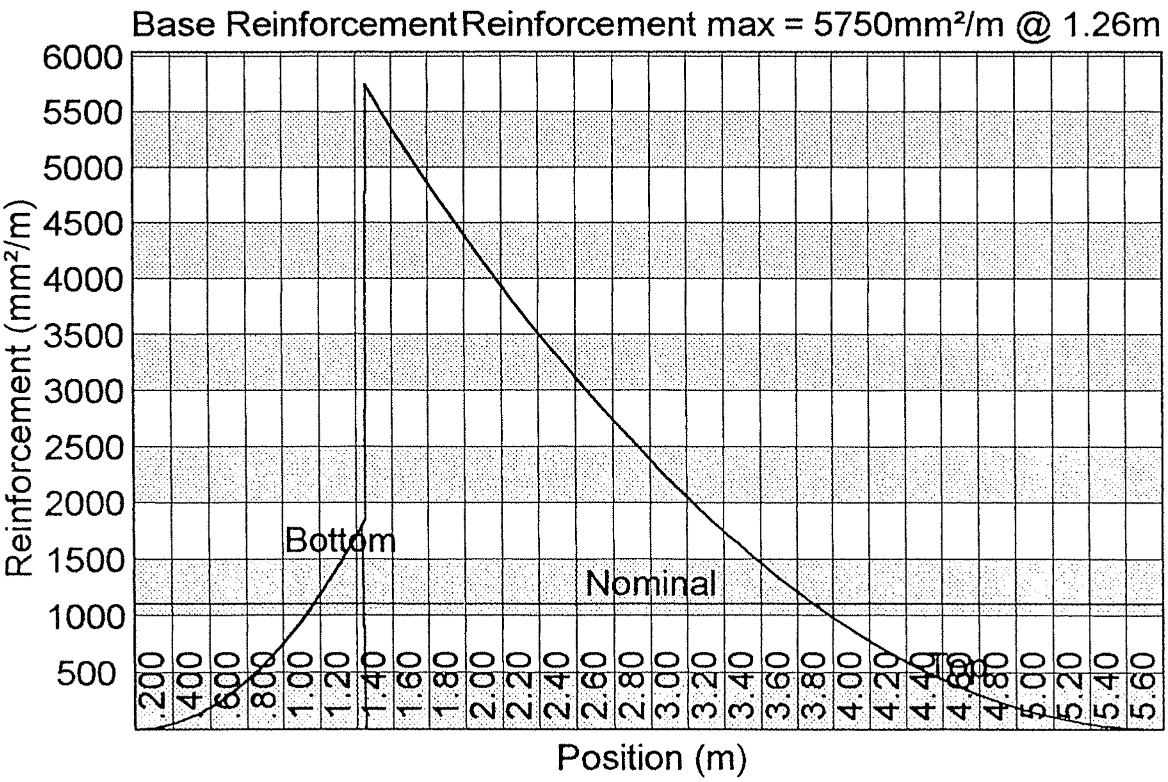
Wall Reinforcement



Base Reinforcement



C14

Retaining Wall Design : Height = 8.0 m

Input Data

Seepage allowed

Theory : Coulomb
Wall type : Cantilever

SEISMIC ANALYSIS SETTINGS:

Seismic Analysis ON/OFF:ON

VALUES OF PRESSURE COEFFICIENTS:

Active Pressure coefficient Ka :0.32
Passive Pressure coefficient Kp :4.98
Seismic Active Pressure coefficient Kas :0.45
Seismic Passive Pressure coefficient Kps :2.04
Base frictional constant μ :0.58

FORCES ACTING ON THE WALL:

FORCES (kN) and their LEVER ARMS (m)

Description	F Horizontal left (+)	Lever arm	F Vertical down (+)	Lever arm
Destabilizing forces:				
Total Active pressure Pa	346.790	3.819	109.221	1.565
Siesmic component of Pa	98.287	5.640	31.445	1.480
As a result of surcharge w	62.105	4.700	19.869	1.568
Siesmic wall inertia	46.555	5.150		
Stabilizing forces:				
Passive pressure on base Pp	-14.891	0.300		
Siesmic component of Pp	21.388	0.540		
Weight of the wall + base			257.925	2.345
Weight of soil on the base			647.310	3.772
UDL of 16.2 kN/m2			72.333	3.650

EQUILIBRIUM CALCULATIONS AT SLS

1.Moment Equilibrium

Point of rotation: bottom front corner of base.

For Overturning moment Mo calculate as follows:
  Mo = Sum(hor. forces x l.a.) - Sum(vert. forces x l.a.)
For Stabilizing moment Mr calculate as follows:
  Mr = -Sum(hor. forces x l.a.) + Sum(vert. forces x l.a.)
where l.a. = lever arm of each force

```
        Stabilizing moment Mr : 3314.7 kNm
      Destabilizing moment Mo : 1654.0 kNm

Safety factor against overturning = Mr/Mo = 2.00

2.Force Equilibrium

    Sum of Vertical forces Pv : 1106.7 kN
  Frictional resistance Pfric : 638.9 kN
Passive Pressure on shear key :  32.2 kN
     Passive pressure on base :  14.9 kN
  => Horizontal resistance Fr : 686.0 kN
  Horizontal sliding force Fh : 455.4 kN

Safety factor against overall sliding = Fr/Fh = 1.51
```

SOIL PRESSURES UNDER BASE

Maximum pressure :491.6 kPa
Minimum pressure : 0 kPa at 1.5 m from right hand side of base.
Maximum pressure occurs at left hand side of base

***** MAXIMUM PRESSURE EXCEEDS SPECIFIED ALLOWABLE PRESSURE BY 91.6 kPa *****

WALL MOMENTS (ULS) AND REINFORCEMENT TO BS8110 - 1997

Position from base top (m)	Moment (kNm)	Front Reinforcing (mm²/m)	Back Reinforcing (mm²/m)	Nominal (0.13%) (mm²/m)
0.00	2153.61	0.00	8128.18	986.70
0.19	2060.95	0.00	7879.93	975.00
0.38	1959.66	0.00	7591.66	963.30
0.56	1861.37	0.00	7307.49	951.60
0.75	1766.07	0.00	7027.44	939.90
0.94	1673.70	0.00	6751.55	928.20
1.13	1584.24	0.00	6479.86	916.50
1.32	1497.66	0.00	6212.40	904.80
1.50	1413.92	0.00	5949.21	893.10
1.69	1332.98	0.00	5690.34	881.40
1.88	1254.82	0.00	5435.84	869.70
2.07	1179.39	0.00	5185.74	858.00
2.26	1106.67	0.00	4940.10	846.30
2.44	1036.63	0.00	4698.97	834.60
2.63	969.22	0.00	4462.41	822.90
2.82	904.41	0.00	4230.49	811.20
3.01	842.18	0.00	4003.26	799.50
3.20	782.48	0.00	3780.79	787.80
3.38	725.28	0.00	3563.16	776.10
3.57	670.55	0.00	3350.45	764.40
3.76	618.26	0.00	3142.74	752.70
3.95	568.37	0.00	2940.11	741.00
4.14	520.85	0.00	2742.68	729.30
4.32	475.65	0.00	2550.53	717.60
4.51	432.76	0.00	2363.78	705.90
4.70	392.14	0.00	2182.55	694.20
4.89	353.75	0.00	2006.98	682.50
5.08	317.55	0.00	1837.19	670.80
5.26	283.52	0.00	1673.33	659.10
5.45	251.62	0.00	1515.58	647.40
5.64	221.82	0.00	1364.11	635.70
5.83	194.08	0.00	1219.10	624.00
6.02	168.37	0.00	1080.76	612.30
6.20	144.66	0.00	949.33	600.60
6.39	122.91	0.00	825.05	588.90
6.58	103.08	0.00	708.19	577.20
6.77	85.15	0.00	599.04	565.50
6.96	69.08	0.00	497.94	553.80

7.33	42.39	0.00	321.32	530.40
7.52	31.69	0.00	246.65	518.70
7.71	22.73	0.00	181.69	507.00
7.90	15.45	0.00	126.99	495.30
8.08	9.83	0.00	83.15	483.60
8.27	5.84	0.00	50.84	471.90
8.46	3.43	0.00	30.82	460.20
8.65	2.19	0.00	20.29	448.50
8.84	1.24	0.00	11.90	436.80
9.02	0.56	0.00	5.58	425.10
9.21	0.15	0.00	1.55	413.40
9.40	0.01	0.00	0.06	401.70

BASE MOMENTS (ULS) AND REINFORCEMENT TO BS8110 - 1997

Position from left (m)	Moment (kNm)	Top Reinforcing (mm²/m)	Bot Reinforcing (mm²/m)	Nominal (0.13%) (mm²/m)
0.12	-0.00	0.00	0.00	1170.00
0.24	-5.13	0.00	16.10	1170.00
0.36	-20.51	0.00	64.41	1170.00
0.48	-46.15	0.00	144.93	1170.00
0.60	-82.04	0.00	257.65	1170.00
0.72	-128.18	0.00	402.58	1170.00
0.84	-184.58	0.00	579.72	1170.00
0.96	-251.24	0.00	789.06	1170.00
1.08	-328.15	0.00	1030.61	1170.00
1.20	-415.31	0.00	1304.37	1170.00
1.32	-512.73	0.00	1610.33	1170.00
1.44	-620.41	0.00	1948.50	1170.00
1.56	-673.18	2114.26	0.00	1170.00
1.68	2027.22	6366.86	0.00	1170.00
1.80	1970.64	6189.16	0.00	1170.00
1.92	1868.29	5867.70	0.00	1170.00
2.04	1768.66	5554.82	0.00	1170.00
2.16	1671.77	5250.50	0.00	1170.00
2.28	1577.60	4954.76	0.00	1170.00
2.40	1486.17	4667.59	0.00	1170.00
2.52	1397.46	4388.99	0.00	1170.00
2.64	1311.49	4118.96	0.00	1170.00
2.76	1228.24	3857.51	0.00	1170.00
2.88	1147.72	3604.63	0.00	1170.00
3.00	1069.93	3360.32	0.00	1170.00
3.12	994.87	3124.58	0.00	1170.00
3.24	922.54	2897.42	0.00	1170.00
3.36	852.94	2678.83	0.00	1170.00
3.48	786.07	2468.81	0.00	1170.00
3.60	721.93	2267.36	0.00	1170.00
3.72	660.52	2074.48	0.00	1170.00
3.84	601.84	1890.18	0.00	1170.00
3.96	545.88	1714.45	0.00	1170.00
4.08	492.66	1547.29	0.00	1170.00
4.20	442.17	1388.70	0.00	1170.00
4.32	394.40	1238.69	0.00	1170.00
4.44	349.37	1097.25	0.00	1170.00
4.56	307.06	964.38	0.00	1170.00
4.68	267.48	840.08	0.00	1170.00
4.80	230.64	724.35	0.00	1170.00
4.92	196.52	617.20	0.00	1170.00
5.04	165.13	518.62	0.00	1170.00
5.16	136.47	428.61	0.00	1170.00
5.28	110.54	347.18	0.00	1170.00
5.40	87.34	274.31	0.00	1170.00
5.52	66.87	210.02	0.00	1170.00
5.64	49.13	154.30	0.00	1170.00
5.76	34.12	107.15	0.00	1170.00
5.88	21.84	68.58	0.00	1170.00
6.00	12.28	38.58	0.00	1170.00

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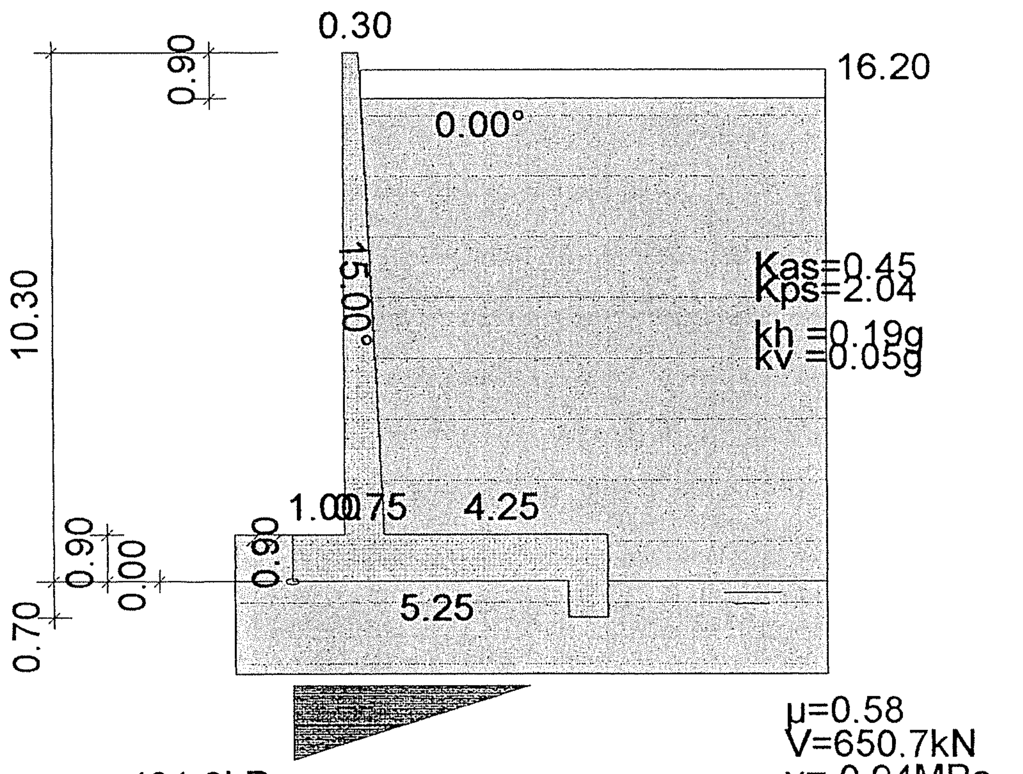
SHEAR CHECK AT WALL-BASE JUNCTION TO BS8110 - 1997

```
 Shear force at bottom of wall V = 650.7 kN
Shear stress at bottom of wall v = 0.94 MPa INADEQUATE
       Allowable shear stress vc = 0.67 MPa (based on Wall tensile reinf.)
```

Sketch of Wall

Design code: BS8110 - 1997



Wall type: Cantilever SFslip = 1.51
Theory: Coulomb SFovt = 2.00

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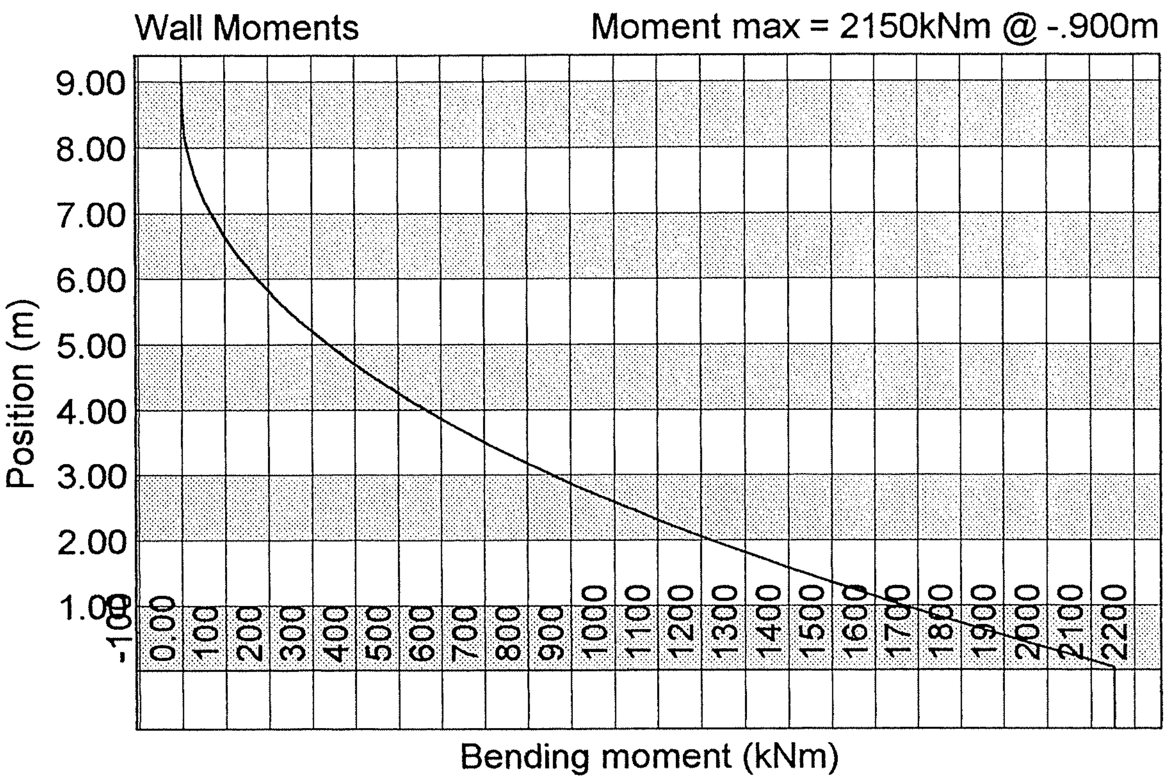
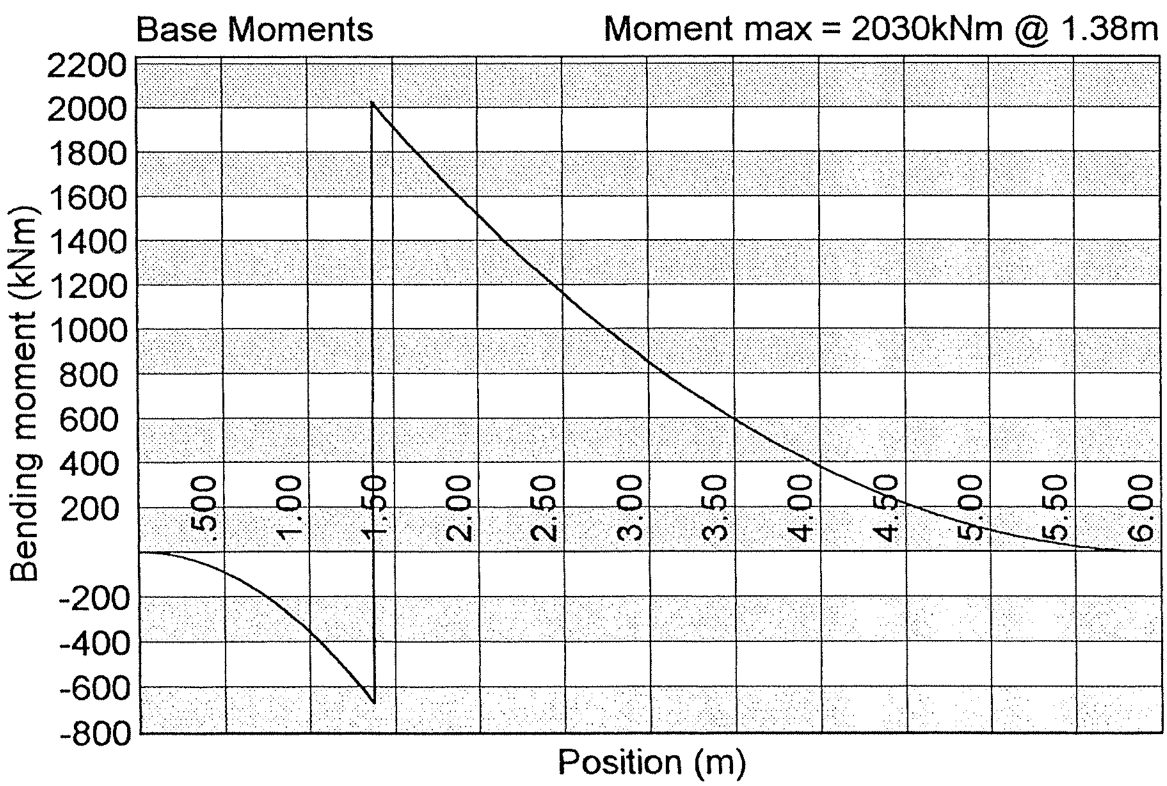
Wall Bending Moments



Base Bending Moments



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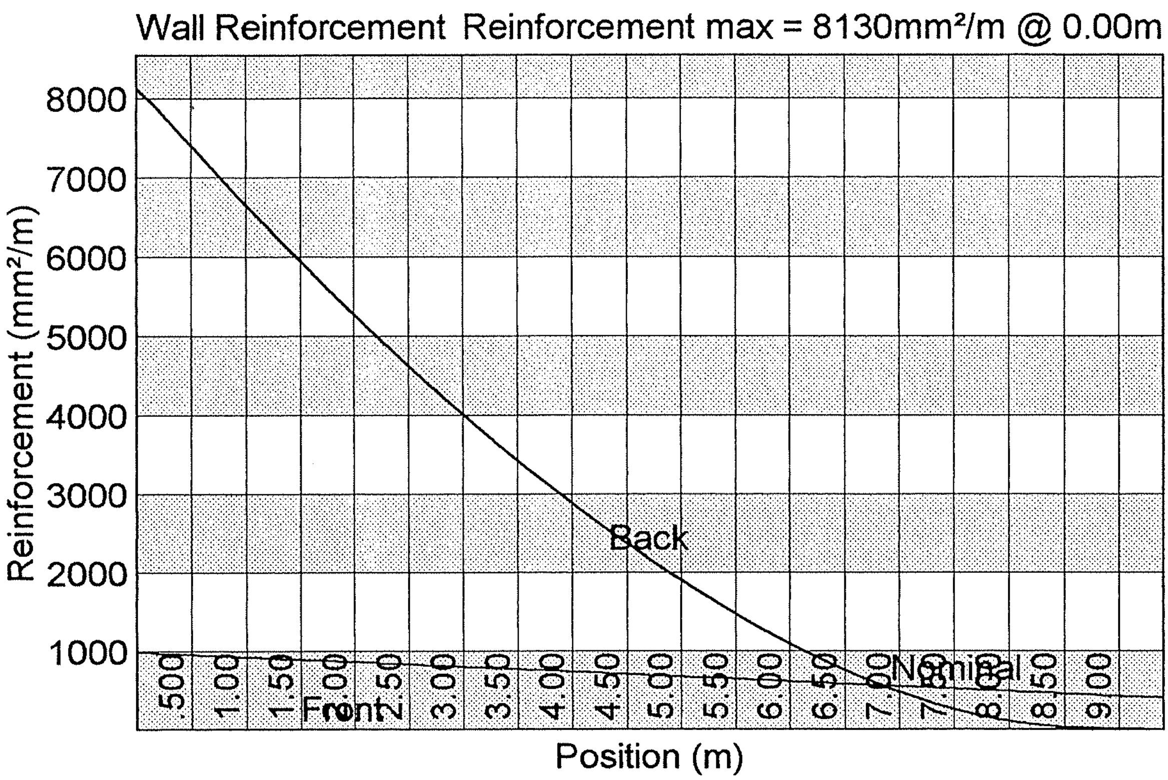
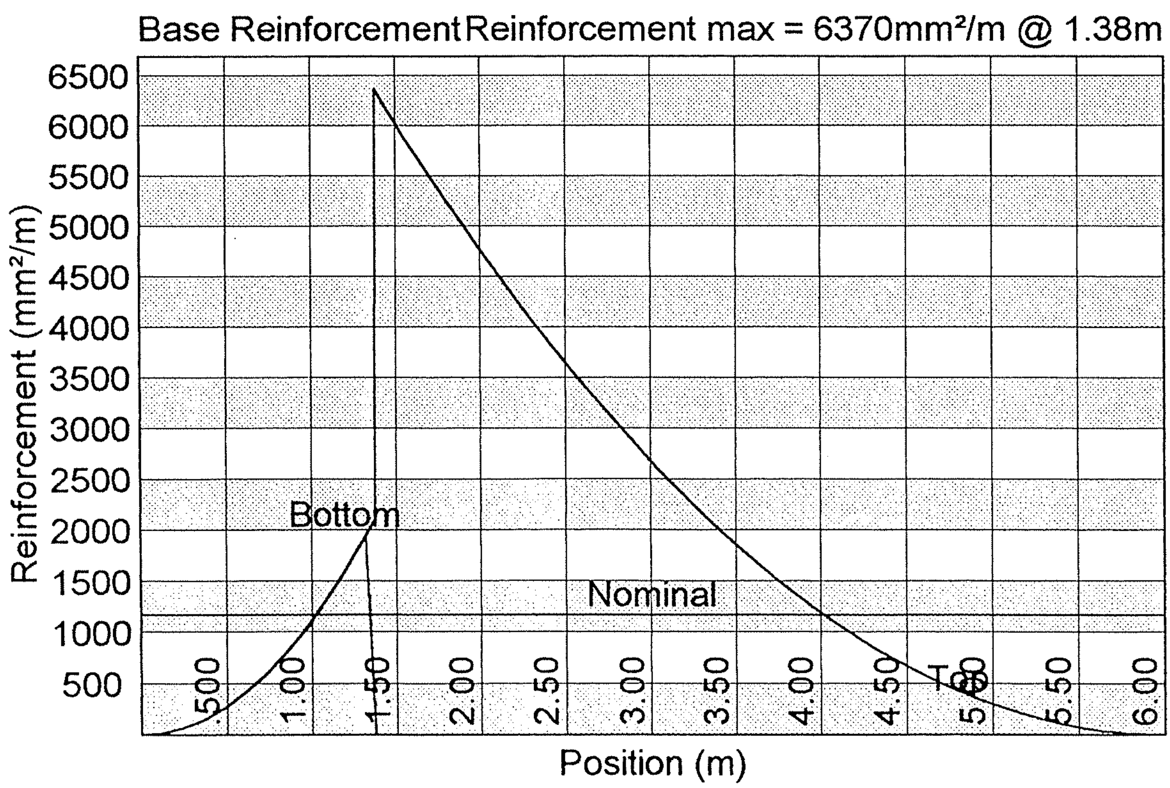
	Client Ministry of Tourism		
	Calcs by N.O	Checked by	Date FEB,2000

Wall Reinforcement



Base Reinforcement



C14

Retaining Wall Design : Height = 8.5 m

Input Data

Seepage allowed

Theory : Coulomb
Wall type : Cantilever

SEISMIC ANALYSIS SETTINGS:

Seismic Analysis ON/OFF:ON

VALUES OF PRESSURE COEFFICIENTS:

Active Pressure coefficient Ka :0.32
Passive Pressure coefficient Kp :4.98
Seismic Active Pressure coefficient Kas :0.45
Seismic Passive Pressure coefficient Kps :2.04
Base frictional constant μ :0.58

FORCES ACTING ON THE WALL:

FORCES (kN) and their LEVER ARMS (m)

Description	F Horizontal left (+)	Lever arm	F Vertical down (+)	Lever arm
Destabilizing forces:				
Total Active pressure Pa	389.129	4.044	123.711	1.795
Siesmic component of Pa	110.122	5.970	35.550	1.700
As a result of surcharge w	65.859	4.975	21.261	1.797
Siesmic wall inertia	52.214	5.425		
Stabilizing forces:				
Passive pressure on base Pp	-14.891	0.300		
Siesmic component of Pp	21.388	0.540		
Weight of the wall + base			289.275	2.596
Weight of soil on the base			727.527	4.135
UDL of 16.2 kN/m2			76.950	4.000

EQUILIBRIUM CALCULATIONS AT SLS

1.Moment Equilibrium

Point of rotation: bottom front corner of base.

For Overturning moment Mo calculate as follows:
Mo = Sum(hor. forces x l.a.) - Sum(vert. forces x l.a.)
For Stabilizing moment Mr calculate as follows:
Mr = -Sum(hor. forces x l.a.) + Sum(vert. forces x l.a.)
where l.a. = lever arm of each force

```
        Stabilizing moment Mr : 4071.2 kNm
      Destabilizing moment Mo : 1924.5 kNm

Safety factor against overturning = Mr/Mo = 2.12

2.Force Equilibrium

    Sum of Vertical forces Pv : 1238.7 kN
  Frictional resistance Pfric : 715.2 kN
Passive Pressure on shear key :  32.2 kN
     Passive pressure on base :  14.9 kN
  => Horizontal resistance Fr : 762.2 kN
  Horizontal sliding force Fh : 507.2 kN

Safety factor against overall sliding = Fr/Fh = 1.50
```

SOIL PRESSURES UNDER BASE

Maximum pressure :476.5 kPa
Minimum pressure : 0 kPa at 1.3 m from right hand side of base.
Maximum pressure occurs at left hand side of base

***** MAXIMUM PRESSURE EXCEEDS SPECIFIED ALLOWABLE PRESSURE BY 76.5 kPa *****

WALL MOMENTS (ULS) AND REINFORCEMENT TO BS8110 - 1997

Position from base top (m)	Moment (kNm)	Front Reinforcing (mm^2/m)	Back Reinforcing (mm^2/m)	Nominal (0.13%) (mm^2/m)
0.00	2534.63	0.00	8915.74	1053.00
0.20	2426.45	0.00	8650.53	1040.00
0.40	2307.63	0.00	8339.64	1027.00
0.59	2192.33	0.00	8033.00	1014.00
0.79	2080.52	0.00	7730.66	1001.00
0.99	1972.14	0.00	7432.64	988.00
1.19	1867.16	0.00	7138.99	975.00
1.39	1765.55	0.00	6849.74	962.00
1.58	1667.25	0.00	6564.94	949.00
1.78	1572.24	0.00	6284.62	936.00
1.98	1480.47	0.00	6008.84	923.00
2.18	1391.91	0.00	5737.65	910.00
2.38	1306.51	0.00	5471.09	897.00
2.57	1224.23	0.00	5209.23	884.00
2.77	1145.03	0.00	4952.12	871.00
2.97	1068.88	0.00	4699.83	858.00
3.17	995.74	0.00	4452.43	845.00
3.37	925.56	0.00	4209.98	832.00
3.56	858.31	0.00	3972.57	819.00
3.76	793.94	0.00	3740.29	806.00
3.96	732.42	0.00	3513.21	793.00
4.16	673.71	0.00	3291.44	780.00
4.36	617.77	0.00	3075.08	767.00
4.55	564.56	0.00	2864.24	754.00
4.75	514.03	0.00	2659.04	741.00
4.95	466.16	0.00	2459.61	728.00
5.15	420.89	0.00	2266.10	715.00
5.35	378.20	0.00	2078.65	702.00
5.54	338.03	0.00	1897.44	689.00
5.74	300.36	0.00	1722.63	676.00
5.94	265.14	0.00	1554.42	663.00
6.14	232.33	0.00	1393.04	650.00
6.34	201.90	0.00	1238.70	637.00
6.53	173.80	0.00	1091.68	624.00
6.73	147.99	0.00	952.24	611.00
6.93	124.43	0.00	820.69	598.00
7.13	103.09	0.00	697.38	585.00
7.33	83.93	0.00	582.69	572.00

7.72	51.97	0.00	380.83	546.00
7.92	39.09	0.00	294.66	533.00
8.12	28.23	0.00	219.06	520.00
8.32	19.35	0.00	154.70	507.00
8.51	12.41	0.00	102.28	494.00
8.71	7.36	0.00	62.65	481.00
8.91	4.17	0.00	36.71	468.00
9.11	2.59	0.00	23.54	455.00
9.31	1.47	0.00	13.84	442.00
9.50	0.67	0.00	6.51	429.00
9.70	0.18	0.00	1.82	416.00
9.90	0.01	0.00	0.07	403.00

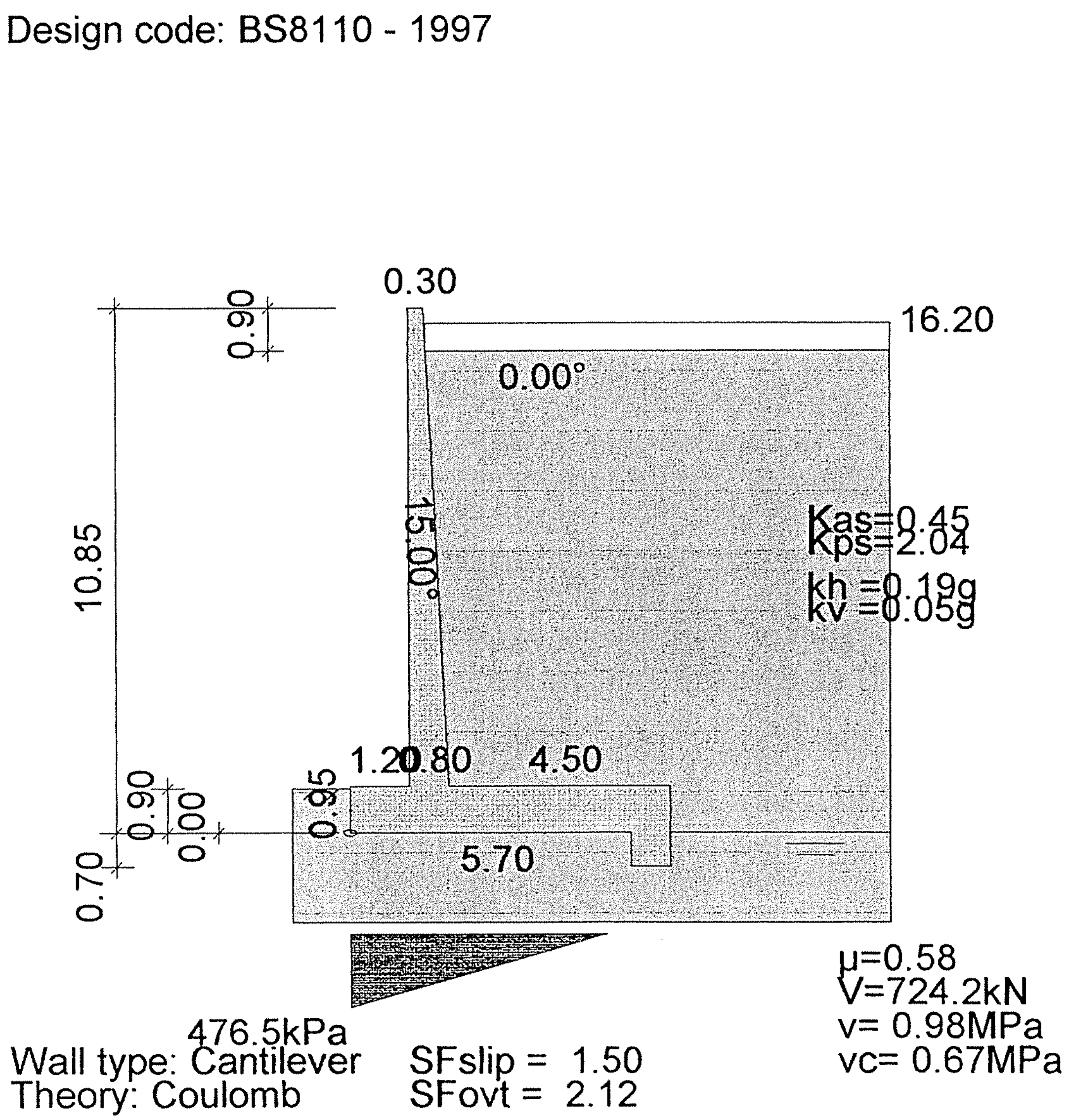
BASE MOMENTS (ULS) AND REINFORCEMENT TO BS8110 - 1997

Position from left (m)	Moment (kNm)	Top Reinforcing (mm^2/m)	Bot Reinforcing (mm^2/m)	Nominal (0.13%) (mm^2/m)
0.13	-0.00	0.00	0.00	1235.00
0.26	-5.62	0.00	16.65	1235.00
0.39	-22.47	0.00	66.61	1235.00
0.52	-50.56	0.00	149.86	1235.00
0.65	-89.88	0.00	266.42	1235.00
0.78	-140.43	0.00	416.28	1235.00
0.91	-202.23	0.00	599.45	1235.00
1.04	-275.25	0.00	815.91	1235.00
1.17	-359.51	0.00	1065.68	1235.00
1.30	-455.01	0.00	1348.75	1235.00
1.43	-561.74	0.00	1665.13	1235.00
1.56	-679.70	0.00	2014.81	1235.00
1.69	-808.90	0.00	2397.79	1235.00
1.82	-850.92	2522.33	0.00	1235.00
1.95	2327.64	6899.69	0.00	1235.00
2.08	2242.92	6648.56	0.00	1235.00
2.21	2123.32	6294.04	0.00	1235.00
2.34	2007.00	5949.23	0.00	1235.00
2.47	1893.95	5614.13	0.00	1235.00
2.60	1784.18	5288.74	0.00	1235.00
2.73	1677.69	4973.07	0.00	1235.00
2.86	1574.47	4667.11	0.00	1235.00
2.99	1474.53	4370.86	0.00	1235.00
3.12	1377.87	4084.33	0.00	1235.00
3.25	1284.48	3807.50	0.00	1235.00
3.38	1194.37	3540.40	0.00	1235.00
3.51	1107.54	3283.00	0.00	1235.00
3.64	1023.98	3035.32	0.00	1235.00
3.77	943.70	2797.35	0.00	1235.00
3.90	866.70	2569.09	0.00	1235.00
4.03	792.97	2350.55	0.00	1235.00
4.16	722.52	2141.72	0.00	1235.00
4.29	655.35	1942.60	0.00	1235.00
4.42	591.45	1753.20	0.00	1235.00
4.55	530.83	1573.51	0.00	1235.00
4.68	473.49	1403.53	0.00	1235.00
4.81	419.42	1243.27	0.00	1235.00
4.94	368.63	1092.72	0.00	1235.00
5.07	321.12	951.88	0.00	1235.00
5.20	276.88	820.75	0.00	1235.00
5.33	235.92	699.34	0.00	1235.00
5.46	198.24	587.64	0.00	1235.00
5.59	163.84	485.65	0.00	1235.00
5.72	132.71	393.38	0.00	1235.00
5.85	104.86	310.82	0.00	1235.00
5.98	80.28	237.97	0.00	1235.00
6.11	58.98	174.83	0.00	1235.00
6.24	40.96	121.41	0.00	1235.00
6.37	26.21	77.70	0.00	1235.00
6.50	14.75	43.71	0.00	1235.00

SHEAR CHECK AT WALL-BASE JUNCTION TO BS8110 - 1997

```
 Shear force at bottom of wall V = 724.2 kN
Shear stress at bottom of wall v = 0.98 MPa INADEQUATE
       Allowable shear stress vc = 0.67 MPa (based on Wall tensile reinf.)
```

Sketch of Wall



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	Job Title Dead Sea Parkway - JICA		
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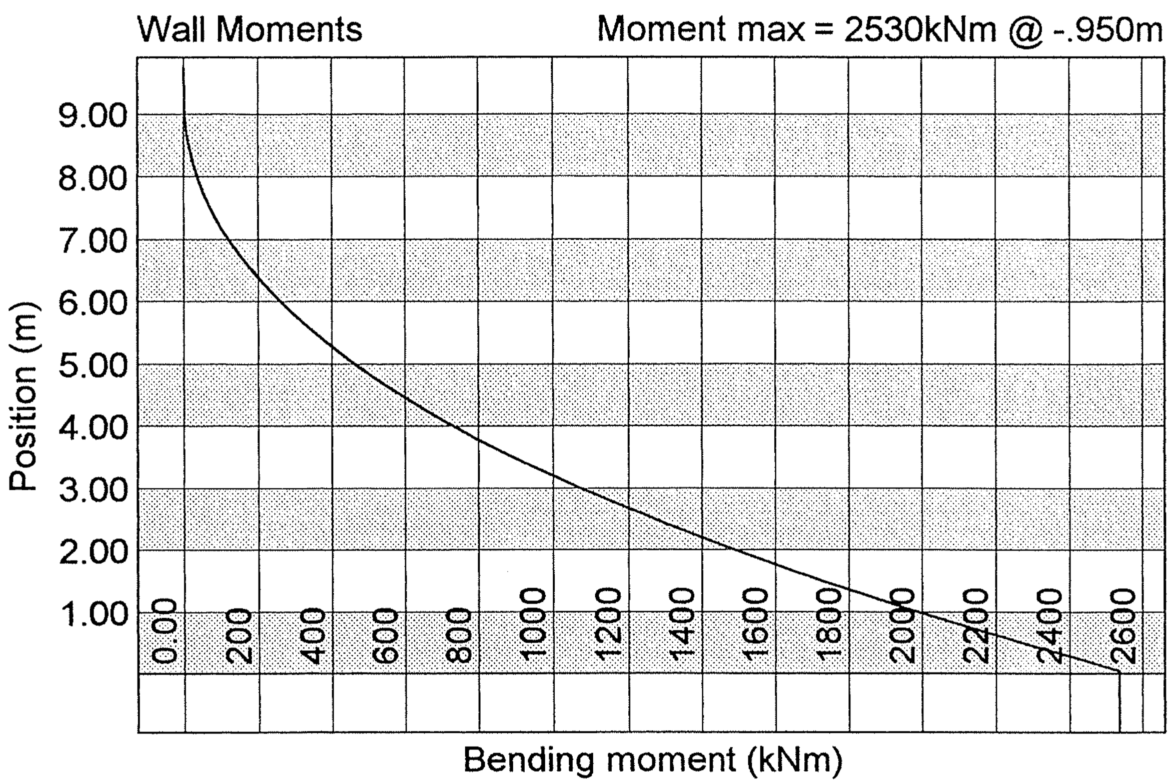
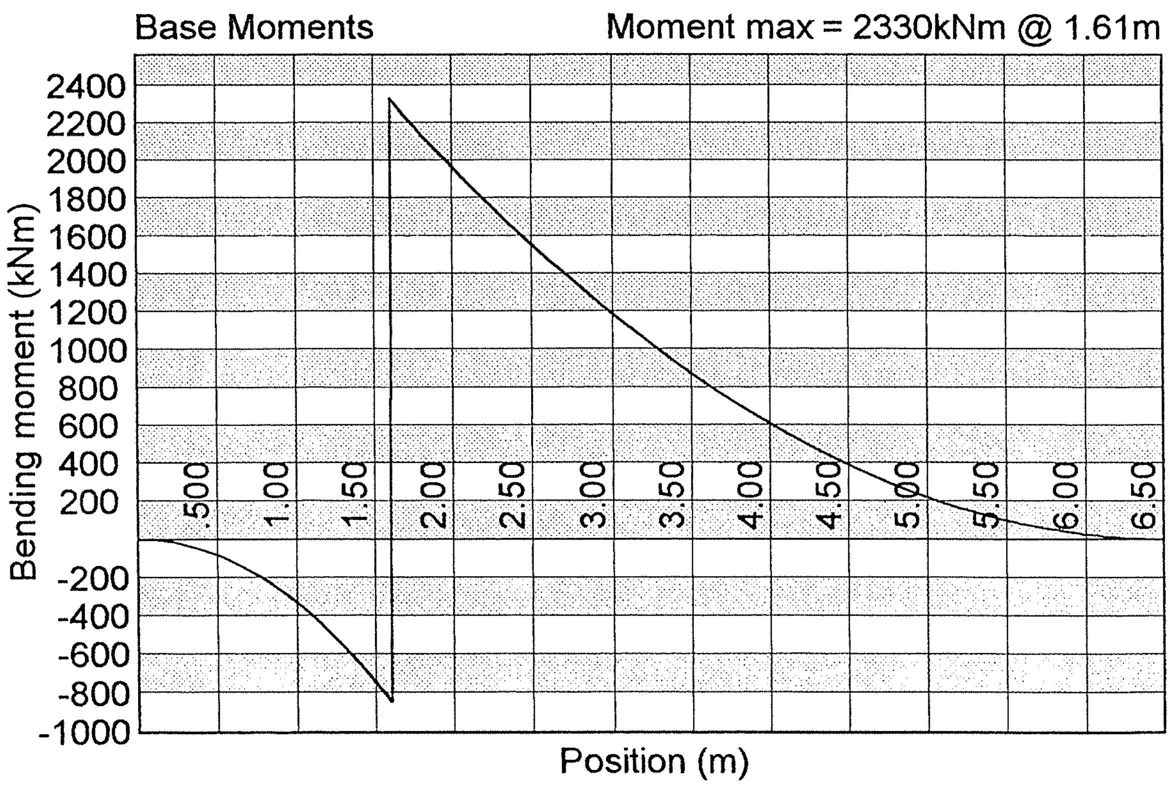
	Calcs by N.O	Checked by	Date FEB,2000

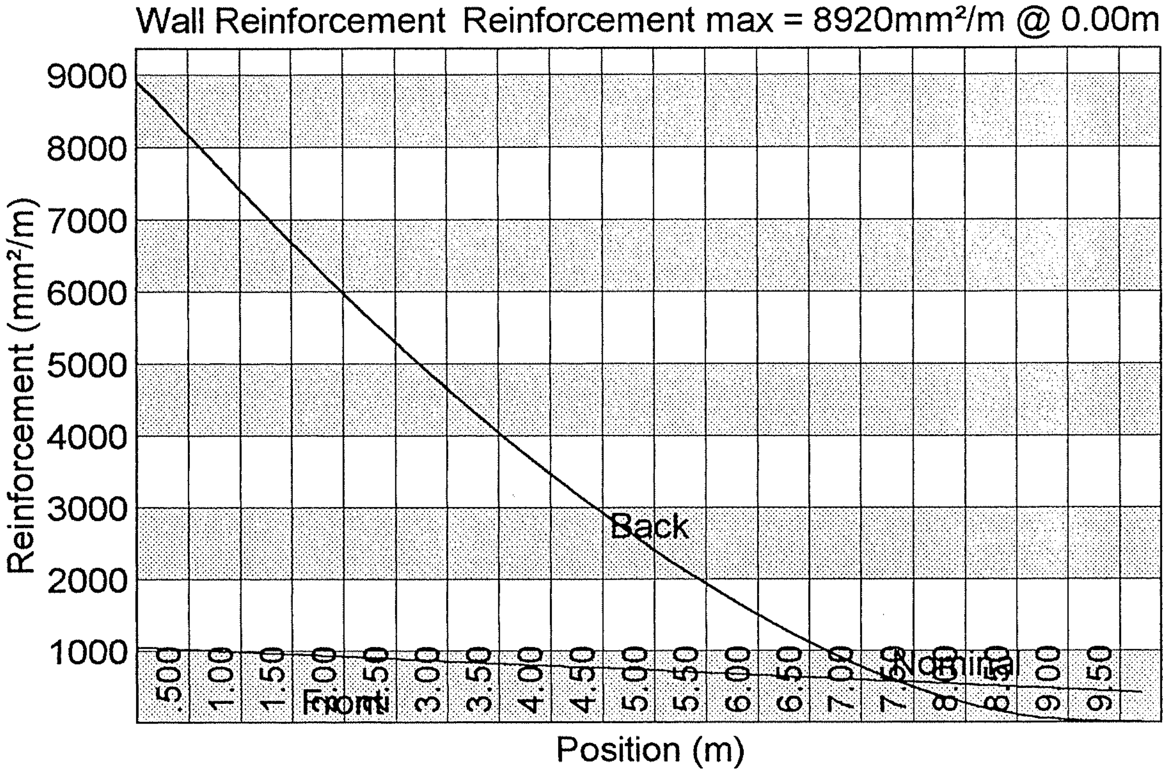
Wall Bending Moments



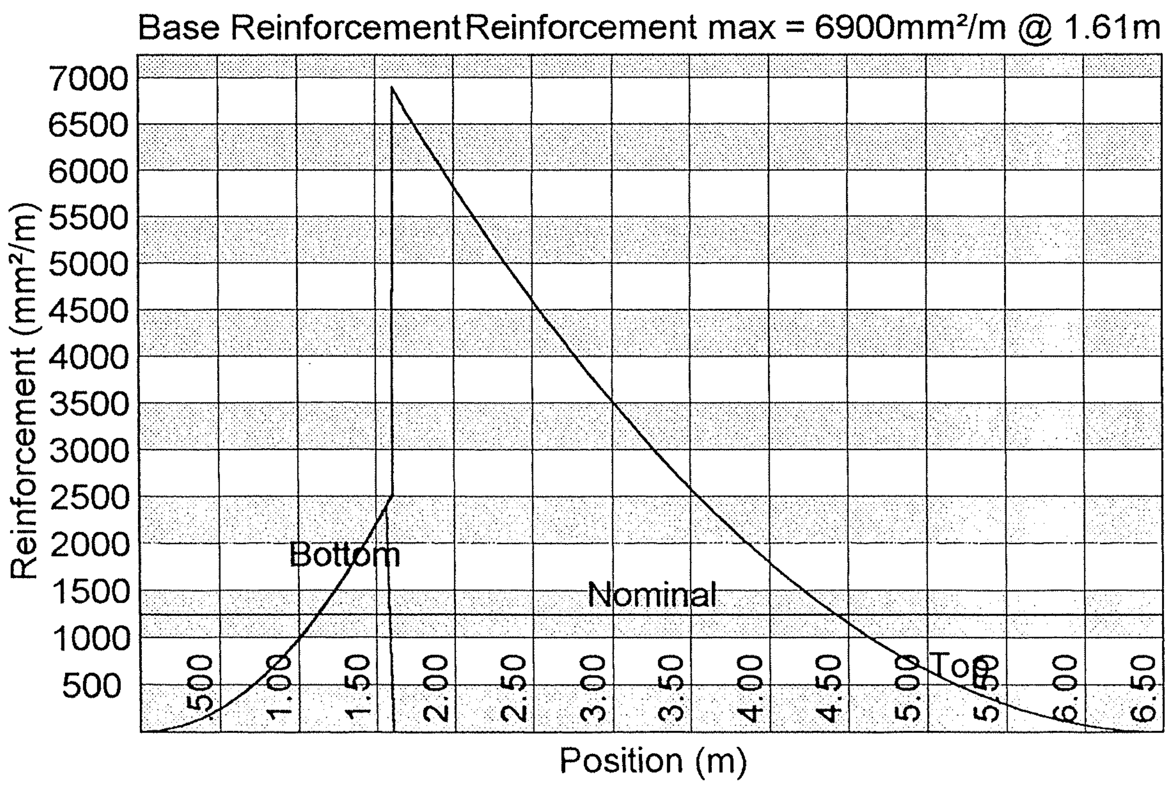
Base Bending Moments



Wall Reinforcement



Base Reinforcement



Retaining Wall Design : Height = 9.0 m

C14

Input Data

Seepage allowed

Theory : Coulomb
Wall type : Cantilever

SEISMIC ANALYSIS SETTINGS:

Seismic Analysis ON/OFF:ON

VALUES OF PRESSURE COEFFICIENTS:

```
          Active Pressure coefficient Ka :0.32
         Passive Pressure coefficient Kp :8.32
 Seismic Active Pressure coefficient Kas :0.48
Seismic Passive Pressure coefficient Kps :1.60
              Base frictional constant μ :0.58
```

FORCES ACTING ON THE WALL:

FORCES (kN) and their LEVER ARMS (m)

Description	F Horizontal left (+)	Lever arm	F Vertical down (+)	Lever arm
Destabilizing forces:				
Total Active pressure Pa	425.954	4.374	237.919	2.047
Siesmic component of Pa	138.421	6.300	79.280	1.940
As a result of surcharge w	67.651	5.250	38.747	2.055
Siesmic wall inertia	63.040	5.700		
Stabilizing forces:				
Passive pressure on base Pp	-11.691	0.300		
Siesmic component of Pp	48.939	0.540		
Weight of the wall + base			331.788	2.875
Weight of soil on the base			824.278	4.561
UDL of 16.2 kN/m2			83.106	4.400

EQUILIBRIUM CALCULATIONS AT SLS

1.Moment Equilibrium

Point of rotation: bottom front corner of base.

For Overturning moment Mo calculate as follows:
  Mo = Sum(hor. forces x l.a.) - Sum(vert. forces x l.a.)
For Stabilizing moment Mr calculate as follows:
  Mr = -Sum(hor. forces x l.a.) + Sum(vert. forces x l.a.)
where l.a. = lever arm of each force

Stabilizing moment Mr : 5082.1 kNm
Destabilizing moment Mo : 2011.0 kNm

Safety factor against overturning = Mr/Mo = 2.53

2.Force Equilibrium

Sum of Vertical forces Pv : 1515.8 kN
Frictional resistance Pfric : 875.2 kN
Passive Pressure on shear key : 25.3 kN
Passive pressure on base : 11.7 kN
=> Horizontal resistance Fr : 912.1 kN
Horizontal sliding force Fh : 556.6 kN

Safety factor against overall sliding = Fr/Fh = 1.64

SOIL PRESSURES UNDER BASE

Maximum pressure :498.8 kPa
Minimum pressure : 0 kPa at 1.0 m from right hand side of base.
Maximum pressure occurs at left hand side of base

WALL MOMENTS (ULS) AND REINFORCEMENT TO BS8110 - 1997

Position from base top (m)	Moment (kNm)	Front Reinforcing (mm^2/m)	Back Reinforcing (mm^2/m)	Nominal (0.13%) (mm^2/m)
0.00	3017.63	0.00	9343.93	1185.60
0.21	2894.21	0.00	9089.80	1170.00
0.42	2755.58	0.00	8779.82	1154.40
0.62	2620.88	0.00	8473.46	1138.80
0.83	2490.09	0.00	8170.76	1123.20
1.04	2363.16	0.00	7871.75	1107.60
1.25	2240.05	0.00	7576.47	1092.00
1.46	2120.73	0.00	7284.97	1076.40
1.66	2005.15	0.00	6997.28	1060.80
1.87	1893.28	0.00	6713.45	1045.20
2.08	1785.08	0.00	6433.53	1029.60
2.29	1680.50	0.00	6157.58	1014.00
2.50	1579.51	0.00	5885.64	998.40
2.70	1482.08	0.00	5617.79	982.80
2.91	1388.15	0.00	5354.08	967.20
3.12	1297.70	0.00	5094.59	951.60
3.33	1210.68	0.00	4839.39	936.00
3.54	1127.06	0.00	4588.55	920.40
3.74	1046.79	0.00	4342.17	904.80
3.95	969.84	0.00	4100.33	889.20
4.16	896.17	0.00	3863.14	873.60
4.37	825.73	0.00	3630.70	858.00
4.58	758.49	0.00	3403.13	842.40
4.78	694.42	0.00	3180.55	826.80
4.99	633.47	0.00	2963.11	811.20
5.20	575.59	0.00	2750.94	795.60
5.41	520.77	0.00	2544.21	780.00
5.62	468.94	0.00	2343.10	764.40
5.82	420.09	0.00	2147.80	748.80
6.03	374.16	0.00	1958.52	733.20
6.24	331.12	0.00	1775.49	717.60
6.45	290.92	0.00	1598.97	702.00
6.66	253.54	0.00	1429.23	686.40
6.86	218.93	0.00	1266.59	670.80
7.07	187.05	0.00	1111.40	655.20
7.28	157.86	0.00	964.02	639.60
7.49	131.32	0.00	824.90	624.00
7.70	107.41	0.00	694.50	608.40
7.90	86.06	0.00	573.36	592.80
8.11	67.26	0.00	462.07	577.20

8.53	37.10	0.00	271.88	546.00
8.74	25.67	0.00	194.62	530.40
8.94	16.62	0.00	130.53	514.80
9.15	9.92	0.00	80.77	499.20
9.36	5.52	0.00	46.65	483.60
9.57	3.28	0.00	28.85	468.00
9.78	1.86	0.00	17.06	452.40
9.98	0.84	0.00	8.08	436.80
10.19	0.23	0.00	2.27	421.20
10.40	0.01	0.00	0.09	405.60

BASE MOMENTS (ULS) AND REINFORCEMENT TO BS8110 - 1997

Position from left (m)	Moment (kNm)	Top Reinforcing (mm^2/m)	Bot Reinforcing (mm^2/m)	Nominal (0.13%) (mm^2/m)
0.14	-0.00	0.00	0.00	1300.00
0.28	-6.79	0.00	19.06	1300.00
0.43	-27.16	0.00	76.24	1300.00
0.57	-61.12	0.00	171.54	1300.00
0.71	-108.66	0.00	304.95	1300.00
0.85	-169.78	0.00	476.49	1300.00
0.99	-244.48	0.00	686.14	1300.00
1.14	-332.76	0.00	933.92	1300.00
1.28	-434.63	0.00	1219.81	1300.00
1.42	-550.07	0.00	1543.82	1300.00
1.56	-679.10	0.00	1905.95	1300.00
1.70	-821.72	0.00	2306.20	1300.00
1.85	-977.91	0.00	2744.57	1300.00
1.99	-1147.69	0.00	3221.06	1300.00
2.13	-1152.66	0.00	3235.03	1300.00
2.27	2618.36	7348.61	0.00	1300.00
2.41	2482.52	6967.36	0.00	1300.00
2.56	2346.52	6585.66	0.00	1300.00
2.70	2214.35	6214.72	0.00	1300.00
2.84	2086.01	5854.52	0.00	1300.00
2.98	1961.50	5505.08	0.00	1300.00
3.12	1840.82	5166.39	0.00	1300.00
3.27	1723.97	4838.45	0.00	1300.00
3.41	1610.96	4521.26	0.00	1300.00
3.55	1501.77	4214.82	0.00	1300.00
3.69	1396.42	3919.14	0.00	1300.00
3.83	1294.89	3634.21	0.00	1300.00
3.98	1197.20	3360.03	0.00	1300.00
4.12	1103.34	3096.61	0.00	1300.00
4.26	1013.31	2843.93	0.00	1300.00
4.40	927.11	2602.01	0.00	1300.00
4.54	844.75	2370.84	0.00	1300.00
4.69	766.21	2150.42	0.00	1300.00
4.83	691.50	1940.75	0.00	1300.00
4.97	620.63	1741.84	0.00	1300.00
5.11	553.59	1553.68	0.00	1300.00
5.25	490.37	1376.27	0.00	1300.00
5.40	430.99	1209.61	0.00	1300.00
5.54	375.44	1053.71	0.00	1300.00
5.68	323.72	908.55	0.00	1300.00
5.82	275.84	774.15	0.00	1300.00
5.96	231.78	650.50	0.00	1300.00
6.11	191.55	537.61	0.00	1300.00
6.25	155.16	435.46	0.00	1300.00
6.39	122.59	344.07	0.00	1300.00
6.53	93.86	263.43	0.00	1300.00
6.67	68.96	193.54	0.00	1300.00
6.82	47.89	134.40	0.00	1300.00
6.96	30.65	86.02	0.00	1300.00
7.10	17.24	48.38	0.00	1300.00

SHEAR CHECK AT WALL-BASE JUNCTION TO BS8110 - 1997

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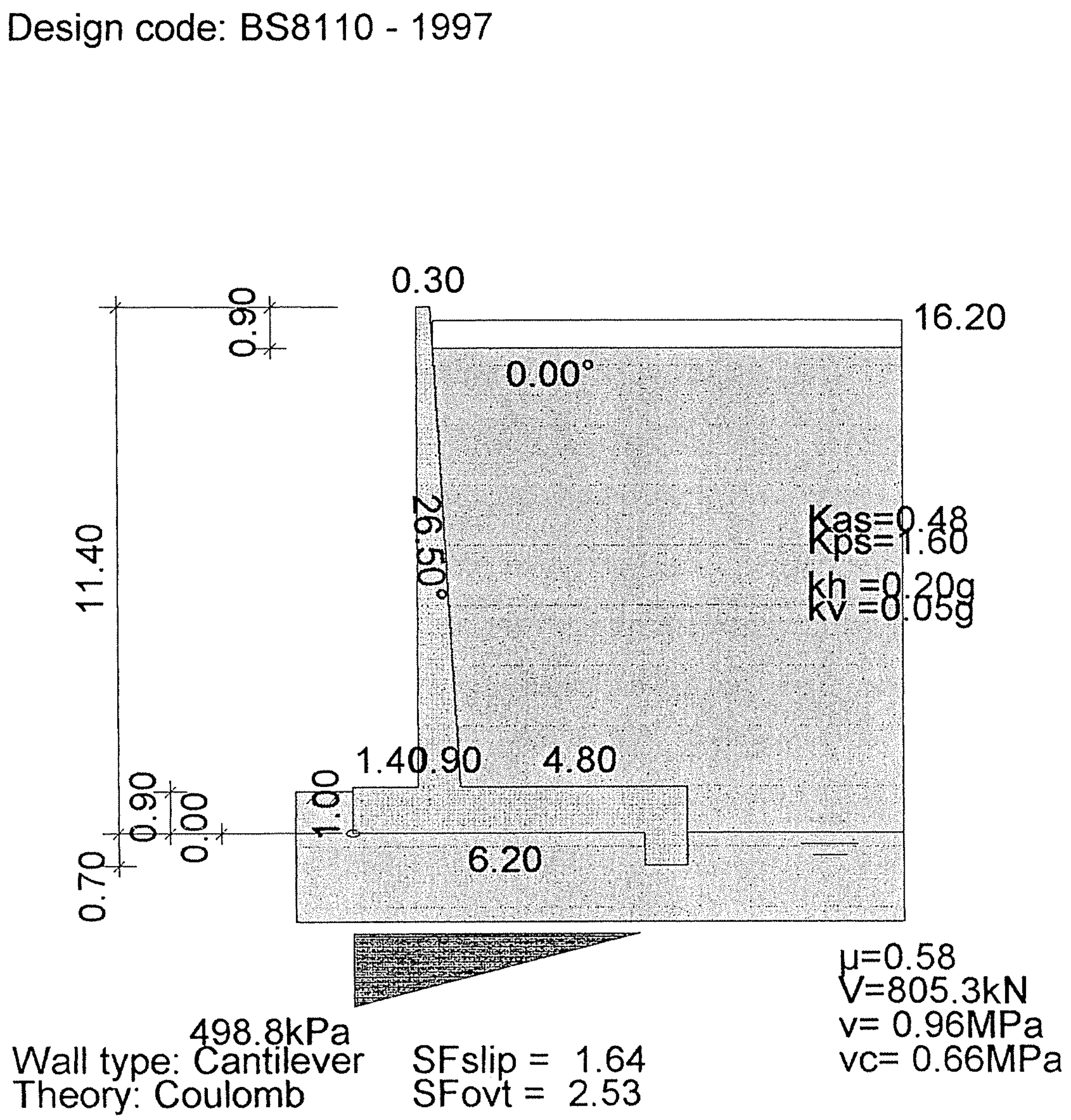
	Calcs by N.O	Checked by	Date FEB,2000

```
 Shear force at bottom of wall V = 805.3 kN
Shear stress at bottom of wall v = 0.96 MPa INADEQUATE
       Allowable shear stress vc = 0.66 MPa (based on Wall tensile reinf.)
```

Sketch of Wall



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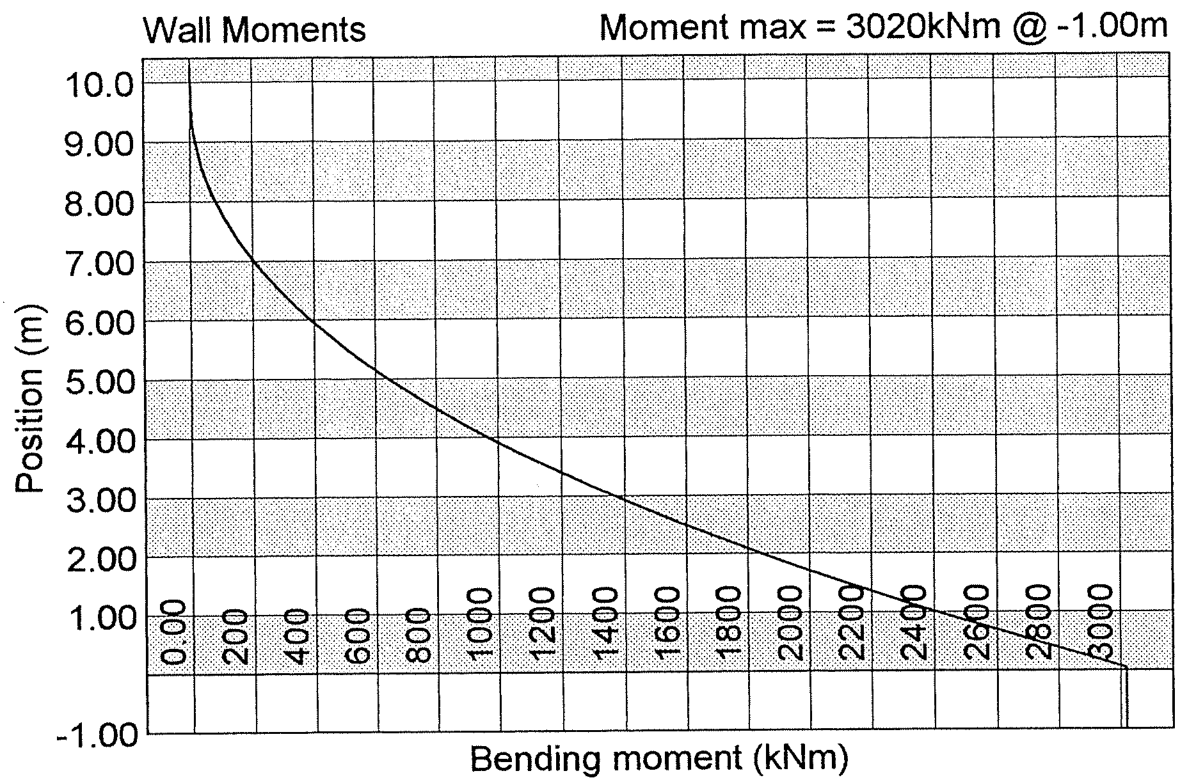
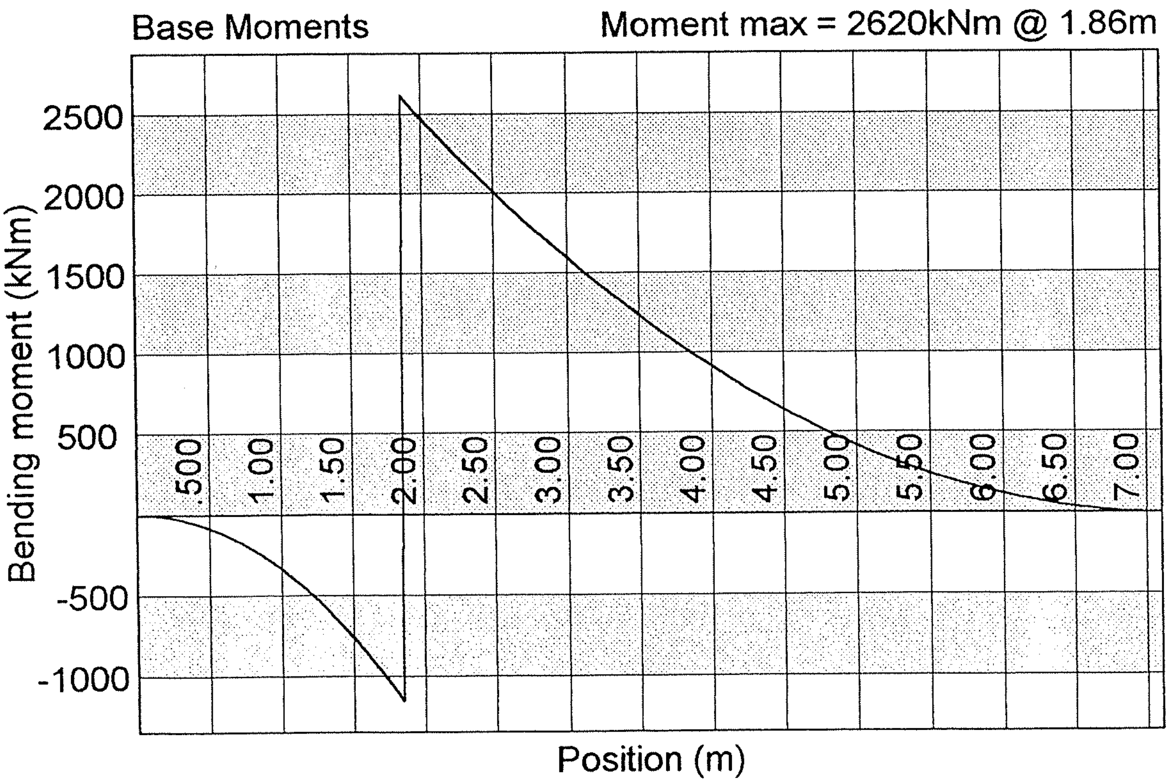
	Job Title Dead Sea Parkway - JICA		
	Client Ministry of Tourism		
	Calcs by N.O	Checked by	Date FEB,2000

Wall Bending Moments



Base Bending Moments



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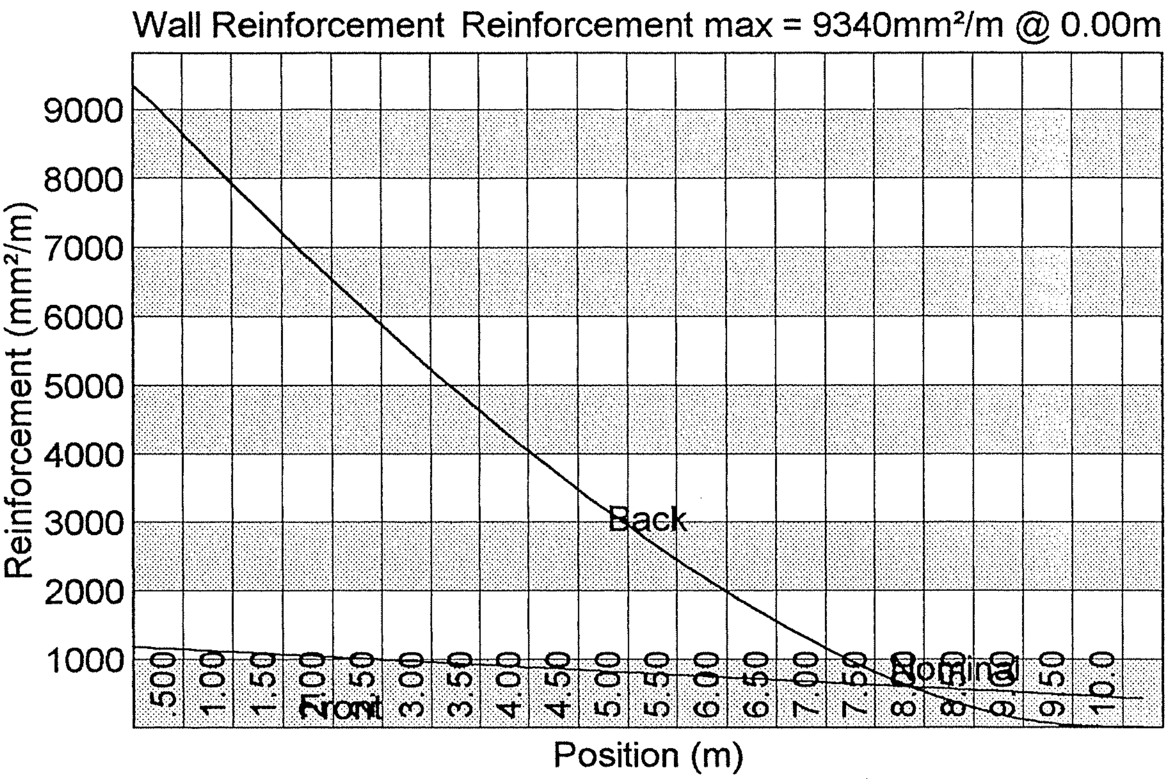
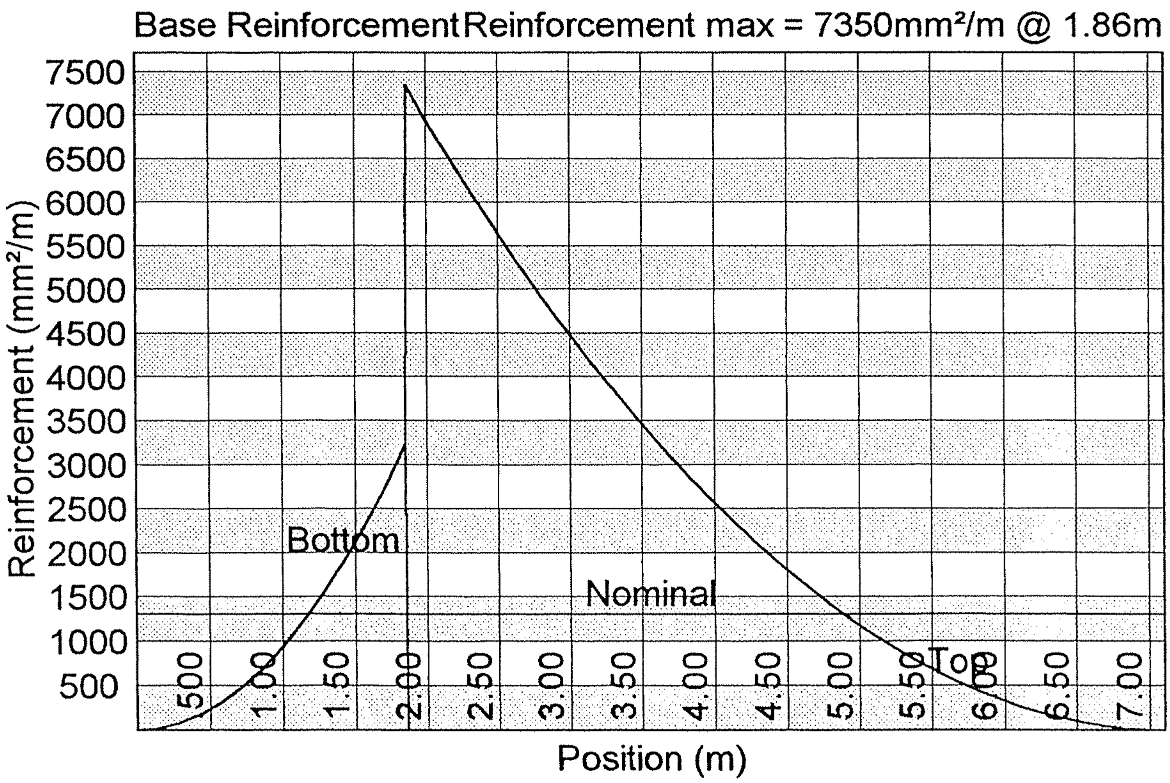
	Client Ministry of Tourism		
	Calcs by N.O	Checked by	Date FEB,2000

Wall Reinforcement



Base Reinforcement



C14

Retaining Wall Design : Height = 10.0 m

Input Data

Seepage allowed

Theory : Coulomb
Wall type : Cantilever

SEISMIC ANALYSIS SETTINGS:

Seismic Analysis ON/OFF:ON

VALUES OF PRESSURE COEFFICIENTS:

Active Pressure coefficient Ka :0.33
Passive Pressure coefficient Kp :4.98
Seismic Active Pressure coefficient Kas :0.47
Seismic Passive Pressure coefficient Kps :2.05
Base frictional constant μ :0.58

FORCES ACTING ON THE WALL:

FORCES (kN) and their LEVER ARMS (m)

Description	F Horizontal left (+)	Lever arm	F Vertical down (+)	Lever arm
Destabilizing forces:				
Total Active pressure Pa	535.262	4.718	177.250	2.311
Siesmic component of Pa	158.325	6.900	53.209	2.180
As a result of surcharge w	78.423	5.750	26.356	2.316
Siesmic wall inertia	79.330	6.200		
Stabilizing forces:				
Passive pressure on base Pp	-14.965	0.300		
Siesmic component of Pp	21.314	0.540		
Weight of the wall + base			417.525	3.285
Weight of soil on the base			1007.794	5.136
UDL of 16.2 kN/m2			93.879	4.950

EQUILIBRIUM CALCULATIONS AT SLS

1.Moment Equilibrium

Point of rotation: bottom front corner of base.

For Overturning moment Mo calculate as follows:
Mo = Sum(hor. forces x l.a.) - Sum(vert. forces x l.a.)
For Stabilizing moment Mr calculate as follows:
Mr = -Sum(hor. forces x l.a.) + Sum(vert. forces x l.a.)
where l.a. = lever arm of each force

Stabilizing moment Mr : 7016.9 kNm
Destabilizing moment Mo : 2997.6 kNm

Safety factor against overturning = Mr/Mo = 2.34

2.Force Equilibrium

Sum of Vertical forces Pv : 1722.8 kN
Frictional resistance Pfric : 994.7 kN
Passive Pressure on shear key : 32.3 kN
Passive pressure on base : 15.0 kN
=> Horizontal resistance Fr : 1042.0 kN
Horizontal sliding force Fh : 693.0 kN

Safety factor against overall sliding = Fr/Fh = 1.50

SOIL PRESSURES UNDER BASE

Maximum pressure :492.3 kPa
Minimum pressure : 0 kPa at 1.0 m from right hand side of base.
Maximum pressure occurs at left hand side of base

***** MAXIMUM PRESSURE EXCEEDS SPECIFIED ALLOWABLE PRESSURE BY 92.3 kPa *****

WALL MOMENTS (ULS) AND REINFORCEMENT TO BS8110 - 1997

Position from base top (m)	Moment (kNm)	Front Reinforcing (mm^2/m)	Back Reinforcing (mm^2/m)	Nominal (0.13%) (mm^2/m)
0.00	3957.25	0.00	10943.32	1318.20
0.22	3794.87	0.00	10650.58	1300.00
0.45	3612.08	0.00	10290.82	1281.80
0.67	3434.58	0.00	9935.32	1263.60
0.90	3262.31	0.00	9584.12	1245.40
1.12	3095.22	0.00	9237.25	1227.20
1.34	2933.25	0.00	8894.75	1209.00
1.57	2776.35	0.00	8556.66	1190.80
1.79	2624.46	0.00	8223.02	1172.60
2.02	2477.52	0.00	7893.87	1154.40
2.24	2335.47	0.00	7569.27	1136.20
2.46	2198.27	0.00	7249.27	1118.00
2.69	2065.84	0.00	6933.91	1099.80
2.91	1938.15	0.00	6623.27	1081.60
3.14	1815.12	0.00	6317.40	1063.40
3.36	1696.70	0.00	6016.38	1045.20
3.58	1582.84	0.00	5720.27	1027.00
3.81	1473.47	0.00	5429.16	1008.80
4.03	1368.55	0.00	5143.14	990.60
4.26	1268.02	0.00	4862.29	972.40
4.48	1171.81	0.00	4586.72	954.20
4.70	1079.88	0.00	4316.53	936.00
4.93	992.16	0.00	4051.85	917.80
5.15	908.60	0.00	3792.81	899.60
5.38	829.15	0.00	3539.54	881.40
5.60	753.74	0.00	3292.20	863.20
5.82	682.32	0.00	3050.96	845.00
6.05	614.83	0.00	2816.01	826.80
6.27	551.21	0.00	2587.54	808.60
6.50	491.42	0.00	2365.79	790.40
6.72	435.39	0.00	2151.00	772.20
6.94	383.06	0.00	1943.45	754.00
7.17	334.39	0.00	1743.43	735.80
7.39	289.31	0.00	1551.30	717.60
7.62	247.76	0.00	1367.43	699.40
7.84	209.69	0.00	1192.25	681.20
8.06	175.05	0.00	1026.23	663.00
8.29	143.77	0.00	869.92	644.80

8.74	91.08	0.00	588.94	608.40
8.96	69.56	0.00	465.77	590.20
9.18	51.18	0.00	355.32	572.00
9.41	35.88	0.00	258.64	553.80
9.63	23.61	0.00	176.95	535.60
9.86	14.31	0.00	111.68	517.40
10.08	7.92	0.00	64.48	499.20
10.30	4.39	0.00	37.32	481.00
10.53	2.49	0.00	22.18	462.80
10.75	1.13	0.00	10.54	444.60
10.98	0.30	0.00	2.97	426.40
11.20	0.01	0.00	0.12	408.20

BASE MOMENTS (ULS) AND REINFORCEMENT TO BS8110 - 1997

Position from left (m)	Moment (kNm)	Top Reinforcing (mm^2/m)	Bot Reinforcing (mm^2/m)	Nominal (0.13%) (mm^2/m)
0.16	-0.00	0.00	0.00	1560.00
0.32	-8.38	0.00	19.39	1560.00
0.48	-33.51	0.00	77.55	1560.00
0.64	-75.40	0.00	174.49	1560.00
0.80	-134.05	0.00	310.21	1560.00
0.96	-209.45	0.00	484.70	1560.00
1.12	-301.61	0.00	697.97	1560.00
1.28	-410.52	0.00	950.02	1560.00
1.44	-536.19	0.00	1240.84	1560.00
1.60	-678.61	0.00	1570.44	1560.00
1.76	-837.79	0.00	1938.81	1560.00
1.92	-1013.73	0.00	2345.96	1560.00
2.08	-1206.42	0.00	2791.89	1560.00
2.24	-1415.87	0.00	3276.59	1560.00
2.40	-1443.23	3339.91	0.00	1560.00
2.56	3629.54	8399.44	0.00	1560.00
2.72	3459.33	8005.55	0.00	1560.00
2.88	3269.82	7566.97	0.00	1560.00
3.04	3085.64	7140.75	0.00	1560.00
3.20	2906.80	6726.89	0.00	1560.00
3.36	2733.30	6325.37	0.00	1560.00
3.52	2565.14	5936.21	0.00	1560.00
3.68	2402.32	5559.41	0.00	1560.00
3.84	2244.83	5194.96	0.00	1560.00
4.00	2092.68	4842.86	0.00	1560.00
4.16	1945.88	4503.12	0.00	1560.00
4.32	1804.41	4175.73	0.00	1560.00
4.48	1668.27	3860.70	0.00	1560.00
4.64	1537.48	3558.02	0.00	1560.00
4.80	1412.03	3267.70	0.00	1560.00
4.96	1291.91	2989.73	0.00	1560.00
5.12	1177.13	2724.11	0.00	1560.00
5.28	1067.70	2470.85	0.00	1560.00
5.44	963.60	2229.94	0.00	1560.00
5.60	864.83	2001.39	0.00	1560.00
5.76	771.41	1785.19	0.00	1560.00
5.92	683.33	1581.34	0.00	1560.00
6.08	600.58	1389.85	0.00	1560.00
6.24	523.17	1210.72	0.00	1560.00
6.40	451.10	1043.93	0.00	1560.00
6.56	384.37	889.51	0.00	1560.00
6.72	322.98	747.43	0.00	1560.00
6.88	266.92	617.71	0.00	1560.00
7.04	216.21	500.35	0.00	1560.00
7.20	170.83	395.34	0.00	1560.00
7.36	130.79	302.68	0.00	1560.00
7.52	96.09	222.38	0.00	1560.00
7.68	66.73	154.43	0.00	1560.00
7.84	42.71	98.83	0.00	1560.00
8.00	24.02	55.59	0.00	1560.00

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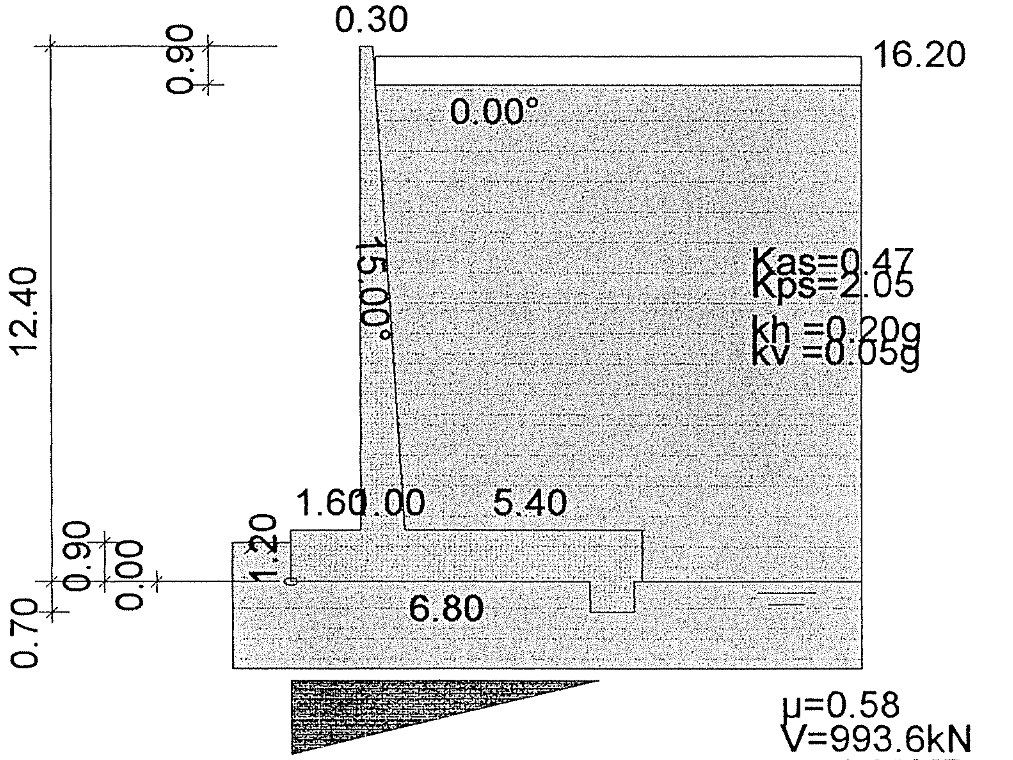
	Job Title Dead Sea Parkway - JICA		
	Client Ministry of Tourism		
	Calcs by N.O	Checked by	Date FEB,2000

```
SHEAR CHECK AT WALL-BASE JUNCTION TO BS8110 - 1997

 Shear force at bottom of wall V = 993.6 kN
Shear stress at bottom of wall v = 1.06 MPa INADEQUATE
       Allowable shear stress vc = 0.67 MPa (based on Wall tensile reinf.)
```

Sketch of Wall

Design code: BS8110 - 1997



Wall type: Cantilever   SFslip = 1.50
Theory: Coulomb   SFovt = 2.34

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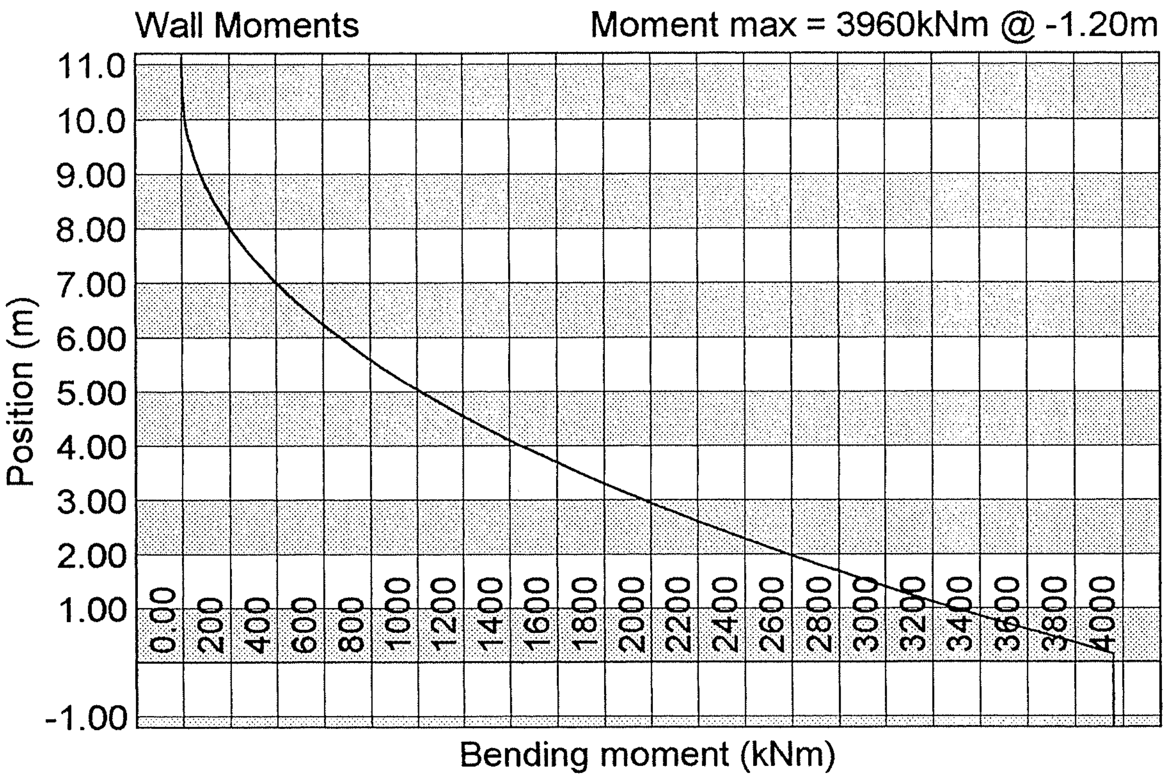
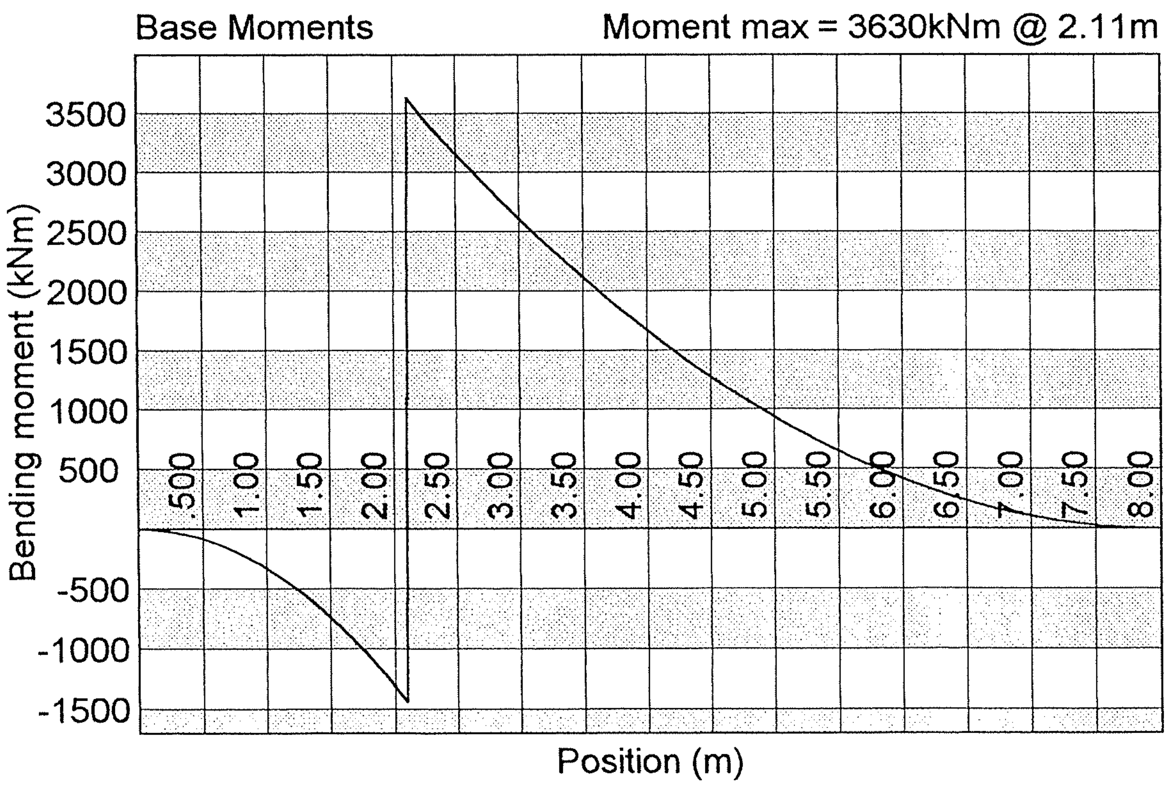
	Job Title Dead Sea Parkway - JICA		
	Client Ministry of Tourism		
	Calcs by N.O	Checked by	Date FEB,2000

Wall Bending Moments



Base Bending Moments



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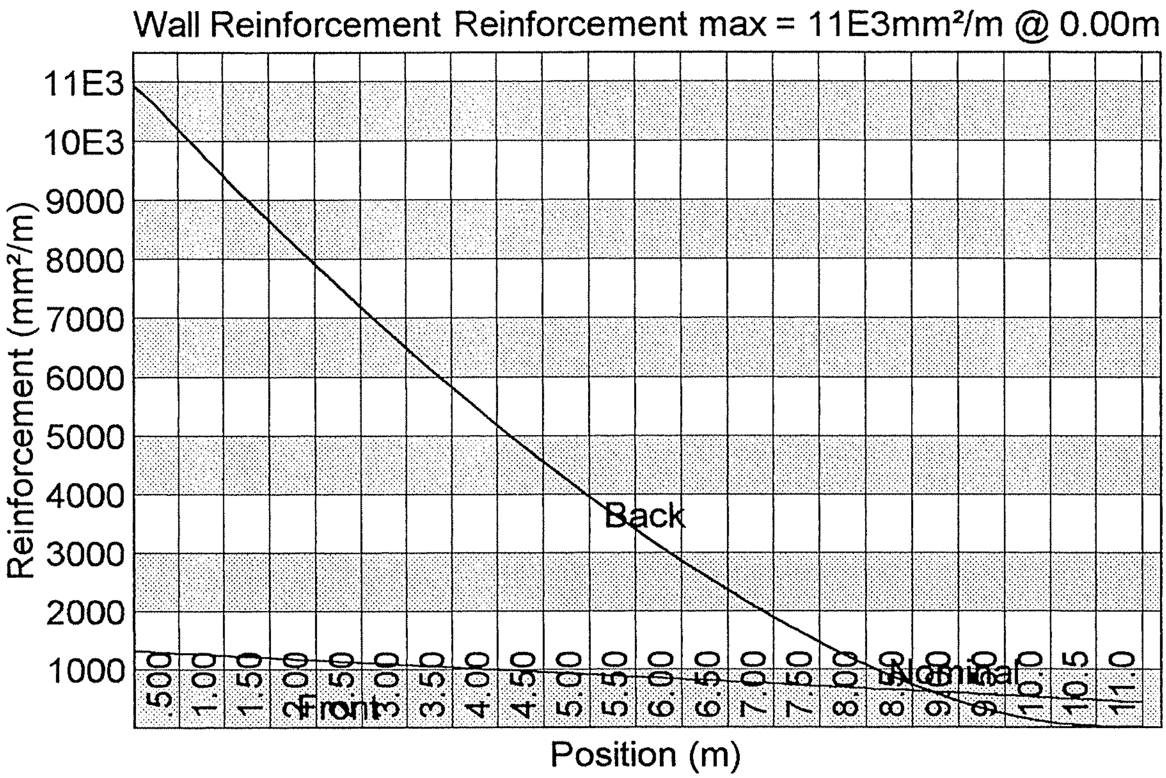
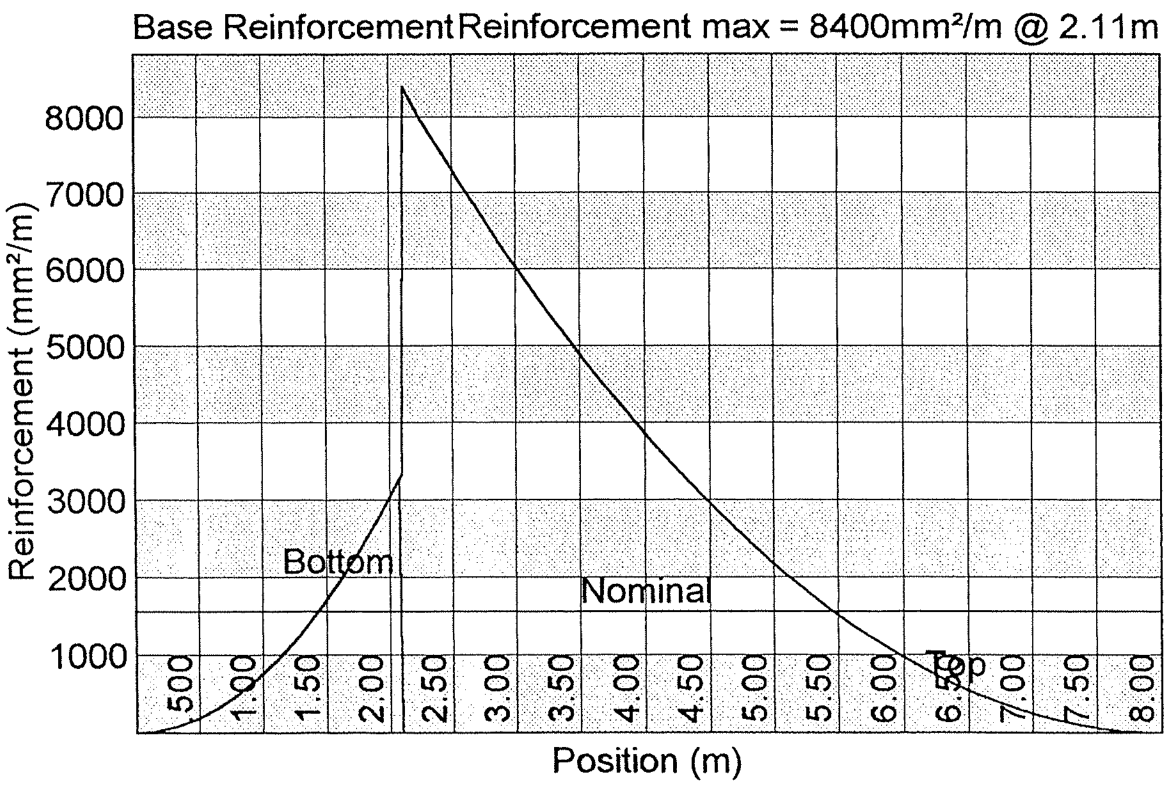
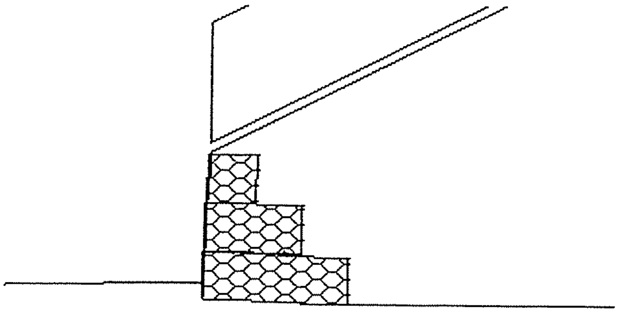
	Job Title Dead Sea Parkway - JICA			
	Client Ministry of Tourism			
	Calcs by N.O	Checked by	Date FEB,2000	

Wall Reinforcement



Base Reinforcement





Gabions

Gabion Walls calculation - GAWAC R 2.0
Dr. Sawan Himmo

File: 4M   Project : JICA PROJECT 4METER WALL

****
DATA INPUT   PAGE 2
****

DATA ABOUT THE UPHILL WATER SURFACE
Initial height [m]............................:
Inclination of the first stretch [deg]........:
Length of the stretch [m].....................:
Inclination of the second stretch [deg].......:
Length of the stretch [m].....................:

GEOMETRICAL DATA ABOUT FOUNDATION SOIL
Elev. of placing from the base [m]............: 0.50
Soil inclination ref.to base [deg]............: 0.00

DATAI ABOUT FOUNDATION SOIL
Unit weight of soil [t/m3]....................: 1.80
Friction angle [deg]..........................: 30.00
Cohesion [t/m2]...............................: 0.00
Allowable stress on foundation [t/m2].........: 50.00
Water level [m]...............................:

DATA ABOUT ADDITIONAL FOUNDATION SOILS

Layer	Depth [m]	Unit weight [t/m3]	Cohesion [t/m2]	Frict.angle [deg]

DATA ABOUT DISTRIBUTED LOADS

Distributed loads on backfill
First stretch [t/m2].....: 1.00   Second stretch [t/m2]....:
Distr. loads on the wall
Surcharge load [t/m2]....:

Point loads on the backfill
1.Load [t/m].............:   Dist. from wall top [m]..:
2.Load [t/m].............:   Dist. from wall top [m]..:
3.Load [t/m].............:   Dist. from wall top [m]..:
Point loads on the wall
Surcharge load [t/m2]....:   Dist. from wall top [m]..:

DATA ABOUT SEISMIC ACTIONS
Horizontal coefficient...: 0.15   Vertical coefficient.....: 0.05

OVERALL STABILITY

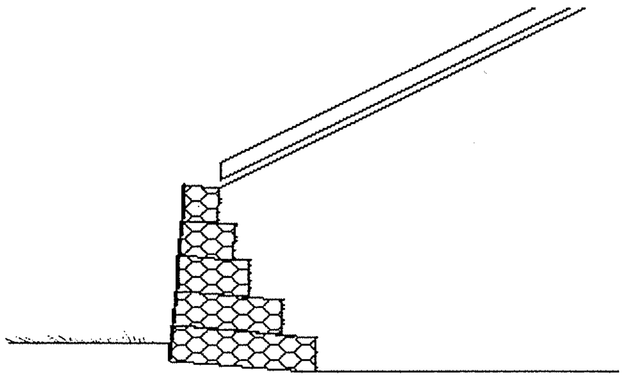
Initial distance at pivot leftside [m]........:
Initial distance at pivot rightside [m].......:
Initial depth referred to base [m] ....:
Max. depth allowed in the calculation [m].....:
Center of the arc referred to X axis [m]......:
Center of the arc referred to Y axis [m]......:
Radius of the arc [m].........................:
Number of search surfaces ....:

Safety coefficient............................:

INTERNAL STABILITY

Layer	H [m]	N [t/m]	T [t/m]	M [tm/m]	tmax [t/m2]	tad [t/m2]	smax [t/m2]

Allowable normal pressure [t/m2].............:



Gabion Walls calculation - GAWAC R 2.0
Dr. Sawan Himmo

File: 5M   Project : 5M JICA PROJECT

****
DATA INPUT   PAGE 1
****

WALL DATA

		Layer	Length [m]	Height [m]	Init. [m]
Wall batter [deg]............:	5.00	1	4.00	1.00	
Unit weight of rockfill [t/m3]:	2.00	2	3.00	1.00	0.00
Porosity of gabions...........:	0.30	3	2.00	1.00	0.00
Geotextile in backfill........:	No	4	1.50	1.00	0.00
Friction reduction [%]........:		5	1.00	1.00	0.00
Geotextile under the base.....:	No				
Friction reduction [%]........:					

GEOMETRICAL PARAMETERS OF BACKFILL
Inclination of the first stretch [deg]........: 26.50
Length of the stretch [m].....................: 15.00
Inclination of the second stretch [deg].......: 0.00

BACKFILL SOIL DATA
Unit weight of soil [t/m3]....................: 1.80
Friction angle [deg]..........................: 33.00
Cohesion [t/m2]...............................: 0.00

BACKFILL SOIL ADDITIONAL DATA

Layer	Initial Height [m]	Inclin. angle [deg]	Unit weight [t/m3]	Cohesion [t/m2]	Frict. angle [deg]

EXTERNAL STABILITY

Active thrust [t/m]............................:	61.08
Point of application ref. to X axis [m].......:	2.62
Point of application ref. to Y axis [m].......:	2.45
Direction of thrust ref. to X axis [deg]......:	58.96
Passive thrust [t/m]...........................:	0.68
Point of application ref. to X axis [m].......:	0.01
Point of application ref. to Y axis [m].......:	0.17
Direction of thrust ref. to X axis [deg]......:	0.00

SLIDING

Normal force on the base [t/m]................:	71.60
Point of application ref. to X axis [m].......:	1.14
Point of application ref. to Y axis [m].......:	-0.10
Shear force on the base [t/m].................:	27.17
Resisting force on the base [t/m].............:	42.08
Safety coefficient............................:	1.511

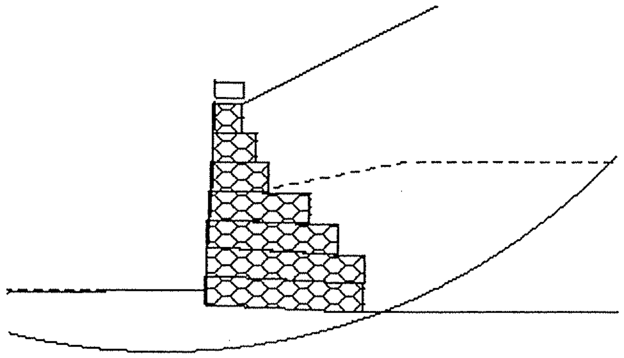
OVERTURNING

Overturning moment [tm/m].....................:	81.88
Restoring moment [tm/m].......................:	163.37
Safety coefficient............................:	1.995

STRESSES ACTING ON FOUNDATION

Stress on outer foundation border [t/m2]......:	42.08
Stress on inner foundation border [t/m2]......:	0.00
Max. allowable stress on the foundation [t/m2]:	50.00

Notice: the base is partialized!



DATA ABOUT THE UPHILL WATER SURFACE
Initial height [m]..............................: 4.00
Inclination of the first stretch [deg]........: 10.00
Length of the stretch [m].......................: 5.00
Inclination of the second stretch [deg].......: 0.00
Length of the stretch [m].......................: 10.00

GEOMETRICAL DATA ABOUT FOUNDATION SOIL
Elev. of placing from the base [m].............: 0.50
Soil inclination ref.to base [deg].............: 1.00

DATAI ABOUT FOUNDATION SOIL
Unit weight of soil [t/m3].....................: 2.00
Friction angle [deg]...........................: 33.00
Cohesion [t/m2].................................: 0.00
Allowable stress on foundation [t/m2].........: 50.00
Water level [m].................................: 0.50

DATA ABOUT ADDITIONAL FOUNDATION SOILS

Layer	Depth [m]	Unit weight [t/m3]	Cohesion [t/m2]	Frict.angle [deg]

DATA ABOUT DISTRIBUTED LOADS

Distributed loads on backfill
First stretch [t/m2].....:   Second stretch [t/m2]....: 1.00
Distr. loads on the wall
Surcharge load [t/m2]....: 0.50

Point loads on the backfill
1.Load [t/m].............:   Dist. from wall top [m]..:
2.Load [t/m].............:   Dist. from wall top [m]..:
3.Load [t/m].............:   Dist. from wall top [m]..:
Point loads on the wall
Surcharge load [t/m2]....:   Dist. from wall top [m]..:

DATA ABOUT SEISMIC ACTIONS
Horizontal coefficient...: 0.15   Vertical coefficient.....: 0.05

EXTERNAL STABILITY

```
Active thrust [t/m]............................:  134.78
Point of application ref. to X axis [m].......:    3.30
Point of application ref. to Y axis [m].......:    3.54
Direction of thrust ref. to X axis [deg]......:   62.88

Passive thrust [t/m]...........................:    0.51
Point of application ref. to X axis [m].......:    0.01
Point of application ref. to Y axis [m].......:    0.17
Direction of thrust ref. to X axis [deg]......:   -1.00
```

SLIDING

```
Normal force on the base [t/m]................:  152.81
Point of application ref. to X axis [m].......:    1.48
Point of application ref. to Y axis [m].......:   -0.07
Shear force on the base [t/m].................:   57.86
Resisting force on the base [t/m].............:   99.78

Safety coefficient............................:   1.710
```

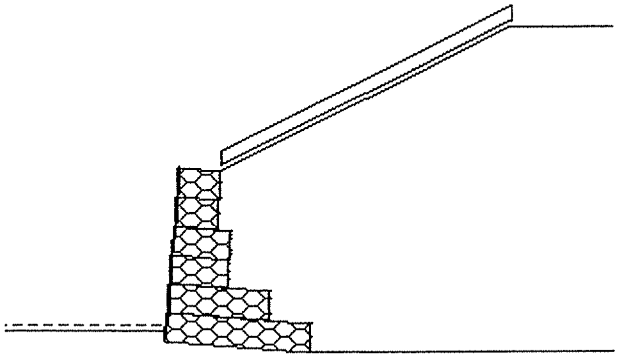
OVERTURNING

```
Overturning moment [tm/m].....................:  229.85
Restoring moment [tm/m].......................:  455.78

Safety coefficient............................:   1.983
```

STRESSES ACTING ON FOUNDATION

```
Stress on outer foundation border [t/m2]......:   68.95
Stress on inner foundation border [t/m2]......:    0.00
Max. allowable stress on the foundation [t/m2]:   50.00
Notice: the base is partialized!
```



Gabion Walls calculation - GAWAC R 2.0

Dr. Sawan Himmo

File: 6M   Project : 6M

****

DATA INPUT   PAGE 1

****

WALL DATA

		Layer	Length [m]	Height [m]	Init. [m]
Wall batter [deg]............:	5.00				
Unit weight of rockfill [t/m3]:	2.00				
Porosity of gabions...........:	0.30				
Geotextile in backfill........:	No	1	5.00	1.00	
Friction reduction [%]........:		2	3.50	1.00	0.00
Geotextile under the base.....:	No	3	2.00	1.00	0.00
Friction reduction [%]........:		4	2.00	1.00	0.00
		5	1.50	1.00	0.00
		6	1.50	1.00	0.00

GEOMETRICAL PARAMETERS OF BACKFILL

Inclination of the first stretch [deg]........: 26.50

Length of the stretch [m]....................: 10.00

Inclination of the second stretch [deg].......: 0.00

BACKFILL SOIL DATA

Unit weight of soil [t/m3]...................: 1.80

Gabion Walls calculation - GAWAC R 2.0
Dr. Sawan Himmo

File: 6M   Project : 6M

****
DATA INPUT   PAGE 2
****

DATA ABOUT THE UPHILL WATER SURFACE
Initial height [m].............................:
Inclination of the first stretch [deg]........:
Length of the stretch [m].....................:
Inclination of the second stretch [deg].......:
Length of the stretch [m].....................:

GEOMETRICAL DATA ABOUT FOUNDATION SOIL
Elev. of placing from the base [m]............: 0.30
Soil inclination ref.to base [deg]............: 0.00

DATAI ABOUT FOUNDATION SOIL
Unit weight of soil [t/m3]....................: 1.80
Friction angle [deg]..........................: 30.00
Cohesion [t/m2]...............................: 0.00
Allowable stress on foundation [t/m2].........: 50.00
Water level [m]...............................: 0.50

DATA ABOUT ADDITIONAL FOUNDATION SOILS

Layer	Depth [m]	Unit weight [t/m3]	Cohesion [t/m2]	Frict.angle [deg]

DATA ABOUT DISTRIBUTED LOADS

Gabion Walls calculation - GAWAC R 2.0
Dr. Sawan Himmo

File: 6M   Project : 6M

****
RESULTS   PAGE 3
****

EXTERNAL STABILITY

Active thrust [t/m]............................:	58.22
Point of application ref. to X axis [m].......:	3.29
Point of application ref. to Y axis [m].......:	2.74
Direction of thrust ref. to X axis [deg]......:	55.26
Passive thrust [t/m]...........................:	0.13
Point of application ref. to X axis [m].......:	0.01
Point of application ref. to Y axis [m].......:	0.10
Direction of thrust ref. to X axis [deg]......:	0.00

SLIDING

Normal force on the base [t/m]................:	74.20
Point of application ref. to X axis [m].......:	1.37
Point of application ref. to Y axis [m].......:	-0.12
Shear force on the base [t/m].................:	30.21
Resisting force on the base [t/m].............:	46.38
Safety coefficient............................:	1.528

OVERTURNING

Overturning moment [tm/m].....................:	99.36
Restoring moment [tm/m].......................:	201.37
Safety coefficient............................:	2.027

STRESSES ACTING ON FOUNDATION

```
Stress on outer foundation border [t/m2]......:   36.00
Stress on inner foundation border [t/m2]......:    0.00
Max. allowable stress on the foundation [t/m2]:   50.00
Notice: the base is partialized!
```