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Scour Depth Computation in Soft Rock

Satyendra Mittal, V.A. Sawant, J.P. Sahoo*

**Department of Civil Engineering, IIT Roorkee, Roorkee*

**Email: satyendramittal@gmail.com*

ABSTRACT

There is a large impetus globally on infrastructure development. Many bridges have to be constructed on all type of available strata. There are no clear guidelines for computations of scour depth in weathered rock. The Lacey's theory is applicable to alluvium soils only. Sometimes, the soft/weathered rocks are presented at site which may have very low RQD, negligible core recovery and UCS to the order of 16 MPa. The difficulty arises when such rock is present at shallow depth which though is not strong enough for placement of foundation of a bridge pier which is supposed to bear heavy loads. In the present paper, a study has been carried out based on erodibility index of rock and scour depth computation had been done for a river having discharge of about 47,000 m^3/s. Rock chunk samples were collected from site for testing in labs.

Keywords: Scour depth; Soft rock; Erodibility index; Foundation depth; Well foundation

1. INTRODUCTION

A bridge is proposed over a river in eastern part of India. The bridge will have 49 piers (generally spaced at 3.0m c/c) and 2 abutments. The bridge piers had been proposed on open foundation. The geotechnical investigation at this site illustrates non-homogeneity in bore holes. The boreholes depict varying sequence of rock mass. For example, bore log for pier P-6 shows top 7.2m soft rock, whereas data for piers P-7 to P-13 shows hard rock in top 6m. The data at abutment A1 location shows sand at top 6.4m depth followed by soft rock. The pier P-1 location illustrates sand upto top 1.8m followed by soft rock and subsequently the hard rock. As the proposed structure is a bridge for which piers are going to be constructed in river bed, the scour depth computation is important for placement of foundation at appropriate depth. Though estimation of scour depth in soil is well established, limited studies are available for scour depth in soft rock. Hopkins and Beckham (1999) reported insignificant rock scour around exposed bridge piers. Only few sites out of 400 sites experienced rock scour holes. FHWA manual (FHWA, 2012) outlined a procedure for contraction scour in erodible rock. The bridge is located on river Mahanadi in Odisha.

This paper is specifically meant for piers P-4 to P-19, where RQD was very small upto 4.5 m of depth and core recovery was of the order of 32% upto 9 m depth.

2. GEOTECHNICAL FEATURES AT SITE

The rock present at P-4 site is by and large soft rock only. The authors visited the site and saw that the soft rock (present at site) was tried to be removed through heavy duty breakers only (Fig.1). Core recovery and RQD at this place were observed to be in the range of 8-10% and zero respectively (Fig. 2).

Figure 1: Rock cutting by breaker in progress at P4 site

The geological data of the site was not available and the study is based on hydrological and geotechnical data only.

The bore log data sheet (Fig. 2) pertains to pier P-4. The summary of geotechnical data of Piers P-4 to P-19 is given in Tables 3 (A) & 3 (B). As indicated in the bore log, there is soft rock from 1.5m depth onwards at site (Mittal & Shukla, 2014). At such site, placement level of foundation is very difficult to decide due to non-estimation of accurate scour level.

2.2 Geotechnical Data

The rock samples collected from site were tested in the lab. The test results are given below in Table 1 for the samples collected from the representative location P-4.

Table 1: Compressive strength test results of rock samples

BH. No.	Core depth in m	Length in mm	Diameter in mm	Length-Dia ratio (L/D)	Correction factor for L/D	Cross-sectional area (mm^2)	Compressive load (kN)	Compressive Strength (N/mm^2)	Corrected compressive strength (N/mm^2)	Crushing strength (MPa)	Rock SBC after considering factor of safety (t/m^2)
P-4	3.0 to 4.5	67	53	1.26	0.92	2206.5	33.4	15.14	13.93	13.93	174.1
	4.5 to 6.0	89	53	1.68	0.96	2206.5	36.7	16.63	15.97	15.97	199.6
	6.0 to 7.5	99	53	1.87	0.99	2206.5	39.2	17.77	17.59	17.59	219.9
	7.5 to 9.0	106	53	2	1	2206.5	46.4	21.03	21.03	21.03	262.9
	Note : Considering Factor of Safety = 5 as per IRC: 78-2000										

Notation: SBC - Safe bearing capacity, c-cohesion, Φ – Angle of internal friction

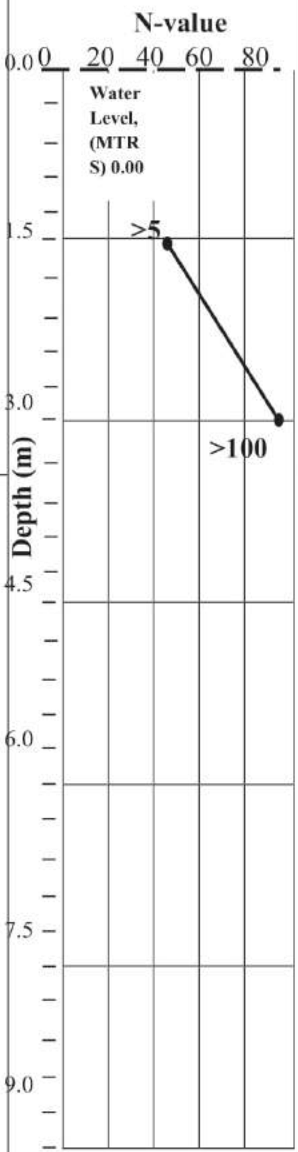
Thickness of strata & Soil profile	Visual representation of Strata	Visual description of soil strata	N (SPT) value	SPT (N-Value) vs. Depth plot	Type of sample	Core recovery (%)	RQD (%)
0-1.5m	0.00	Sand (SP)					
1.5 m to 5.40		Broken rock pieces were collected	50 100		Rock Rock	8 10	0 0
5.4 m to 9 m		Rock samples were collected			Rock Rock Rock Rock	12 24 43 64	0 12 19 32

Figure 2: Bore log data of site, near pier P-4

Table 2 summarizes the geotechnical parameters of locations, where soft rock was absent. Table 3 (A) and Table 3 (B) enlist the borehole data of P-4 to P-11 and P-12 to P-19 respectively.

Table 2: Summary of geotechnical parameters and safe bearing capacity

BH No.	Depth BGL	Type of sample	Specific gravity	SPT-'N' value	Shear Parameters		Computed SBC (t/m^2)	ABP-25 (t/m^2)	ABP-40 (t/m^2)
					c (kg/cm^2)	Φ (°)			
A-1	1.5	SPT	2.60	33	-	-	13.16	-	-
	3.0	SPT	2.59	42	-	-	21.45	-	-
	4.5	SPT	-	>50	-	-	83.94	-	-
A-2	1.5	SPT	2.71	25	-	-	14.63	-	-
	3.0	UDS	2.71	-	0.32	8	13.37	-	-
	4.5	SPT	2.58	44	-	-	31.76	-	-
P-1	1.50	SPT	2.65	32	-	-	13.16	11.30	18.08
P-2	1.50	SPT	-	>50	`10	-	31.14	-	-
P-3	1.50	SPT	-	>50	-	-	31.14	-	-
P-48	1.50	SPT	2.69	11	-	-	14.37	-	-
	3.00	UDS	2.70	-	0.29	11	13.47	-	-
P-49	1.50	SPT	2.67	28	-	-	14.64	18.94	30.30
	3.00	SPT	2.65	>50	-	-	57.14	16.67	26.67
Remarks - All the tests were conducted as per BIS specifications									

Notations: ABP-25&40 - Allowable bearing pressure for 25mm and 40mm of settlement respectively, c-cohesive strength, Φ – Angle of internal friction

Table 3 (A): Summary of bore-logs from P-4 to P-11

Sl. No.	Depth (m)	Item	P-4	P-5	P-6	P-7	P-8	P-9	P-10	P-11
1	0 to 2	A	-	-	-	16.72	17.99	18.72	18.04	16.72
		B	8%, NIL	NIL, NIL	NIL, NIL	36%, NIL	36%, NIL	22%, NIL	32%, NIL	18%, NIL
		C				200.7	200.2	200.2	215.3	200.7
2	2 to 4	A	15.14		15.19	17.86	23.43	19.08	25.74	18.99
		B	10%, NIL	NIL, NIL	8%, NIL	43%, 12%	56%, 14%	28%, NIL	38%, NIL	26%, NIL
		C	174.1		169	223.2	281.2	214.7	318.6	220.8
3	4 to 6	A	16.63	15.46	16.68	19.53	33.18	23.93	25.7	20.3
		B	12%, NIL	NIL, NIL	10%, NIL	62%, 19%	68%, 24%	43%, 11%	53%, NIL	37%, 11%
		C	199.6	173.9	185.5	244.2	414.7	299.1	321.2	238.6
4	6 to 8	A	17.77	17.27	18.21	21.48		26.47	28.91	
		B	49%, 16%	NIL, NIL	18%, 5%	69%, 28%	83%, 32%	54%, 22%	68%, 20%	49%, 16%
		C	219.9	201.2	217.1	268.5		330.8	361.4	
5	8 to 10	A	21.03	19.35	21.35					
		B	64%	13.3%, NIL	22%, 12%					
		C	262.9	338	261.3					
6	10 to 12	A								
		B	67%, 27%							
		C								

Notations: A = Compressive strength (N/mm^2); B = Core recovery and RQD; C= Rock SBC with FOS = 5

Table (3B): Summary of bore-logs from P-12 to P-19

Sl. No.	Depth (m)	Item	P12	P13	P14	P15	P16	P17	P18	P19
1	0 to 2	A	18.22	18.22-						
		B	18%, NIL	16%, NIL	NIL, NIL	NIL, NIL	NIL, NIL	NIL, NIL	NIL, NIL	NIL, NIL
		C	202.7	202.7						
2	2 to 4	A	22.39	18.94		12.60	9.95			
		B	20%, NIL	17%, NIL	NIL, NIL	NIL, NIL	NIL, NIL	NIL, NIL	NIL, NIL	NIL, NIL
		C	260.3	217.9		140.2	110.7			
3	4 to 6	A	23.93	21.17	13.10	13.96	21.17			16.77
		B	34%, 12%	37%, NIL	16%, NIL	18%, NIL	24%, NIL	NIL, NIL	NIL, NIL	NIL, NIL
		C	290.1	261.9	145.7	155.3	261.9			188.6
4	6 to 8	A	24.88	25.29	20.94	22.52	22.57	14.71	15.59	19.35
		B	38%, 14%	38%, 12%	32%, NIL	33%, NIL	18%, NIL	NIL, NIL	NIL, NIL	19%, NIL
		C	311.1	316.1	243.4	261.8	265.2	163.7	173.4	227.4
5	8 to 10	A			27	26.2	23.43	19.83	19.35	18.9
		B			38%, NIL	39%, NIL	24%, NIL	18%, NIL	28%, 10%	38%, 12%
		C			325.2	317.6	272.4	247.6	241.9	231.5
6	10 to 12	A						22.57	23.48	21.26
		B			27%, NIL			49%, 16%	54%, 28%	54%, 16%
		C						282.1	293.5	265.7

Notations: A = Compressive strength (N/mm^2); B = Core recovery and RQD; C= Rock SBC with FOS = 5

On the basis of test results and as per Mittal & Shukla (2014) it is found out that the foundation level should be decided by ensuring minimum level of depth of footing as follows:

(i) Minimum foundation depth should be equal to 1500mm in rocks having UCS < 12.5 MPa
(ii) Minimum foundation depth should be equal to 600mm in rocks having UCS > 12.5 MPa

3. HYDRAULIC DATA OF SITE

Hydraulic data of the site were as follows:

a) Design discharge : 47018m^3/s
b) Maximum mean-velocity of flow : 3.54m/sec
c) HFL : 148.961m
d) LWL : 139.00m
e) Maximum scour level : top of rock
f) Vertical clearance above HFL : 1.50m
g) Formation level : 152.87m
h) Lowest bed level : 137m

The scour depth has been computed keeping all above data in view.

4. COMPUTATIONS OF SCOUR DEPTH

For alluvium soils, the Lacey's theory is well accepted theory for computation of scour depth. But here, it could not be straight away used as here the soil type is varying in nature, which has soft rock and hard rock also. Following calculations uses data from Tables 4 to 8 (FHWA, 2012) given in Appendix - I.

According to FHWA (2012), the erodibility index of rocks is given as

$$K = M_s \times K_b \times J_s \times K_d \tag{1}$$

where M_s = intact rock mass strength parameter = 8.39 for very weak rock (Table 4)

K_b = block size parameter

$= RQD/J_n$ (2)

$K_b = 1/5 = 0.2$ (Presuming $RQD = 1$ and rock joint set number $J_n = 5$ for multiple joint /fissure sets; Table 5)

J_s = relative orientation parameter = 0.5 (assumed in the absence of detailed geological data, Table 8.

K_d = Shear Strength parameter $K_d = J_r/J_a$ (3)

In the absence of geological data on joint sets, values of joint roughness number J_r is taken as 1 (Table 6) and joint alternation number J_a is taken as 10 (Table 7).

Hence from Eq. 1

$K = 8.39 \text{ x } 0.2 \text{ x } 0.5 \text{ x } 0.1 = 0.0839 \approx 0.084 \approx 0.1$

However, as a general rule, rock masses on which bridge piers are founded typically exhibit erodibility index *K* values ranging from 0.1 (very poor rock) up to 10000 or greater (very good rock). The computed value of K is computed as 0.084, which may be taken equal to 0.1

Adopting K = 0.1, the critical stream power Pc for initiating quarrying and plucking is related to K as given by Annandale (1995, 2006)

$$P_c = K^{0.75} = 0.1^{0.75} = 0.17783\, kW/m^2 \tag{4}$$

As developed by Annandale (1995), the stream power is calculated by considering the turbulence production near the bed of the stream:

$$P_a = 7.853\, \rho \left(\frac{\tau}{\rho}\right)^{1.5} W/m^2 \tag{5}$$

where τ = bed shear stress of approach flow (N/m^2), and ρ = mass density of water = 1000 kg/m^3.

In Eq. 5, P_a is expressed in units of W/m^2, whereas the critical shear stress P_C (given by Eq. 4) is expressed in kW/m^2. To convert P_a to KW/m^2, the value from Eq. 5 shall be divided by 1000.

In the vicinity of a bridge pier, the downward flow at the upstream face of the pier creates additional local turbulence in the form of horse shoe vortex. As scour occurs, the stream power (P) at the bottom of the scour hole decreases as the scour hole becomes deeper. Scour will continue until the stream power at the bottom of the scour hole becomes less than the critical stream power (P_c) at which point the scouring process can no longer be sustained. The relationship relating the relative depth of the scour hole to the stream power at the bottom of the hole for a variety of pier shapes (round, square and rectangular) can be expressed as:

$$\frac{P}{P_a} = 8.42\, e^{-0.712(y_s/b)} \quad W/m^2 \tag{6}$$

where

P = Stream power at the bottom of the scour hole (W/m^2)
P_a = Stream power of the approach flow near the stream bed (W/m^2)
y_s = Scour depth (m) = 12m
b = Pier width perpendicular to the flow direction (m) =1.2m

Now, bed shear stress of approach flow as per FHWA (2012)

$$\tau = \gamma y_s S_f \tag{7}$$

Where

S_f = slope of energy grade = 1 in 1000 (Say)
γ = weight of water (N/m^3) = 9800 N/m^3

Therefore

$$\tau = 9800 \times 12 \times \frac{1}{1000} = 117.6\ N/m^2$$

From Eq. 5,

$$P_a = 7.853 \times 1000 \times \left(\frac{117.6}{1000}\right)^{1.5} = 316.7\ W/m^2$$

$$or, \quad P_a = 0.3167\ kW/m^2$$

For critical condition as per FHWA guidelines, using $P = P_c$ in Eq. 6,

$$P_c = P_a \times 8.42\, e^{-0.712(y_s/b)}$$

$$0.17783 = 0.3167 \times 8.42\, e^{-0.712(y_s/b)}$$

$$or, \quad e^{-0.712(y_s/b)} = \frac{0.17783}{0.3167 \times 8.42} = 0.066687$$

$$or, \quad -0.712\left(y_s/b\right) = \ln\left(0.066687\right) = -2.707745$$
$$\left(y_s/b\right) = 3.803$$

$$y_s = 3.803 \times b = 3.803 \times 1.2 = 4.563\ m$$

Say 4.5 m

Taking an increase of 30% in above value, i.e. 4.5x1.3=5.85 (say 6m). This value is increased to account for the abrasion of bed rock.

5. FIXATION OF FOUNDATION LEVEL

Based on above calculation, the scour depth is computed as 5m from general bed level. Hence if the bed level is 137m, The scour level shall be 132m (Annandale, 1995 & 2006). The general cross-section with various levels is shown in Fig.3.

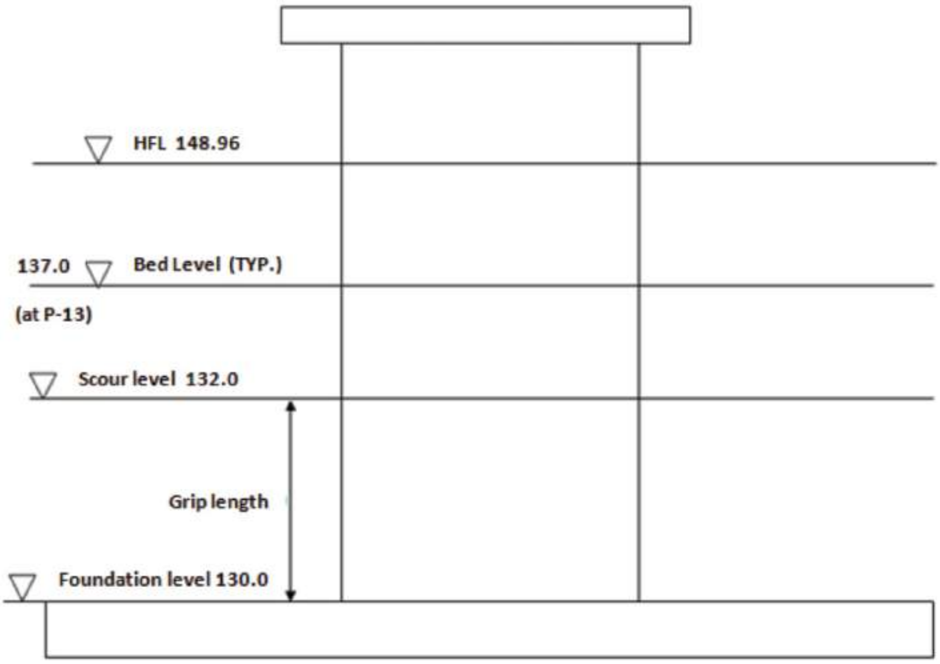


Figure 3: A general cross-section, showing various levels

According to Ranjan & Rao (2000) the foundation depth below scour level should in no case be less than 2 m for piers and abutments with arches and 1.2 m for piers and abutments in other structures (IS: 3955, 1967 and IRC specification, 1966). Grip length of 2m has been adopted here. Therefore foundation level shall be 132.0 - 2.0 = 130.0 for pier (Fig.3).

6. CONCLUSIONS

On the basis of FHWA guidelines and following other hydraulic principles and also considering the geotechnical data, following conclusions are made:

- The rivers flow through varying strata. Computation methods of scour depth in alluvium strata are well established. But computation in soft rock/ weathered rock is a tricky issue. The present study is based on actual geotechnical and hydrological data of a river flowing in Odisha state, where a bridge has to be constructed. The different piers of bridge have to be constructed on different soil strata. FHWA guidelines have been used in the present study for computation of scour depth in soft rock, based on erodibility criterion. The values computed from these guidelines were found reasonable.

- The minimum grip length for well foundation may be adopted 2m as per guidelines suggested by Mittal & Shukla (2014), Mittal (2013) and Annandale (1995, 2006), Ranjan and Rao (2000) and IS:3955 (1967),IRC (1996).

- The present study encourages further research in this area, particularly duly supported by model tests.

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Appendix-I

Table 4: Values of rock mass strength parameter Ms (FHWA, 2012)

Hardness	Identification in Profile	Unconfined Compressive Strength (MPa)	Mass Strength Number (Ms)
Very soft rock	Material crumbles under firm (moderate) blows with sharp end of geological pick and can be peeled off with a knife	< 1.7	0.87
	Material is too hard to cut triaxial sample by hand	1.7–3.3	1.86
Soft rock	Can just be scraped and peeled with a knife	3.3–6.6	3.95
	Indentations 1 to 3 mm show in the specimen with firm (moderate) blows of the pick point	6.6–13.2	8.39
Hard rock	Cannot be scraped or peeled with a knife; hand-held specimen can be broken with hammer end of geological pick with a single firm (moderate) blow	13.2–26.4	17.70
Very hard rock	Hand-held specimen breaks with hammer end of pick under more than one blow	26.4–53.0 53.00–106.0	35.0 70.0
Extremely hard rock	Specimen requires many blows with geological pick to break through intact material	> 212.0	280.0

Table 5: Rock joint set number J_n (FHWA, 2012)

Number of Joint Sets	Joint Set Number (J_n)
Intact, no or few joints/fissures	1.00
One joint/fissure set	1.22
One joint/fissure set plus random	1.50
Two joint/fissure sets	1.83
Two joint/fissure sets plus random	2.24
Three joint/fissure sets	2.73
Three joint/fissure sets plus random	3.34
Four joint/fissure sets	4.09
Multiple joint/fissure sets	5.00

Table 6: Rock joint roughness number J_r (FHWA, 2012)

Condition of Joint	Joint Roughness Number (J_r)
Stepped joints/fissure	4.0
Rough or irregular, undulating	3.0
Smooth undulating	2.0
Slickensided undulating	1.5
Rough or irregular, planar	1.5
Smooth planar	1.0
Slickensided planar	0.5
Joints/fissures either open or containing relatively soft gouge of sufficient thickness to prevent joint/fissure wall contact upon excavation	1.0
Shattered or micro-shattered clays	1.0

Table 7: Rock joint alteration number J_a (FHWA, 2012)

Description of Gouge	Joint Alteration Number (J_a) for Joint Separation (mm)		
	1.0[(1)]	1.0–5.0[(2)]	5.0[(3)]
Tightly healed, hard, non-softening impermeable filling	0.75	-	-
Unaltered joint walls, surface staining only	1.0	-	-
Slightly altered, non-softening, non-cohesive rock mineral or crushed rock filling	2.0	2.0	4.0
Non-softening, slightly clayey non-cohesive filling	3.0	6.0	10.0
Non-softening, strongly over-consolidated clay mineral filling, with or without crushed rock	3.0	6.0**	10.0
Softening or low friction clay mineral coating and small quantities of swelling clays	4.0	8.0	13.0
Softening moderately over-consolidated clay mineral filling, with or without crushed rock	4.0	8.0**	13.0
Shattered or micro-shattered (swelling) clay gouge, with or without crushed rock	5.0	10.0**	18.0

Note: *(1). Joint walls effectively in contact, (2) Joint walls come into contact after approximately 100 mm shear, (3) Joint walls do not come into contact at all upon shear. ** Also applies when crushed rock occurs in clay gouge without rock wall contact*

Table 8: Relative orientation parameter J_s (FHWA, 2012)

Dip Direction of Closer Spaced Joint Set (degrees)	Dip Angle of Closer Spaced Joint Set (degrees)	Ratio of Joint Spacing, r			
Dip Direction	Dip Angle	Ratio 1:1	Ratio 1:2	Ratio 1:4	Ratio 1:8
180/0	90	1.14	1.20	1.24	1.26
In direction of stream flow	89	0.78	0.71	0.65	0.61
In direction of stream flow	85	0.73	0.66	0.61	0.57
In direction of stream flow	80	0.67	0.60	0.55	0.52
In direction of stream flow	70	0.56	0.50	0.46	0.43
In direction of stream flow	60	0.50	0.46	0.42	0.40
In direction of stream flow	50	0.49	0.46	0.43	0.41
In direction of stream flow	40	0.53	0.49	0.46	0.45
In direction of stream flow	30	0.63	0.59	0.55	0.53
In direction of stream flow	20	0.84	0.77	0.71	0.67
In direction of stream flow	10	1.25	1.10	0.98	0.90
In direction of stream flow	5	1.39	1.23	1.09	1.01
In direction of stream flow	1	1.50	1.33	1.19	1.10
0/180	0	1.14	109	1.05	1.02
Against direction of stream flow	–1	0.78	0.85	0.90	0.94
Against direction of stream flow	–5	0.73	0.79	0.84	0.88
Against direction of stream flow	–10	0.67	0.72	0.78	0.81
Against direction of stream flow	–20	0.56	0.62	0.66	0.69
Against direction of stream flow	–30	0.50	0.55	0.58	0.60
Against direction of stream flow	–40	0.49	0.52	0.55	0.57
Against direction of stream flow	–50	0.53	0.56	0.59	0.61
Against direction of stream flow	–60	0.63	0.68	0.71	0.73
Against direction of stream flow	–70	0.84	0.91	0.97	1.01
Against direction of stream flow	–80	1.26	1.41	1.53	1.61
Against direction of stream flow	–85	1.39	1.55	1.69	1.77
Against direction of stream flow	–89	1.50	1.68	1.82	1.91
180/0	–90	1.14	1.20	1.24	1.26

Note: (1) For intact material take J_s = 1.0, (2) For values of 'r' greater than 8, take J_s as for r = 8, and (3) If the flow direction (FD) is not in the direction of the true dip (TD), the effective dip (ED) is determined by adding the ground slope (GS) to the apparent dip (AD): ED = AD + GS