GEO Technical Guidance Note No. 57 (TGN 57)
Foundation Design of Flexible Debris-resisting Barriers

Issue No.: 1	Revision: -	Date: 13.12.2024	Page: 1 of 13

1. SCOPE

1.1 This Technical Guidance Note (TGN) stipulates the recommendations on the foundation design of flexible debris-resisting barriers.

1.2 Any feedback on this TGN should be directed to the Chief Geotechnical Engineer/Landslip Preventive Measures 2 of the Geotechnical Engineering Office (GEO).

2. TECHNICAL POLICY

2.1 The technical recommendations promulgated in this TGN were agreed by GEO Geotechnical Control Conference on 16 October 2024.

3. RELATED DOCUMENTS

3.1 BAFU (2018). *Principles for the Quality Assessment of Rockfall Protection Kits and their Foundations – Practice Manual*. Federal Office for the Environment (BAFU), Switzerland, 42 p.

3.2 BSI (2008). *Steel wire ropes – Safety – Part 2: Definitions, designation and classification (BS EN 12385-2:2002+A1:2008).* The British Standards Institution, U.K., 53 p.

3.3 BSI (2009). *Steel wire and wire products – Non-ferrous metallic coatings on steel wire – Part 2: Zinc or zine alloy coatings (BS EN 10244-2:2009).* The British Standards Institution, U.K., 18 p.

3.4 BSI (2011). *Geotechnical investigation and testing – Field testing – Part 3: Standard penetration test (BS EN ISO 22476-3:2005+A1:2011).* The British Standards Institution, U.K., 14 p.

3.5 BSI (2018). *Code of Practice for Grouted Anchors (BS 8081:2015+A2:2018).* The British Standards Institution, U.K., 116 p.

3.6 GEO (2023). *Guide to Soil Nail Design and Construction (Geoguide 7).* Continuously Updated E-version released on 21 November 2023. Geotechnical Engineering Office, Hong Kong, 90 p.

3.7 GEO (2019). *Detailing of Flexible Debris-resisting Barriers (GEO Technical Guidance Note No. 48).* Geotechnical Engineering Office, Hong Kong, 9 p.

3.8 GEO (2024a). *Design of Flexible Debris-resisting Barriers using Force Approach (GEO Technical Guidance Note No. 55).* Geotechnical Engineering Office, Hong Kong, 6 p.

3.9 GEO (2024b). *Design of Flexible Debris-resisting Barriers using Force Approach (GEO Technical Guidance Note No. 56)*. Geotechnical Engineering Office, Hong Kong, 4 p.

3.10 HKSARG (2023). *General Specification for Civil Engineering Works (Continuously Updated Version incorporating Amendments)*. Hong Kong S.A.R. Government.

3.11 Kwan, J.S.H. & Cheung, R.W.M. (2012). *Suggestions on Design Approaches for Flexible Debris-resisting Barriers (Discussion Note DN 1/2012)*. Geotechnical Engineering Office, Hong Kong, 90 p.

3.12 Wong, E.K.L. (2024). *Final Review of Foundation Design for Flexible Debris-resisting Barriers (Technical Note No. TN 3/2024)*. Geotechnical Engineering Office, Hong Kong, 14 p.

3.13 Wong, E.K.L. & Lam, H.W.K. (2020). *Preliminary Review of Foundation Design for Flexible Debris-resisting Barriers (Technical Note No. TN 4/2020)*. Geotechnical Engineering Office, Hong Kong, 57 p.

3.14 Wong, E.K.L. & Lam, H.W.K. (2021). *Supplementary Review of Foundation Design for Flexible Debris-resisting Barriers (Technical Note No. TN 2/2021)*. Geotechnical Engineering Office, Hong Kong, 40 p.

3.15 Wong, E.K.L., Sze, E.H.Y. & Chung, P.W.K. (2022). *Study of Energy Transfer and Stress Wave Propagation during SPT using Energy Measurements, High Speed Camera and Particle Image Velocimetry (Special Project Report No. SPR 2/2022)*. Geotechnical Engineering Office, Hong Kong, 31 p.

4. BACKGROUND

4.1 Guidance on the design of flexible debris-resisting barriers using the Force Approach and Energy Approach is given in GEO TGN No. 55 (GEO, 2024a) and GEO TGN No. 56 (GEO, 2024b) respectively.

4.2 Aspects of foundation design for flexible barriers were reviewed in Wong & Lam (2020), Wong & Lam (2021) and Wong (2024), including failure mechanisms, design loads, capacity of foundation elements, factors of safety and durability.

4.3 Following the technical development work above involving numerical analyses, landslide studies and back analyses of impact cases on flexible barriers, this TGN makes recommendations on the foundation design of flexible debris-resisting barriers.

5. TECHNICAL RECOMMENDATIONS

5.1 Foundation Load

5.1.1 For flexible barriers designed using the Force Approach, all feasible load cases should be considered. The most critical combination of the impact velocity, debris geometry and impact location should be used to derive the foundation loads. Reasonable assumptions as to the width and depth of the debris should be made with regard to findings of the Natural Terrain Hazard Study and the Design Event concerned.

5.1.2 For flexible barriers designed using the Energy Approach, it is in general conservative to adopt design foundation loads based on the measured peak cable forces during full-scale rockfall tests. (Wong & Lam, 2020). The manufacturer's specifications on the values of foundation loads on the basis of full-scale rockfall tests should in general be followed.

5.1.3 In the absence of foundation loads specified by the flexible barrier manufacturer, the foundation loads for upslope anchors and side/lateral anchors should be taken as the corresponding peak cable loads measured during full-scale rockfall tests.

5.1.4 The foundation compressive load in post base anchors should be calculated from the total measured peak forces in the cables acting at the post top, resolved in the direction of the compressive anchors.

5.1.5 The foundation shear load in post base anchors typically arises from the deflection of the bottom cables upon debris impact. The shear load may be calculated from the maximum measured peak forces in the bottom cables connected to the post base and the deflection angles of the cable during a full-scale rockfall test at maximum elongation of the barrier net (Figure 1).

If the deflection angles are not known, a reasonable assumption may be made following BAFU (2018) in which the post foundation shear force is taken as the measured bottom cable force ($S = F_b$).

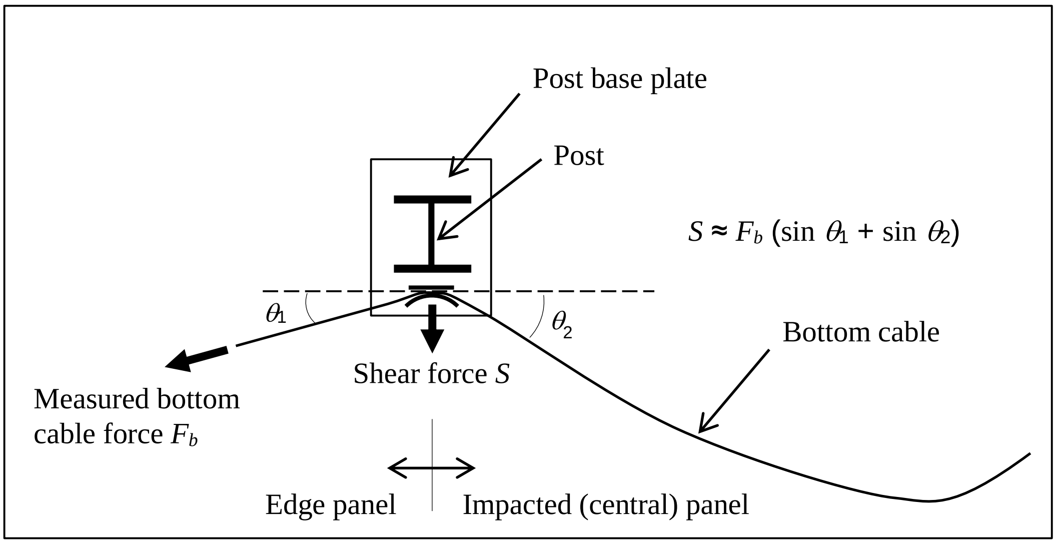


Figure 1. Deflection angle of bottom cables at maximum net elongation during full-scale rockfall test

5.1.6 The impact of debris on the posts of flexible barriers and the drag force exerted on the posts as debris flows around the posts should be taken into account when deriving the foundation loads. The forces may be calculated from Equation (1).

$$F = \tfrac{1}{2}\ C_d\, \rho\, v^2 A \qquad (1)$$

where F = force on post due to debris impact
C_d = drag coefficient, taken as 2.0
ρ = mass density of debris, taken to be 2200 kg/m^3 as a minimum
v = debris velocity
A = projected area of obstruction normal to flow

5.1.7 The static pressure of debris that has stopped and deposited behind the barrier should be considered when deriving the foundation loads. The static pressure may be determined using a coefficient of lateral earth pressure of 1.0.

5.1.8 A load factor of 1.0 is appropriate for foundation loads to be sustained by flexible debris-resisting barriers.

5.2 Design of Wire Rope Anchors

5.2.1 The tensile capacity of wire ropes used in anchors should be calculated from Equation (2).

$$F_{Rd} = F_{\min} / 1.5 \qquad (2)$$

where $F_{\min}$ = minimum breaking force of wire rope

The minimum breaking force should be determined in accordance with BS EN 12385-2 (BSI, 2008) or from the manufacturer's declared breaking force as supported by tensile tests.

5.2.2 The ultimate bond strength between the grout sleeve and wire ropes may be taken as 2 MPa, with a minimum grout compressive strength 30 MPa. A factor of safety of 2.0 should be applied to derive the allowable pull-out resistance.

5.2.3 The pull-out resistance provided by the bond between the grout sleeve and the ground may be determined using the effective stress approach as recommended for soil nails in Geoguide 7 (GEO, 2023).

Alternatively, designers may use well-established empirical methods to determine the design pull-out resistance. Due consideration should be given to the similarity of ground conditions and installation and grouting methods relevant to the empirical relationships adopted. For example, BS 8081 (BSI, 2018) gives a correlation between the ultimate bond strength between the grout sleeve and weathered granites with blow counts from the standard penetration test (SPT). If such an empirical correlation is used for preliminary design of anchors, site specific SPT should be carried out to determine a blow count which is representative of the ground condition along the length of the anchor. The SPT blow count should be adjusted to a reference energy ratio of 60% following BS EN ISO 22476-3 (BSI, 2011). Wong et al. (2022) documented the range of typical energy efficiency of SPT equipment used in Hong Kong.

Bond resistance is not mobilised uniformly along the length of an anchor. The total pull-out resistance does not increase linearly with anchor length. An appropriate efficiency factor should be applied to the ultimate bond strength.

The maximum allowable pull-out resistance provided by the soil-grout bond should not exceed the rock-grout bond strength of 0.35 MPa for partially weathered rock mass of PW 90/100 or better rock zone as stated in Geoguide 7. For wire rope anchors socketed into rock, the guidance in Geoguide 7 should be followed for the bond strength between grout and the rock socket.

A factor of safety of 2.0 should be applied to derive the allowable pull-out resistance.

The bond length of an anchor should not be less than 3 m in soil or 2 m in rock.

5.2.4 Pull-out tests should be performed to verify design assumptions about the bond strength between the grout sleeve and the ground. The recommendations in Geoguide 7 should be followed for the test set-up and procedure for pull-out tests, with the maximum test load applied to be T_{DL2} (i.e. the allowable pull-out resistance of the anchor times the factor of safety against pull-out failure at soil-grout interface). The anchors tested shall not be used as working anchors.

The grouted section of a tension anchor prepared for the pull-out test should be at least 2 m. The material and size of reinforcement, hole diameter and inclination, and the type of grout of the test anchor shall be the same as that of the working anchor. The top of the grouted section shall be at least 5m into the ground along the direction of the drilled hole. The test section chosen shall be representative of the average ground conditions.

The number of pull-out tests to be carried out shall be 5% of the number of working tension anchors but not less than 2 for each entire continuous stretch of flexible barrier. The tests shall be carried out at locations representative of the ground conditions of the stretch of barrier.

Where an empirical correlation between bond strength and SPT is adopted, pull-out tests should be carried out at depths corresponding to the SPT blow count used for deriving the bond strength.

5.2.5 Wire rope anchors should be provided with Class A zinc coating in accordance with BS EN 10244-2 (BSI, 2009). The diameter of individual wires in a wire rope anchor should not be less than 2.8 mm.

5.2.6 If the soil at a site is “potentially aggressive” as defined in Geoguide 7, a detailed soil aggressivity assessment should be carried out in accordance with Geoguide 7. Corrugated plastic sheathing in accordance with the General Specification for Civil Engineering Works (HKSARG, 2023) should be provided if the site is classified as “aggressive” or “highly aggressive”.

5.3 Design of Bar Anchors

5.3.1 Where steel reinforcing bars are used as foundation anchors for flexible barrier structures, the recommendations in Geoguide 7 should be followed to determine the tensile capacity and bond strength between the anchor and the grout sleeve.

5.3.2 Paragraphs 5.2.3 and 5.2.4 apply to bar anchors in tension.

5.3.3 For bar anchors in compression, the buckling capacity of the anchor should also be checked.

5.3.4 Proof-load test for working anchors taking tension or compressive load is not required.

5.3.5 The guidelines on the provision of corrosion protection measures for soil nails in Geoguide 7 should be followed for bar anchors.

5.4 Design of Flexible Anchor Head

5.4.1 Flexible anchor heads typically consist of a wire rope bent to create an eye loop. In a double-leg wire rope anchor, the wire rope bends around a thimble and/or a U-shaped steel tube and both legs, which are typically of equal length, are inserted into a drillhole. In a single-leg wire rope anchor, the eye loop is typically held in place by wire rope clips (U-bolt grips) or ferrules and the wire rope terminates a short distance from the ferrule or final wire rope clip. The capacity of a flexible anchor head should be determined from the capacity of the eye loop as a whole instead of the breaking load of the constituent wire rope alone.

5.4.2 The allowable tension resistance F_{Rd} of a single-leg wire rope anchor head should be calculated from Equation (3)

$$F_{Rd} = F_{\min} k_e / 1.5 \qquad (3)$$

where $F_{\min}$ = minimum breaking force of wire rope
k_e = loss factor, taken as 0.9 for eye loop constructed from ferrule-secured eye or 0.8 for wire rope clips

5.4.3 Double-leg wire rope anchor heads are typically proprietary products, with breaking loads specified by manufacturers. The breaking load increases with the diameter of the loading pin (Figure 2) used for testing the wire rope anchor (Wong & Lam, 2021).

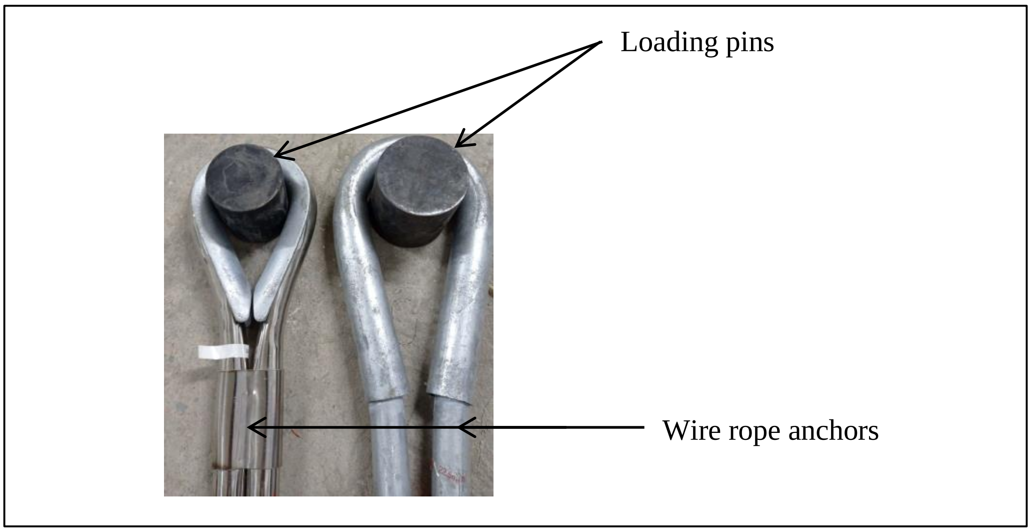


Figure 2. Loading pins used in tensile tests for double-leg wire rope anchors

In the field, loading is normally applied to anchors through shackles or by connecting superstructure wire rope cables to the anchor eye loop directly (Figure 3). The loading condition in tensile tests for establishing the breaking load of wire rope anchors should be consistent with the loading condition in the field. The diameter of loading pins therefore should not normally be greater than 50 mm, unless otherwise justified by the actual loading condition in the field. Shackles of bow diameter less than 50 mm may also be used for transferring tensile load to a wire rope anchor during testing (Wong & Lam, 2021). The allowable tension resistance F_{Rd} of a double-leg wire rope anchor head should be calculated from Equation (4)

$$F_{Rd} = F_{\min,50} / 1.5 \qquad (4)$$

where $F_{min,50}$ = minimum breaking force of wire rope anchor head with loading pin diameter < 50 mm

Alternatively, the breaking load achieved using a loading pin with diameter greater than 50 mm may be multiplied by 0.75 to give the value of $F_{min,50}$ for use in Equation (4).

Loading applied through shackle

Loading applied through wire rope

Figure 3. Field loading conditions

5.4.4 The axis of wire rope anchors should be aligned with the resultant cable load direction as far as practicable in order to minimise any unbalanced lateral force (Wong & Lam, 2021).

5.4.5 At the anchorages for bottom longitudinal cables in a flexible barrier, it is typically not feasible to align the resultant cable load with the axis of the anchor. Given that the bottom cables are critical to the integrity of a flexible barrier system with limited redundancy, thrust blocks should be provided at the anchorages of bottom cables for post-supported flexible barriers. The design chart provided in Annex TGN 57 A1 may be used for the range of geometry and cable force specified. Alternatively, suitable engineering principles may be followed to design the thrust blocks. Where a pseudo-static analysis is adopted, the minimum factor of safety against bearing failure of a thrust block may be taken as 1.1.

5.5 Design of Rigid Anchor Head

5.5.1 Rigid anchor heads are typically used in conjunction with bar anchors. The capacity of all individual components of a rigid anchor head shall be checked in accordance with relevant structural design codes and guidelines. For steel components embedded in reinforced concrete, adequate anchorage lengths should be provided.

5.5.2 In the absence of detailed design or tensile tests, a single 40 mm diameter high yield steel reinforcing bar bent into a U-shaped connector with a leg spacing of 500 mm embedded in a reinforced concrete block may be assumed to have an allowable capacity of 550 kN if the cable forces are applied within ±30° from the anchor axis and are inclined upwards by not greater than 45° (Figure 4).

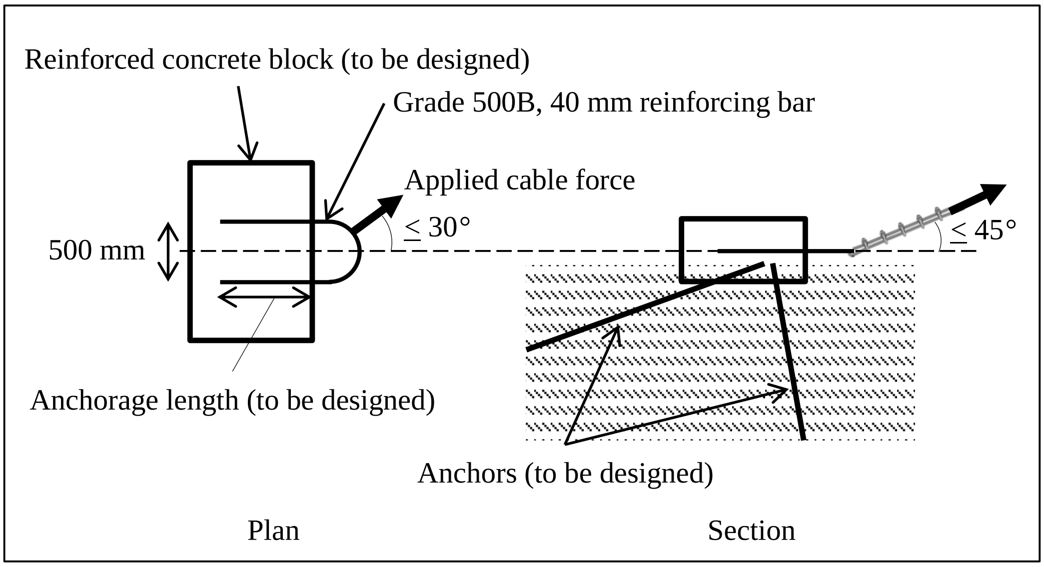
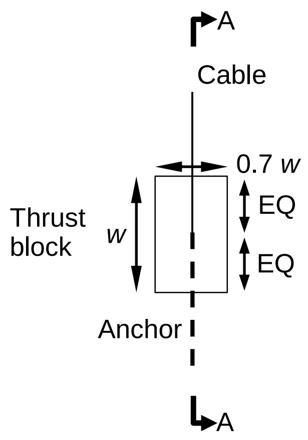


Figure 4. Prescriptive rigid anchor head

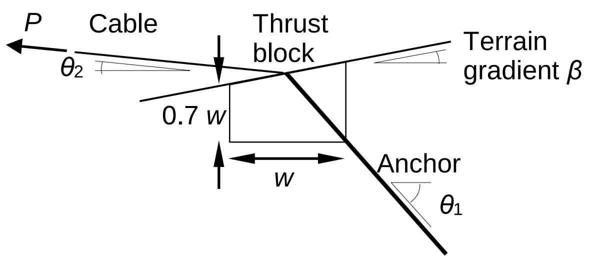
6. ANNEX

6.1 TGN 57 A1 – Design chart for thrust block

(Raymond W M Cheung)
Head, Geotechnical Engineering Office



Plan



Section A-A

See Sheets 2/4 to 4/4 for minimum values of *w*

β = 0°, $P \leq$ 150 kN

θ_2 \ θ_1	10°	20°	30°	40°
0°	0.35	0.45	0.60	0.85
10°		0.35	0.55	0.80
20°			0.45	0.70

β = 0°, $P \leq$ 200 kN

θ_2 \ θ_1	10°	20°	30°	40°
0°	0.35	0.50	0.70	0.95
10°		0.40	0.60	0.90
20°			0.50	0.75

β = 0°, $P \leq$ 250 kN

θ_2 \ θ_1	10°	20°	30°	40°
0°	0.40	0.55	0.75	1.05
10°		0.45	0.65	0.95
20°			0.50	0.85

β = 0°, $P \leq$ 300 kN

θ_2 \ θ_1	10°	20°	30°	40°
0°	0.40	0.60	0.80	1.10
10°		0.45	0.70	1.00
20°			0.55	0.90

$\beta \leq$ 10°, $P \leq$ 150 kN

θ_2 \ θ_1	10°	20°	30°	40°
0°	0.40	0.55	0.70	0.95
10°		0.40	0.60	0.85
20°			0.50	0.75

$\beta \leq$ 10°, $P \leq$ 200 kN

θ_2 \ θ_1	10°	20°	30°	40°
0°	0.40	0.60	0.80	1.05
10°		0.45	0.70	0.95
20°			0.55	0.85

$\beta \le 10°$, $P \le 250$ kN

θ_2 \ θ_1	10°	20°	30°	40°
0°	0.45	0.65	0.85	1.15
10°		0.50	0.75	1.05
20°			0.60	0.90

$\beta \le 10°$, $P \le 300$ kN

θ_2 \ θ_1	10°	20°	30°	40°
0°	0.50	0.70	0.90	1.25
10°		0.55	0.80	1.10
20°			0.60	0.95

$\beta \le 20°$, $P \le 150$ kN

θ_2 \ θ_1	10°	20°	30°	40°
0°	0.45	0.65	0.80	1.05
10°		0.50	0.70	0.95
20°			0.55	0.85

$\beta \le 20°$, $P \le 200$ kN

θ_2 \ θ_1	10°	20°	30°	40°
0°	0.40	0.55	0.70	0.90
10°		0.45	0.60	0.85
20°			0.50	0.70

$\beta \le 20°$, $P \le 250$ kN

θ_2 \ θ_1	10°	20°	30°	40°
0°	0.55	0.75	1.00	1.30
10°		0.60	0.85	1.20
20°			0.65	1.00

$\beta \le 20°$, $P \le 300$ kN

θ_2 \ θ_1	10°	20°	30°	40°
0°	0.55	0.80	1.05	1.40
10°		0.65	0.90	1.25
20°			0.70	1.10

β = 30°, $P \leq$ 150 kN

θ_2 \ θ_1	10°	20°	30°	40°
0°	0.60	0.80	1.05	1.30
10°		0.65	0.90	1.15
20°			0.70	1.00

β = 30°, $P \leq$ 200 kN

θ_2 \ θ_1	10°	20°	30°	40°
0°	0.65	0.90	1.15	1.45
10°		0.70	1.00	1.30
20°			0.80	1.15

β = 30°, $P \leq$ 250 kN

θ_2 \ θ_1	10°	20°	30°	40°
0°	0.70	1.00	1.25	1.60
10°		0.75	1.10	1.45
20°			0.85	1.25

β = 30°, $P \leq$ 300 kN

θ_2 \ θ_1	10°	20°	30°	40°
0°	0.75	1.05	1.35	1.70
10°		0.80	1.15	1.55
20°			0.90	1.35

Thrust block width (m)	Reinforcement
$w < 0.4$	3B12 U-bars both ways
$0.4 \leq w < 0.7$	3B16 U-bars both ways
$0.7 \leq w < 1$	4B16 U-bars both ways
$1 \leq w \leq 1.3$	6B16 U-bars both ways

Note: B denotes Grade 500B ribbed reinforcement.