3.4.2 Access Road

1. Pavement Design

1.1 Design Approach

The pavement design procedure prevailing in Japan "Manual for Asphalt Pavement 1989", which includes the structure design method for rigid pavement, is applied to the study from following technical viewpoints;

1) It will accommodate considerable road traffic comprising heavy vehicles of large buses and trucks including 40 ft. container truck in future;
2) Loading conditions of heavy vehicles and their axle load distribution are deemed equivalent; and
3) Natural conditions such as high ground water level and annual rainfall are similar.

However, the proposed pavement structures are examined by design procedures presented in "AASHTO Guide for Design of Pavement Structures 1993".

1.2 Design Procedure

1.2.1 Classification of Roadway by Traffic Flow Volume

A pavement standard should be determined from five classifications as shown in Table 1.1 on the basis of the estimation of one-way daily traffic volume of heavy vehicles in the fifth year of operation. Heavy vehicles denote cargo trucks, buses, construction machines and special large-size motor vehicles.

Table 1.1 Road Classification by Traffic Volume

Classification	One-Way Daily Traffic Volume Of Heavy Vehicles
L	100 or less
A	101 to 250
B	251 to 1,000
C	1,001 to 3,000
D	more than 3,000

Forecasted traffic volume of the SSKB Access Road at the highest section is as shown in Table 1.2, and the estimation of one-way daily traffic volume of heavy vehicles in the fifth year of operation is 661 veh./day. Thus, the road classification of the SSKB Access Road is determined as "B".

Table 1.2 Forecasted Traffic Volume

Unit: Vehicles/day

Year	M.cycle	A.rickshaw	Car	Bus	Truck	Total	Bus/Truck Total
2005	26	209	78	266	226	804	492
2006	27	221	82	281	241	851	522
2007	29	233	86	296	257	901	554
2008	30	245	91	313	275	954	587
2009	32	259	96	330	294	1,010	623
2010	34	273	102	348	314	1,070	661
2011	36	290	108	369	332	1,134	701
2012	38	307	114	391	352	1,202	743
2013	40	325	121	414	373	1,274	788
2014	42	345	128	439	396	1,351	835
2015	45	366	136	466	420	1,433	886

1.2.2 Design CBR Value

(1) Preliminary Investigation and CBR Value

The design CBR value should be determined by sampling subgrade soils to design the thickness of the pavement.

The embankment structure of the SSKB comprises Lower Roadbed, Upper Roadbed and Improved Subgrade, and a subgrade refers to the soil about one (1) meter under the pavement. The lower roadbed will be built by side borrow materials of which CBR value may remain in a range of 2 by a normal compaction method, while the upper roadbed will be also built by the same materials but the CBR value will be increased up to 2 – 4 by a special compaction method. 30cm thick improved subgrade will be constructed by improved soils mixed side borrow materials and imported fill materials such as dredged sand, and the CBR value normally is expected more than 10.

(2) Determination of Design CBR Value

The average CBR value of the soils within 1m depth from the subgrade level should taken as the CBR value. The average CBR value is calculated according to the following formula:

$$CBR_m = \left(\frac{h_1 * CBR_1^{1/3} + h_2 * CBR_2^{1/3} \cdots\cdots + h_n * CBR_n^{1/3}}{100} \right)^3$$

where:

CBR_m : average CBR value

CBR_1, CBR_2 ...CBR_n : CBR value of soil layers No. 1, 2 ...n

h_1, h_2 .... h_n : thickness of soil layers No. 1, 2....n (cm)

$h_1 + h_2 \ldots + h_n$ = 100cm

Using the formula and applying h_1 = 70cm CBR_1 = 3, h_2 = 30cm CBR_2 = 10, the design CBR value is calculated 4.

(3) Design of Pavement Thickness

Using the design CBR value and the road classification, the pavement thickness of each layer is designed so that the desirable T_A value is assured, and the total thickness H should be larger than 80% of the target value in Table 1.3.

Table 1.3 Target Value for T_A and Total Thickness H

Design CBR	Target Value (cm)									
	L Traffic		A Traffic		B Traffic		C Traffic		D Traffic	
	T_A	H	T_A	H	T_A	H	T_A	H	T_A	H
2	17	52	21	61	29	74	39	90	51	105
3	15	41	19	48	26	58	35	70	45	90
4	14	35	18	41	24	49	32	59	41	70
6	12	27	16	32	21	38	28	47	37	55
8	11	23	14	27	19	32	26	39	34	46
12	-	-	13	21	17	26	23	31	30	36
20	-	-	-	-	-	-	20	23	26	27

(4) Determination of Pavement Structure

The following equations are used to determine a pavement structure.

$$T_A = a_1*T_1 + a_2*T_2 + \bullet\bullet\bullet\bullet\bullet + a_n*T_n$$

$$H = T_1 + T_2 + \bullet\bullet\bullet\bullet\bullet + T_n$$

where :

$a_1, a_2, \bullet\bullet\ a_n$: Conversion coefficient shown in Table 1.4 for reference.

$T_1, T_2 \bullet\bullet\ T_n$: Thickness of each layer (cm)

Table 1.4 additionally contains conversion coefficients for local materials commonly used in Bangladesh, which is referred to "Fig. 2.7 Various in Granular Subbase Layer Coefficient" contained in the AASHTO.

The minimum combined thickness of binder and surface courses excluding wearing course is specified in Table 1.5.

Table 1.5 Minimum Combined Thickness of Binder and Surface Courses

Road Classification	Minimum Combined Thickness (cm)
L, A	5
B	10 (5)
C	15 (10)
D	20 (15)

Note: Figures in parentheses indicate the minimum thickness applicable to pavement with a base course using the bituminous stabilization.

A final structure may be determined if it satisfy the required values shown in Tables 1.3 and 1.5.

Table 1.4 (1) Conversion Coefficient for T_A

Course	ethod and Mateerial of Constructio	Conditions	Coefficient a_n	
			per inch	per cm
Surface & Binder	Plant mixed dense asphalt concrete		0.44	1.00
Base	Bituminous stabilization	Stability: 350kgf or more	0.34	0.85
		Stability: 250kgf or more	0.22	0.55
	Cement stabilization	UC strength (7days): 30kgf/sq.cm or m	0.22	0.55
	Lime stabilization	UC strength (10days): 10kgf/sq.cm or	0.18	0.45
	Crushed stone for mechanical stabili	Modified CBR: 80 or more	0.14	0.35
	Slag for mechanical stabilization	Modified CBR: 80 or more	0.22	0.55
	Hydraukic slag	UC strength (14days): 12kgf/sq.cm or	0.22	0.55
Subbase	Crusheer-run, slag, sand, etc.	Modified CBR: 30 or more	0.10	0.25
		Modified CBR: 20 - 30	0.08	0.20
	Cement stabilization	UC strength (7days): 10kgf/sq.cm or m	0.10	0.25
	Lime stabilization	UC strength (10days): 7kgf/sq.cm or m	0.10	0.25

Table 1.4 (2) Conversion Coefficient for T_A

Description of Material	Soaked CBR	Coefficient a_n	
		per inch	per cm
Hand crushed bricks (with 0 to 20% local sand)	60	0.12	0.30
Well graded plant crushed bricks (0/37.5 mm)	150	0.14	0.35
Hand/Plant crushed bricks with 30%-50% local	30	0.11	0.28
Mixture of crushed boulders (30%), shingles(30%), pea-gravel (20%), sand (20%)	100~60	0.14	0.35
Mixture of Coarse (Sylhet)sand (40%) and local sand (60%)	30	0.11	0.28
Hand crushed boulder (60%), pea-gravel (20%), sand (20%)	80	0.13	0.33
Well graded plandt crushed boulders (0/37.5 m	150	0.14	0.35
Hand/ Plant crushed boulders with 50% local sa	30	0.11	0.28
Soil stabilized with lime	45~60	0.12	0.30
Sand/clay mixture mechanically stabilized	15	0.09	0.23
Local river sand and sandy silt	15	0.09	0.23

(5) Recommended Pavement Structure

The pavement structure is proposed as follows.

Surface & Binder Course: Plant-mixed Asphalt Concrete	10cm
Base Course: Hand crushed bricks (with 0 to 20% local sand)	20cm
Subbase Course: Sand/clay mixture mechanically stabilized	35cm

The proposed pavement structure has the design factors of T_A = 24.05 cm and H = 65cm to satisfy fully the required values.

3.5 Intersection Analysis

1) General

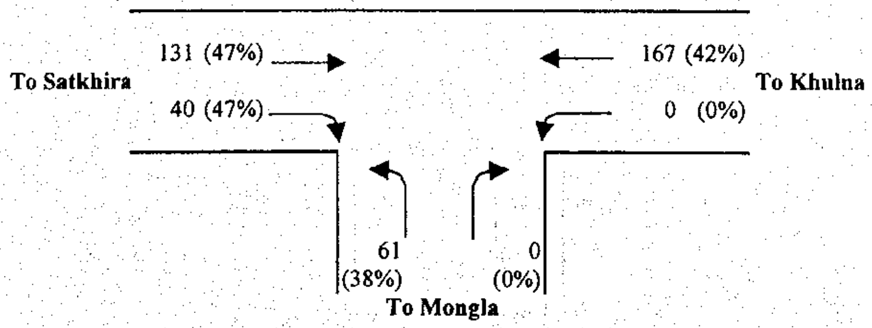
In The Study, 4 Intersections were planed. The intersection analyses were conducted by Japanese Standard. Because, The form of intersections was channelaized and vehicles are similar between Bangladesh and Japan.

Traffic Volume was used the result of traffic Analysis as shown Fig. 2.6.1.

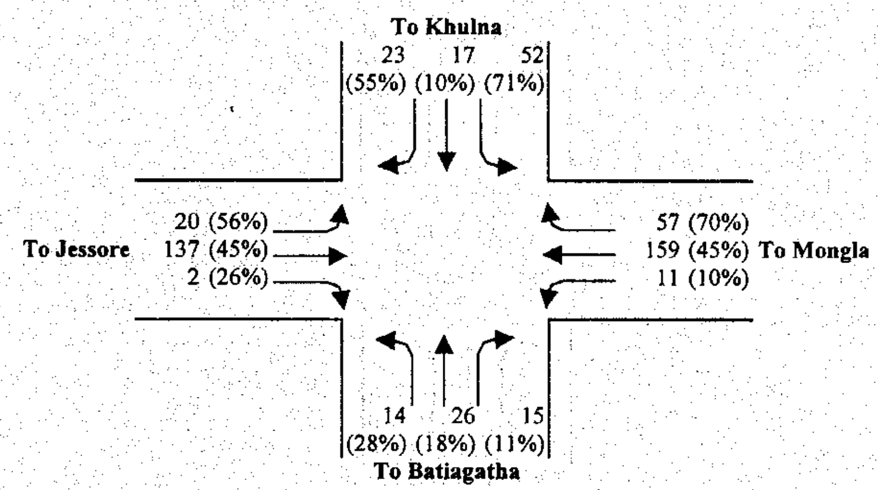
2) Results

According to traffic analysis, Congestion Indexes of all intersections are less than 0.9. And, Required Phase Rates are less than 1.0. Therefore, Layouts of intersection were adopted in The Study.

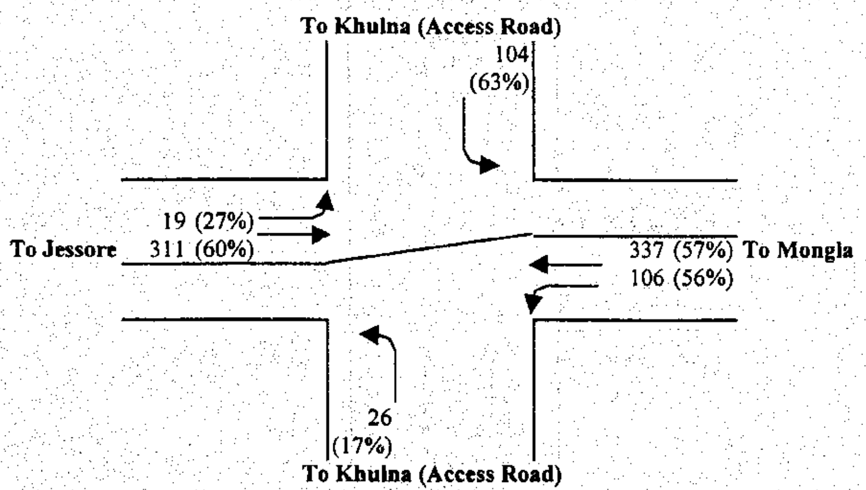
	Congestion Index	Required Phase Rate
A. Krishnagar	0.167	0.168
B. Mohanmad Nagar	0.228	0.200
C. Jabusa	0.439	0.337
D. Teelok	0.713	0.341



No.1 Krishnanagar



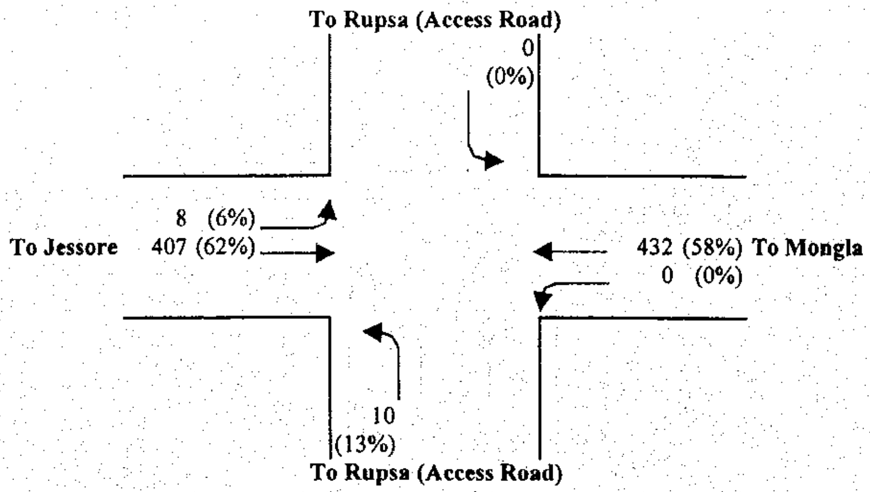
No.2 Mohanmad Nagar



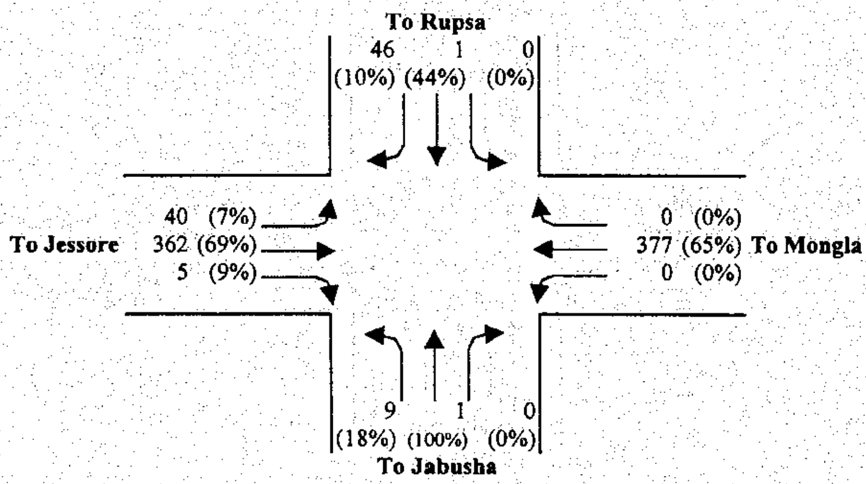
No.3 Laban Chara

Note: Figures show vehicle per hour
Parentheses show heavy vehicle rat

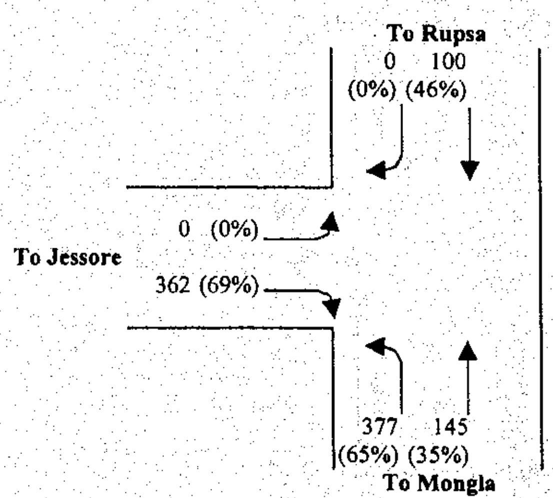
Fig. 2.6 (1) Volume by Direction by Hour in 2015



Eastern Dyke Road



Jabusha Road



Khulna-Mongla Road

Note: Figures show vehicle per hour
Parentheses show heavy vehicle rate

Fig. 2.6 (1) Volume by Direction by Hour in 2015

TABLE 1 INTERSECTION CAPACITY ANALYSIS (KRISHNAGAR)

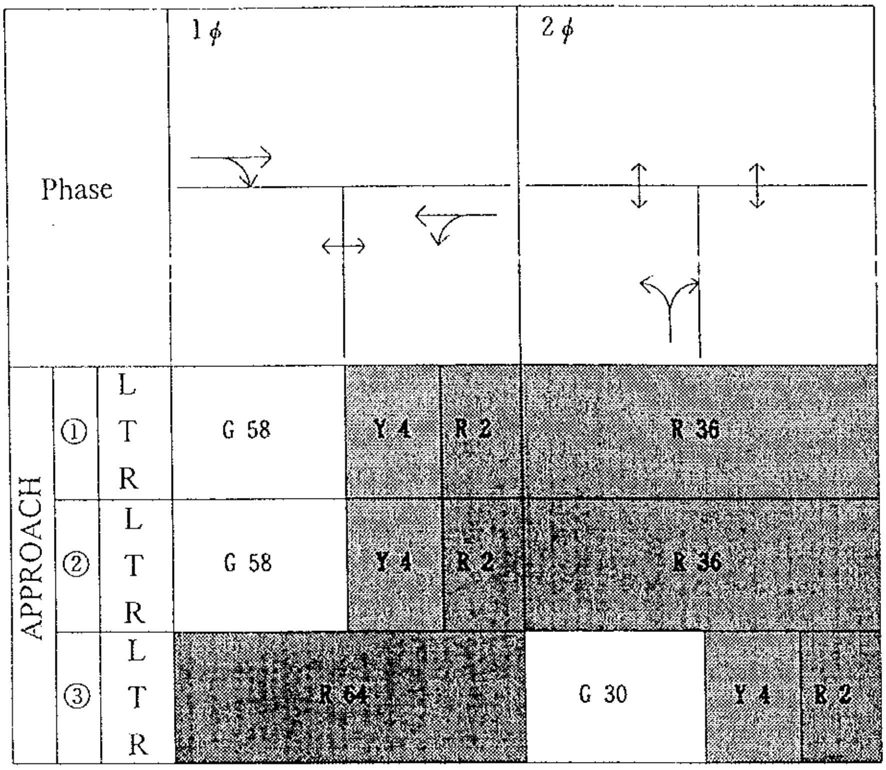
Approach		Satkhira		Khulna		Rupsa			
Movement		T	R	L	T	L	R		
Number of Lanes		1	1	1	1	1	1		
Base Flow Rate	SB	2,000	1,800	1,800	2,000	1,800	1,800		
Factor for Lateral Room	α W	1.000	1.000	1.000	1.000	1.000	1.000		
Width of Lane	(m)	3.00	3.00	3.00	3.00	3.50	3.50		
Factor for Grade	α G	1.000	1.000	1.000	1.000	1.000	1.000		
Factor for Heavy Vehicle	α T	0.752	0.752	1.000	0.773	0.790	1.000		
Heavy Vehcle Rate	(%)	47.00	47.00	0.00	42.00	38.00	0.00		
Factor for Mixed Left Turn Vehicle	α LT								
Left Turn Vehicle Rate	(%)								
Factor for Pedestrian	α L			1.000		1.000			
Factor for Mixed Right Turn Vehicle	α RT								
Right Turn Vehicle Rate	(%)								
Passage Probability of Right Turn Vehicle			0.841				1		
Actual Passage Time			58				30		
Right Turn Vehicle at Transition of Phases	N		3(81)				3(108)		
Saturation Traffic Flow Rate	SA	1,354	*628	1,620	1,391	1,280	1,620		
Traffic Volume	q	131	40	1	167	61	1		
Traffic Volume by Normal Distribution	ρ	0.097	-	0.001	0.120	0.048	0.001	By Phase	Total
Required Phase Rate	1 ϕ	0.097	-	0.001	0.120			0.120	0.168
	2 ϕ					0.048	0.001	0.048	
Green Signal Time	G/C	58/100	58/100	58/100	58/100	30/100	30/100		
Traffic Capacity	Ci	785	628	940	807	384	486		
Congestion Index	V/Ci	0.167	0.064	0.001	0.207	0.159	0.002		
Storage Length	(m)	20.2	6.2	0.1	24.9	14.7	0.2		

Notes.

R: Right, T: Through, L: Left, A: All Direction

N: Number of Right Turn Vehicle at Transition of Phases by Actual 1 Hour

*: Traffic Capacity (Actual 1 Hour)



NOTES: Figures are phase time (minutes).
R, T and L are right, through and left respectively.
G, Y and R are green, yellow and red respectively.

Fig. 2.6.2 Signal Phase Pattern (Krishnagar)

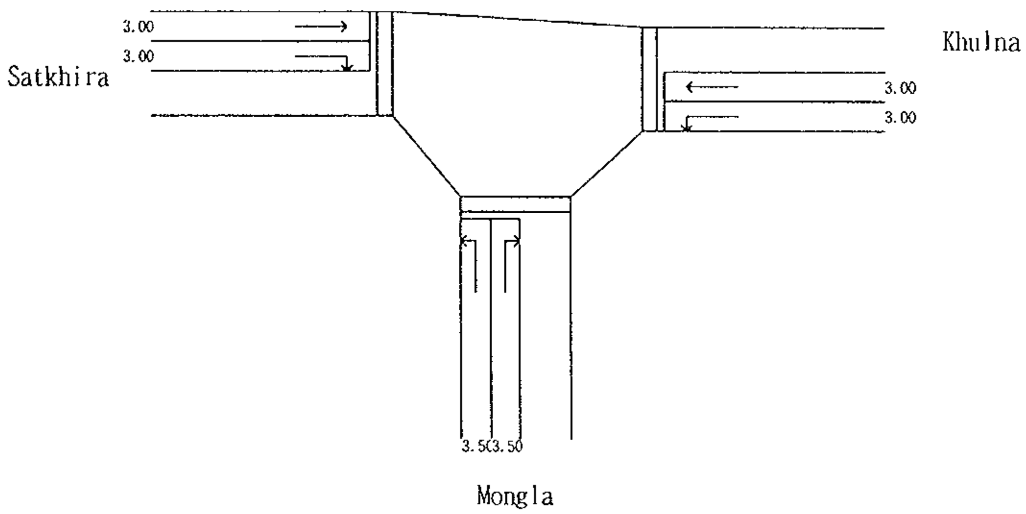


Fig.2.6.3 Intersection Layout (Krishnagar)

TABLE 2 INTERSECTION CAPACITY ANALYSIS (MOHANMAD NAGAR)

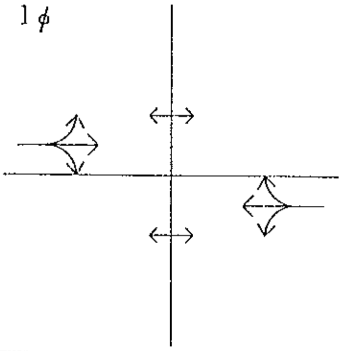
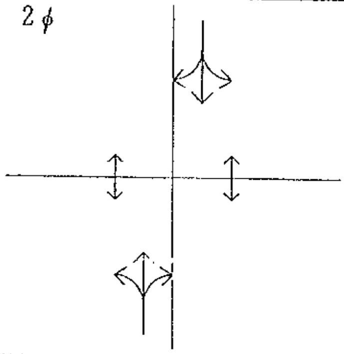
Approach		Jessore			Khulna	Mongla			Batiagata		
Movement		R	T	L	A	L	T	R	A		
Number of Lanes		1	1	1	1	1	1	1	1		
Base Flow Rate	SB	1,800	2,000	1,800	2,000	1,800	2,000	1,800	2,000		
Factor for Lateral Room	α W	1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.000		
Width of Lane	(m)	3.50	3.50	3.50	3.00	3.50	3.50	3.50	3.00		
Factor for Grade	α G	1.000	1.000	1.000	1.000	0.950	0.950	0.950	1.000		
Factor for Heavy Vehicle	α T	0.718	0.760	0.846	0.719	0.935	0.760	0.671	0.885		
Heavy Vehcle Rate	(%)	56.00	45.00	26.00	55.73	10.00	45.00	70.00	18.64		
Factor for Mixed Left Turn Vehicle	α LT				0.941				0.973		
Left Turn Vehicle Rate	(%)				56.5				25.5		
Factor for Pedestrian	α L	1.000				1.000					
Factor for Mixed Right Turn Vehicle	α RT				0.973				0.971		
Right Turn Vehicle Rate	(%)				25.0				27.3		
Passage Probability of Right Turn Vehicle				0.849	0.975			0.870	0.984		
Actual Passage Time				54	34			54	34		
Right Turn Vehicle at Transition of Phases	N			3(91)	3(77)			3(68)	3(95)		
Saturation Traffic Flow Rate	SA	1,163	1,368	*664	1,185	1,439	1,300	*516	1,505		
Traffic Volume	q	20	137	2	92	11	159	57	55		
Traffic Volume by Normal Distribution	ρ	0.017	0.100	-	0.078	0.008	0.122	-	0.037	By Phase	Total
Required Phase Rate	1 φ	0.017	0.100	-		0.008	0.122	-		0.122	0.200
	2 φ				0.078				0.037	0.078	
Green Signal Time	G/C	54/100	54/100	54/100	34/100	54/100	54/100	54/100	34/100		
Traffic Capacity	Ci	628	739	664	403	777	702	516	512		
Congestion Index	V/Ci	0.032	0.185	0.003	0.228	0.014	0.226	0.110	0.107		
Storage Length	(m)	3.6	23.0	0.3	23.6	1.4	27.4	11.1	10.8		

Notes.

R: Right, T: Through, L: Left, A: All Direction

N: Number of Right Turn Vehicle at Transition of Phases by Actual 1 Hour

*: Traffic Capacity (Actual 1 Hour)

Phase			1φ 			2φ 		
APPROACH	①	L T R	G 54	Y 4	R 1	R 41		
	②	L T R	R 59			G 34	Y 3	R 4
	③	L T R	G 54	Y 4	R 1	R 41		
	④	L T R	R 59			G 34	Y 3	R 4

NOTES: Figures are phase time (minutes).
R, T and L are right, through and left respectively.
G, Y and R are green, yellow and red respectively.

Fig. 2.6.4 Signal Phase Pattern (Mohanmad Nagar)

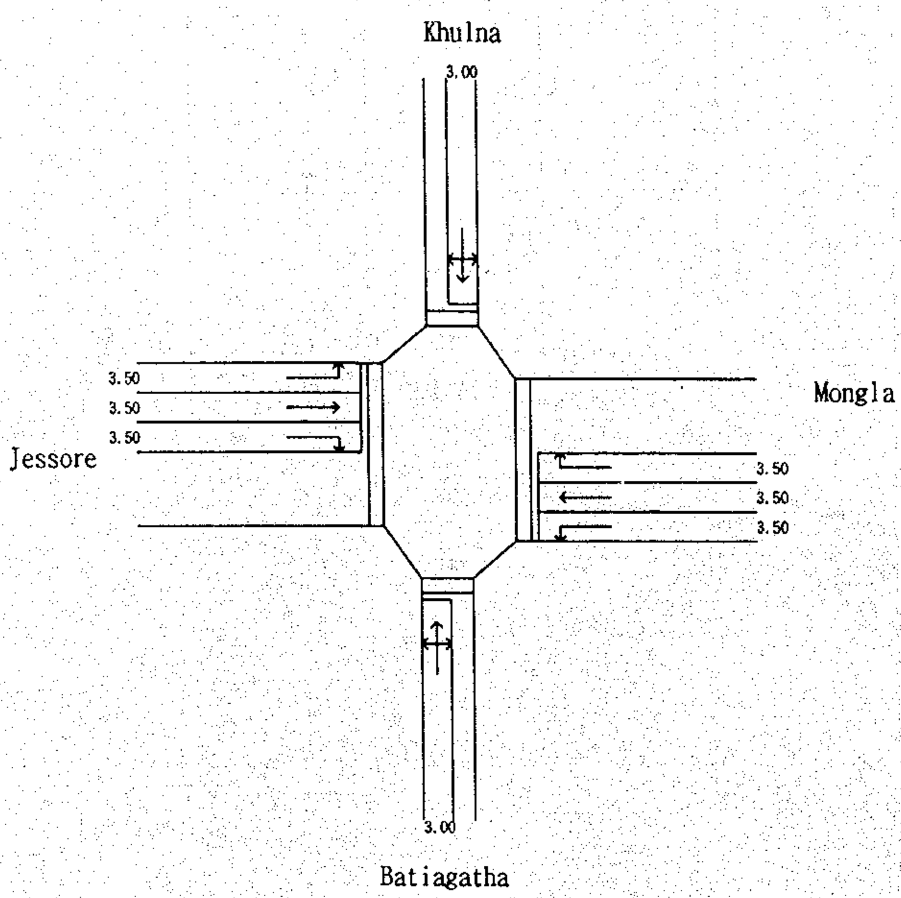


Fig.2.6.5 Intersection Layout (Mohanmad Nagar)

TABLE 3 INTERSECTION CAPACITY ANALYSIS (JABUSA)

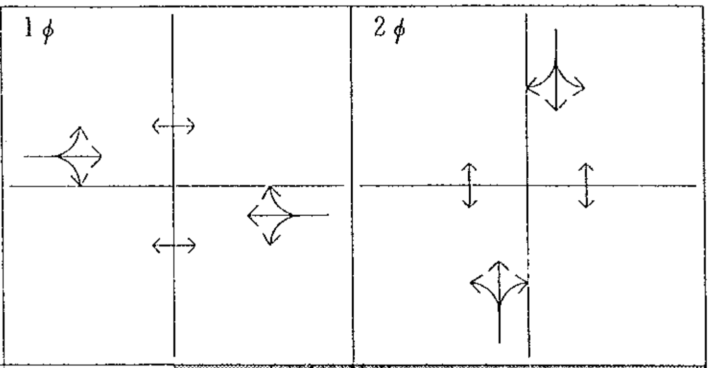
Approach		Jessore			Rupsa	Mongla			Jabusa		
Movement		R	T	L	A	L	T	R	A		
Number of Lanes		1	1	1	1	1	1	1	1		
Base Flow Rate	SB	1,800	2,000	1,800	2,000	1,800	2,000	1,800	2,000		
Factor for Lateral Room	α W	1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.000		
Width of Lane	(m)	3.50	3.50	3.50	3.00	3.50	3.50	3.50	3.00		
Factor for Grade	α G	1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.000		
Factor for Heavy Vehicle	α T	0.953	0.674	0.941	0.932	1.000	0.687	1.000	0.857		
Heavy Vehcle Rate	(%)	7.00	69.00	9.00	10.50	0.00	65.00	0.00	23.82		
Factor for Mixed Left Turn Vehicle	α LT				0.998				0.917		
Left Turn Vehicle Rate	(%)				2.1				81.8		
Factor for Pedestrian	α L	1.000				1.000					
Factor for Mixed Right Turn Vehicle	α RT				0.905				0.990		
Right Turn Vehicle Rate	(%)				95.8				9.1		
Passage Probability of Right Turn Vehicle				0.668	0.999			0.608	0.999		
Actual Passage Time				68	20			68	20		
Right Turn Vehicle at Transition of Phases	N			3(101)	3(100)			3(108)	3(92)		
Saturation Traffic Flow Rate	SA	1,544	1,213	*708	1,516	1,620	1,237	*768	1,400		
Traffic Volume	q	40	362	5	48	1	377	1	11		
Traffic Volume by Normal Distribution	ρ	0.026	0.298	-	0.032	0.001	0.305	-	0.008	By Phase	Total
Required Phase Rate	1 ϕ	0.026	0.298	-	-	0.001	0.305	-		0.305	0.337
	2 ϕ				0.032				0.008	0.032	
Green Signal Time	G/C	68/100	68/100	68/100	20/100	68/100	68/100	68/100	20/100		
Traffic Capacity	Ci	1050	825	708	303	1102	841	768	280		
Congestion Index	V/Ci	0.038	0.439	0.007	0.158	0.001	0.448	0.001	0.039		
Storage Length	(m)	3.4	48.9	0.4	10.6	0.1	49.7	0.1	2.7		

Notes.

R: Right, T: Through, L: Left, A: All Direction

N: Number of Right Turn Vehicle at Transition of Phases by Actual 1 Hour

*: Traffic Capacity (Actual 1 Hour)



Phase			1 φ			2 φ		
APPROACH	①	L T R	G 68	Y 4	R 1	R 27		
	②	L T R	R 73			G 20	Y 3	R 4
	③	L T R	G 68	Y 4	R 1	R 27		
	④	L T R	R 73			G 20	Y 3	R 4

NOTES: Figures are phase time (minutes).
R, T and L are right, through and left respectively.
G, Y and R are green, yellow and red respectively.

Fig. 2.6.6 Signal Phase Pattern (Jubusa)

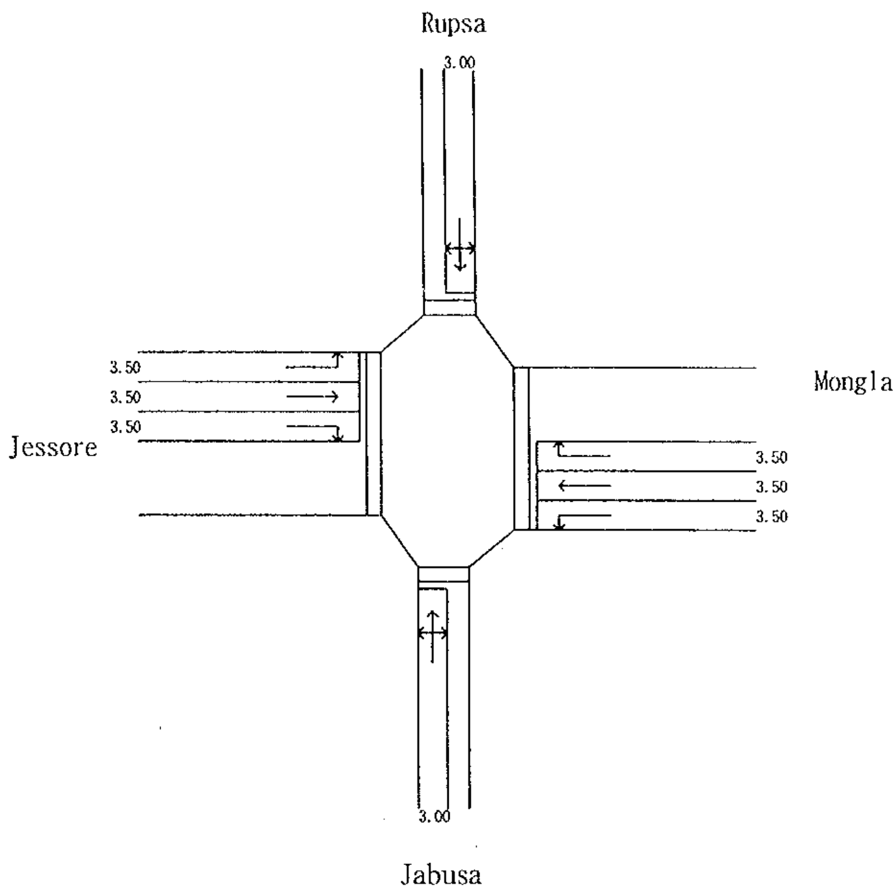


Fig.2.6.7 Intersection Layout (Jabusa)

TABLE 4 INTERSECTION CAPACITY ANALYSIS (TEELOK)

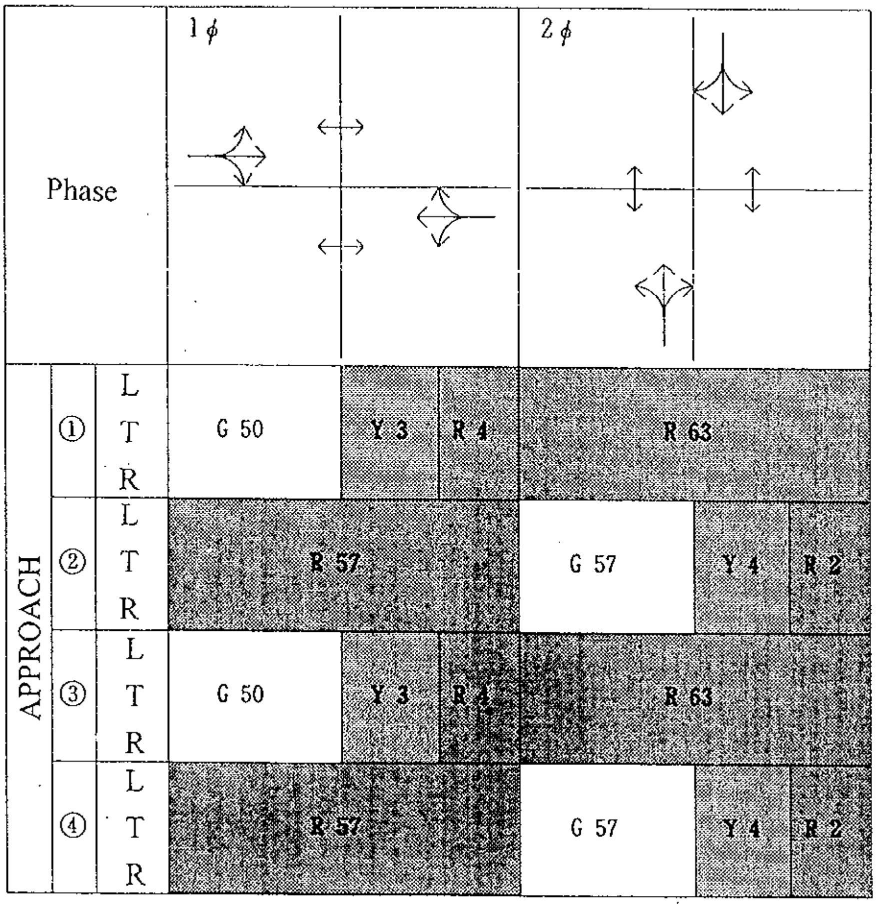
Approach		Jessore			Rupsa		Teelok	Mongla			By Phase	Total
Movement		R	T	L	L & T	R	A	L	T	R		
Number of Lanes		1	1	1	1	1	1	1	1	1		
Base Flow Rate	SB	1,800	2,000	1,800	2,000	1,800	2,000	1,800	2,000	1,800		
Factor for Lateral Room	α W	1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.000		
Width of Lane	(m)	3.50	3.50	3.50	3.50	3.50	3.50	3.50	3.50	3.50		
Factor for Grade	α G	1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.000		
Factor for Heavy Vehicle	α T	1.000	1.000	0.674	0.758	1.000	1.000	0.687	0.803	1.000		
Heavy Vehcle Rate	(%)	0.00	0.00	69.00	45.54	0.00	0.00	65.00	35.00	0.00		
Factor for Mixed Left Turn Vehicle	α LT				0.999		0.965					
Left Turn Vehicle Rate	(%)				1.0		33.3					
Factor for Pedestrian	α L	1.000						1.000				
Factor for Mixed Right Turn Vehicle	α RT						0.965					
Right Turn Vehicle Rate	(%)						33.3					
Passage Probability of Right Turn Vehicle				0.999		0.862	0.999			0.905		
Actual Passage Time				50		57	50			57		
Right Turn Vehicle at Transition of Phases	N			3(60)		3(90)	3(90)			3(90)		
Saturation Traffic Flow Rate	SA	1,620	1,800	*508	1,363	*687	1,676	1,113	1,445	*737		
Traffic Volume	q	1	1	362	101	1	3	377	145	1		
Traffic Volume by Normal Distribution	ρ	0.001	0.001	-	0.074	-	0.002	0.339	0.1	-		
Required Phase Rate	1 ϕ	0.001	0.001	-			0.002				0.002	0.341
	2 ϕ				0.074	-		0.339	0.1	-	0.339	
Green Signal Time	G/C	50/120	50/120	50/120	57/120	57/120	50/120	57/120	57/120	57/120		
Traffic Capacity	Ci	675	750	508	647	687	698	529	686	737		
Congestion Index	V/Ci	0.001	0.001	0.713	0.156	0.001	0.004	0.713	0.211	0.001		
Storage Length	(m)	0.1	0.1	71.4	15.4	0.1	0.3	65.3	20.6	0.1		

Notes.

R: Right, T: Through, L: Left, A: All Direction

N: Number of Right Turn Vehicle at Transition of Phases by Actual 1 Hour

*: Traffic Capacity (Actual 1 Hour)



NOTES: Figures are phase time (minutes).
R, T and L are right, through and left respectively.
G, Y and R are green, yellow and red respectively.

Fig. 2.6.8 Signal Phase Pattern (Teelok)

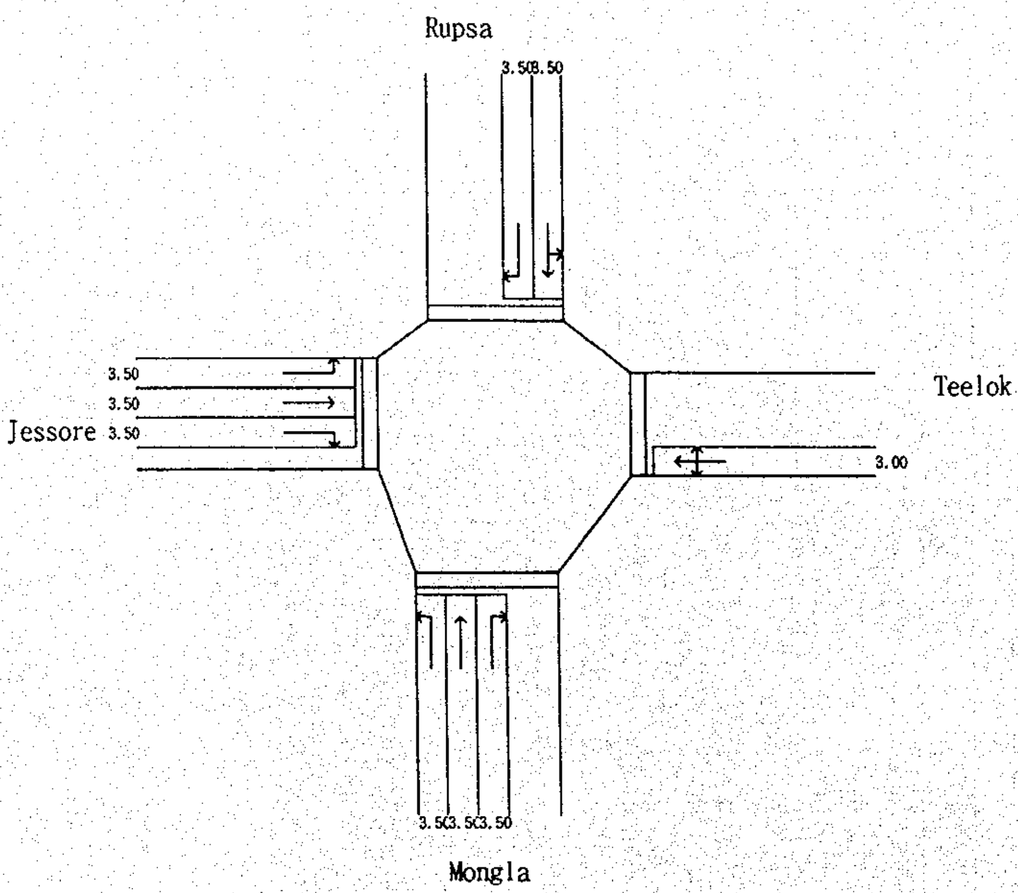


Fig.2.6.9 Intersection Layout (Teelok)

3.6 Geotechnical Engineering Analysis

3.6.1 Consolidation Settlement

Consolidation settlement of the very soft o soft silty and clayey soils is anticipated when the road embankment is placed on the ground.

(1) Magnitude of Settlement

The magnitude of estimated final settlement varies from 0.1 to 0.8m depending on the ground conditions and thickness of the fill placed. Table 3.6.1 shows the estimated final settlements for the highest fill height at each section.

Table 3.6.1 Estimated Consolidation Settlement

Section	Settlement (cm)	*Fill Thickness (m)
STA0 to STA 2+000	37	3.0
STA2+000 to Hatia West Bank	37	3.0
Hatia East Bank to STA3+700	27	2.3
STA3+700 to STA5+400	38	3.7
STA5+400 to STA6+500	44	4.4
STA6+500 to West Viaduct	38	3.7
East Viaduct to STA8+900	38	2.7
STA8+900 to STA9+900	31	4.0
STA9+900 to Molonghata Bridge	22	4.0
Molonghata Bridge to End	19	2.8
West Access Road (River Bank Side)	12	2.6
West Access Road (Viaduct Side)	30	2.6
East Access Road	13	2.0
Hatia Bridge West Approach	66	6.0
Hatia Bridge East Approach	49	5.8
West Viaduct Approach to	79	6.5
East Viaduct Approach to	77	6.5
Molonghata Bridge Approaches	26	4.8

* Fill thickness≒Design height＋Settlement

A following equation was used to estimate the consolidation settlement;

$$Sc = \sum_{i=1}^{n} (e_0 - e_1 / 1 + e_0)\, Hi \qquad \cdots\cdots\cdot \quad \text{Eq.3.6.1}$$

where Sc = primary consolidation settlement (m)

e_0 = initial void ratio at initial effective overburden pressure

e_1 = final void ratio at final effective overburden pressure

Hi = thickness of silty or clayey soil layer (m)

n = number of layer

Entire stretch of the Route 1 was divided into the sections of which boundaries were assumed to be present at the mid point of adjacent two boreholes. A ground model of the settlement analysis was prepared for the each section and is presented in Figures 1 through 19 in Appendix A. Wet densities of the layers shown in the models are estimated from Fig.3.6.1. e-logP curves shown in the models, which are necessary for determination of void ratios, were selected from the consolidation test results illustrated in Figures 3.6.2 through 3.6.3. The selection was made based on moisture contents, depth, and type of soil and N-values of each layer. Details of the calculation for the each section are tabulated in Settlement Calculation Sheet in Appendix B.

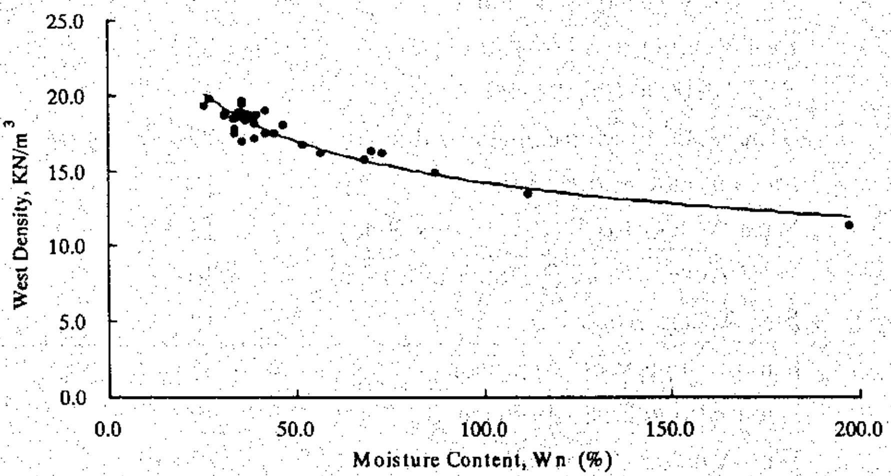


Fig. 3.6.1 West Density VS. Moisture Content

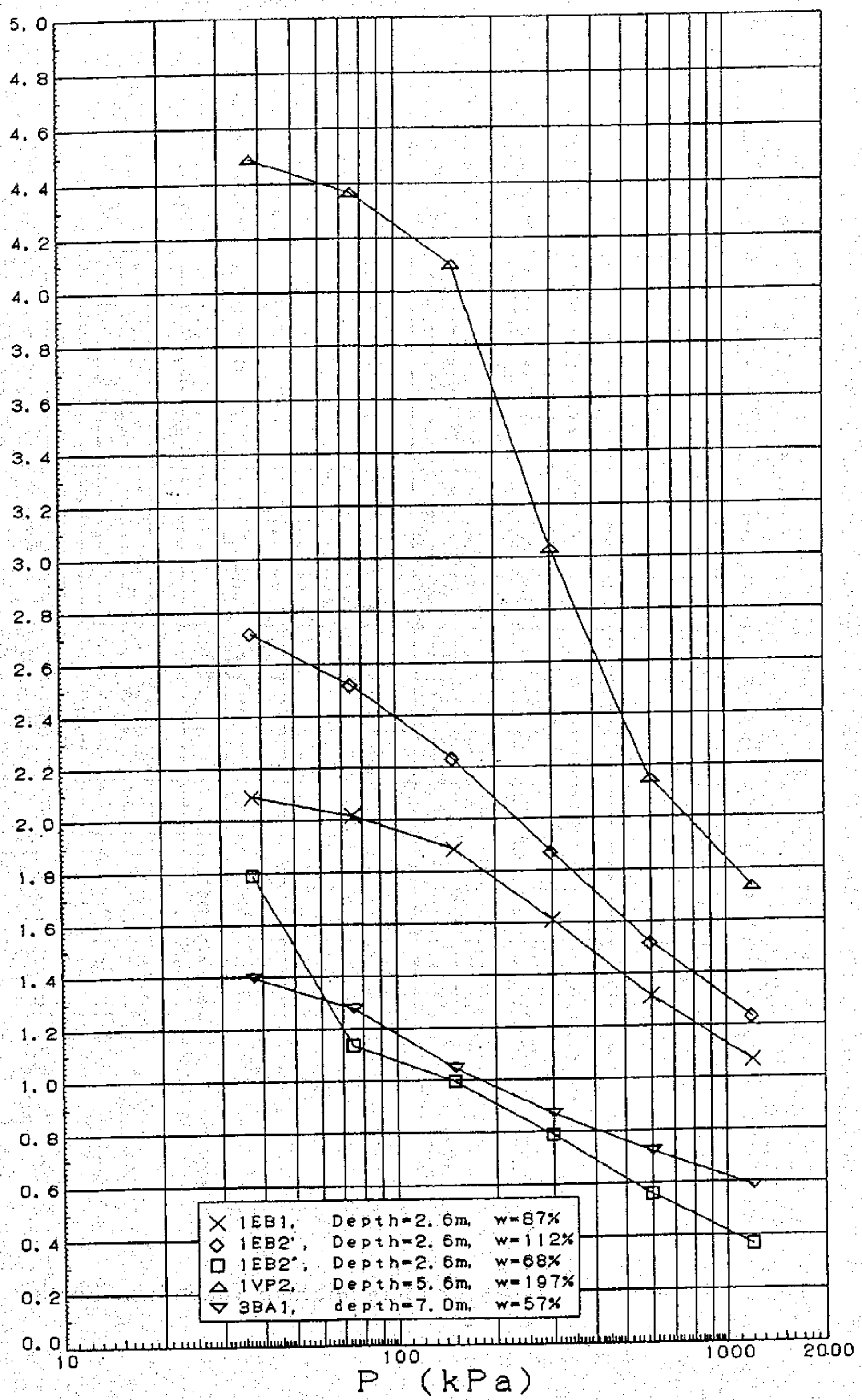


Fig. 3.6.2 e-Log P Curves for Soils with Organic Matter

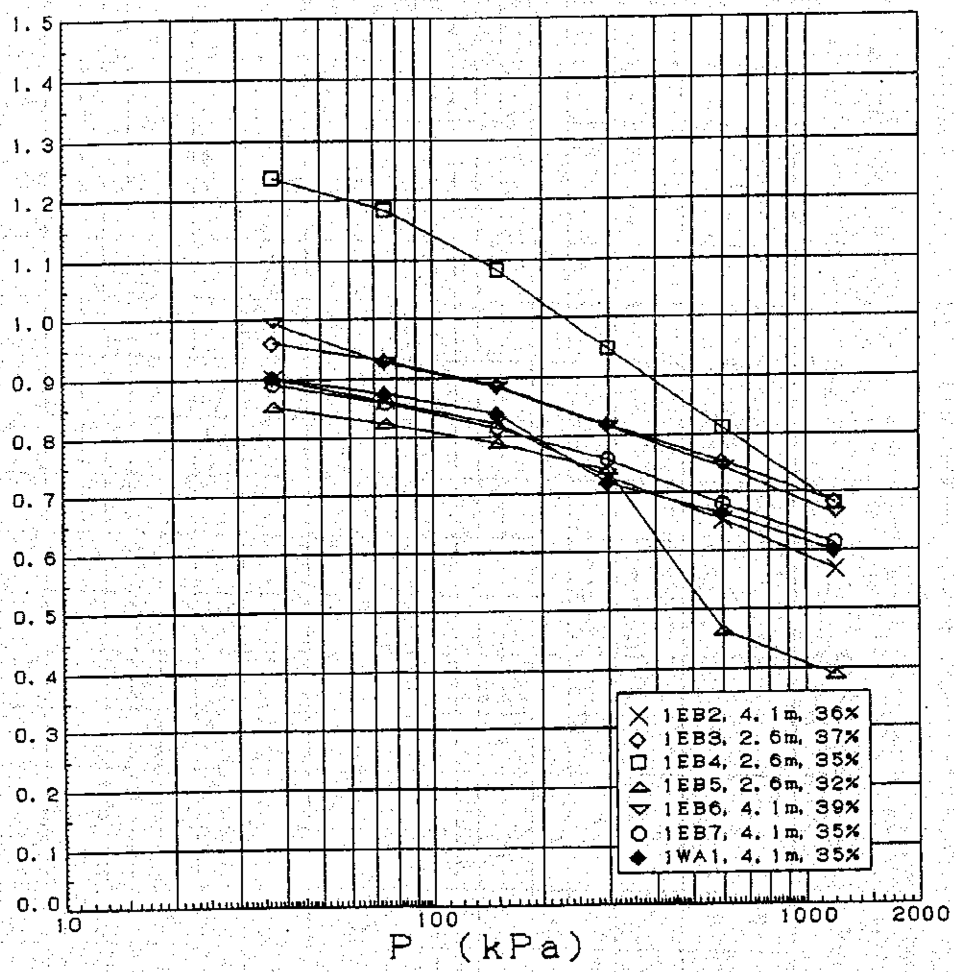


Fig. 3.6.3 e-Log P Curves for Inorganic Soils Taken from Shallow Depth

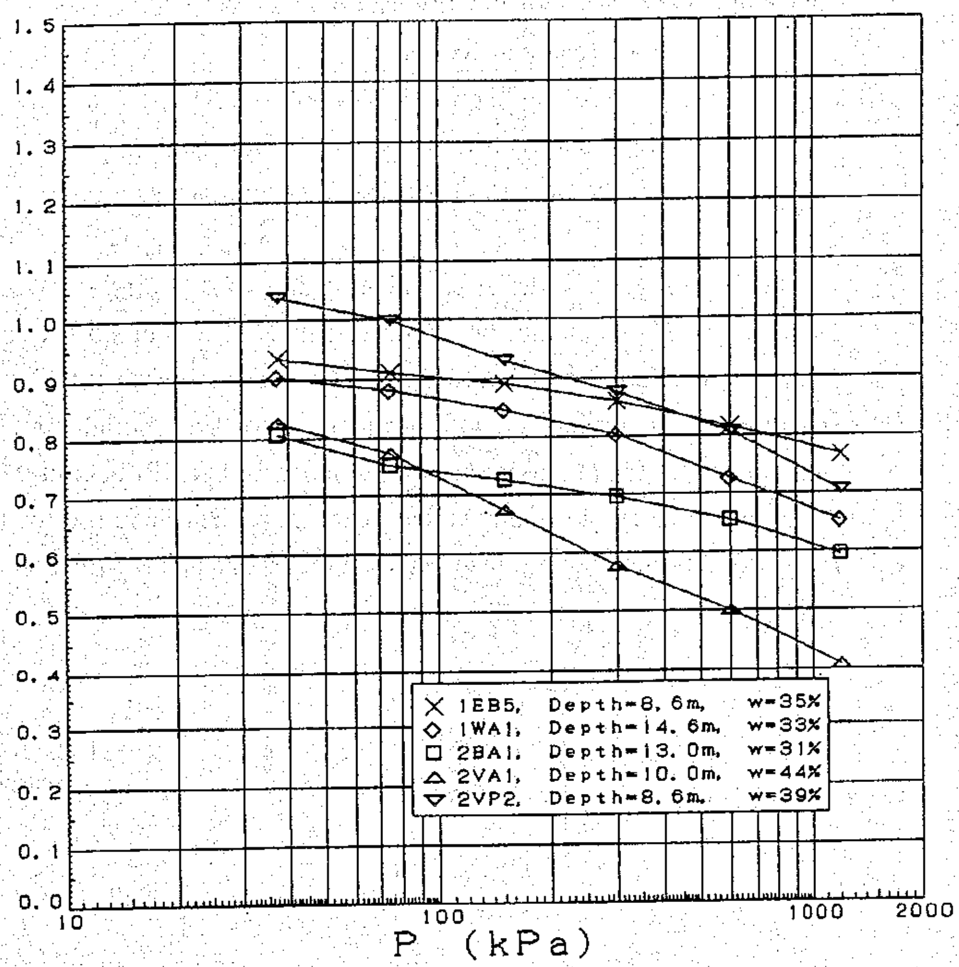


Fig. 3.6.4 e-Log P Curves for Inorganic Soils Taken from Deep Depth

(2) Time for Settlement

Time for the consolidation settlement to take place depends on the permeability of ground and the drainage of path. Most part of the estimated settlement shown in Table 3.6.1 will be developed within 5 years after completion of the filling work at the individual road section, and residual settlement at 2 years after completion of the filling work will vary from 1 to 22cm. The largest residual settlements will be expected at sections shown below ;

for main road and access road ......

17cm at Section STA 0+00 and STA 2+000

for approach roads to bridges and viaducts......

22cm at Hatia Bridge west approach section

As shown in Fig.3.6.5, the effect of preloading just behind the west abutment of the Hatia River Bridge cannot be expected fully. The preload fill cannot be extended farther to the Hatia River in order to secure the stability of the fill. Also, due to the schedule of the Project, it will be difficult to take enough time to reduce future settlement near the abutment after completion of backfill behind the abutment. It is necessary to carry out maintenance of the pavement in future for keeping a smooth riding condition at the interface between the bridge and the road embankment.

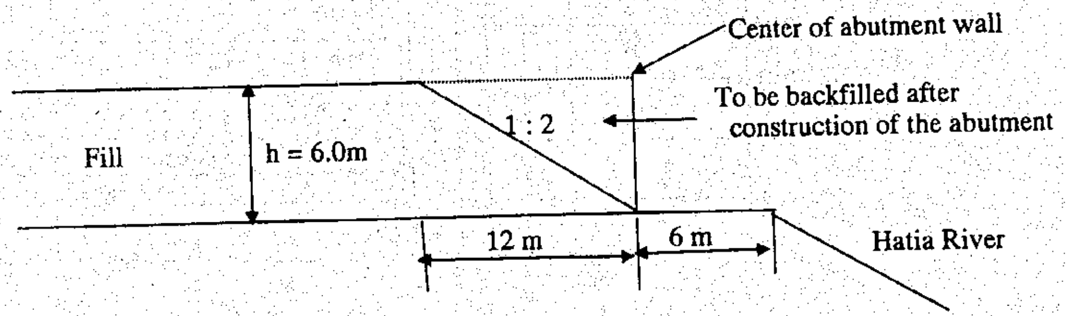


Fig. 3.6.5 Longitudinal Section of Fill before Pilling for Hatia Bridge West Approach

It is necessary to place a 50cm thick sand mat on the existing ground surface prior to placing and compaction of embankment material at the sections of main road on the west bank of the Rupsa River and at the approach sections to the bridges and viaducts. The sand mat ensures adequate drainage of pore water during consolidation of the very soft to soft silty and clayey soils.

A following equation was used to estimate time for the consolidation settlement;

$$t = d^2 T_v / C_v \qquad \cdots\cdots\cdots\cdots\cdots \text{Eq.3.6.2}$$

where t : time (day)

d : half of thickness of consolidation layer for double drainage (cm)
full thickness of consolidation layer for single drainage (cm)

C_v : vertical coefficient of consolidation (cm^2/day)

T_v : vertical consolidation time coefficient determined by Terzaghi's formula

The C_v–values shown in the ground models were determined based on the consolidation test results shown in Figures 3.6.6 thorough 3.6.8 and the ground conditions. In calculations of T_v , corrected thickness (d') was used in consideration of differences in C_v – value of each layer.
Progress of the settlement at the each section is summarized in table, Settlement VS. Time, and is presented in Appendix.

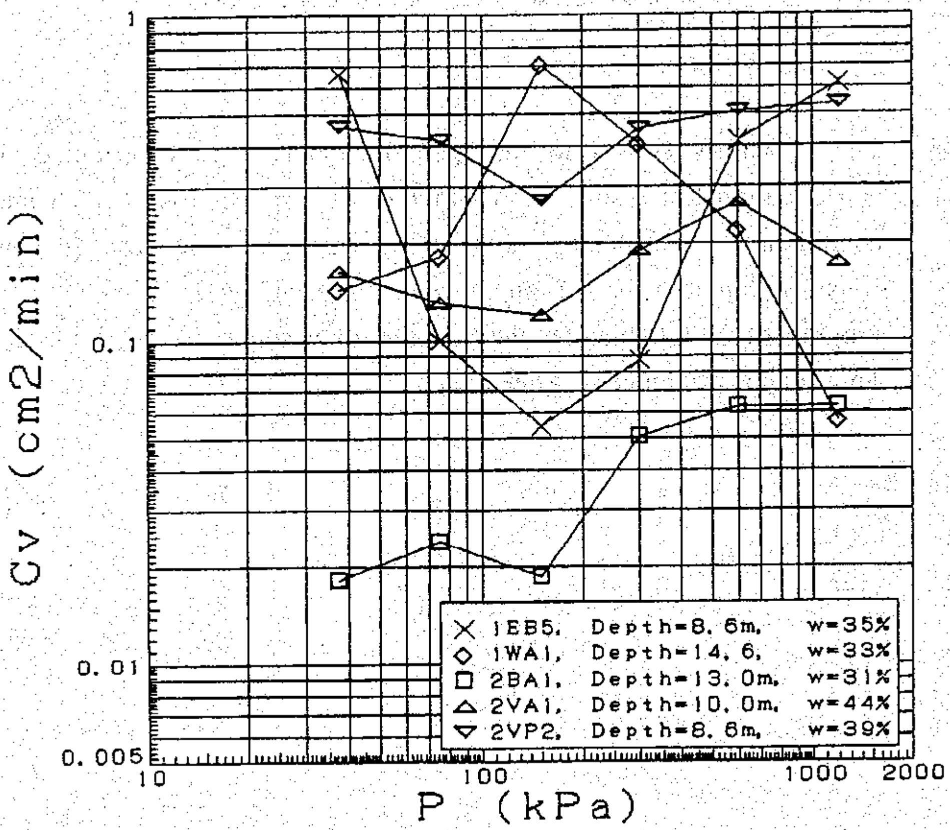


Fig. 3.6.6 log Cv - log P Curves for Soils with Organic Matter

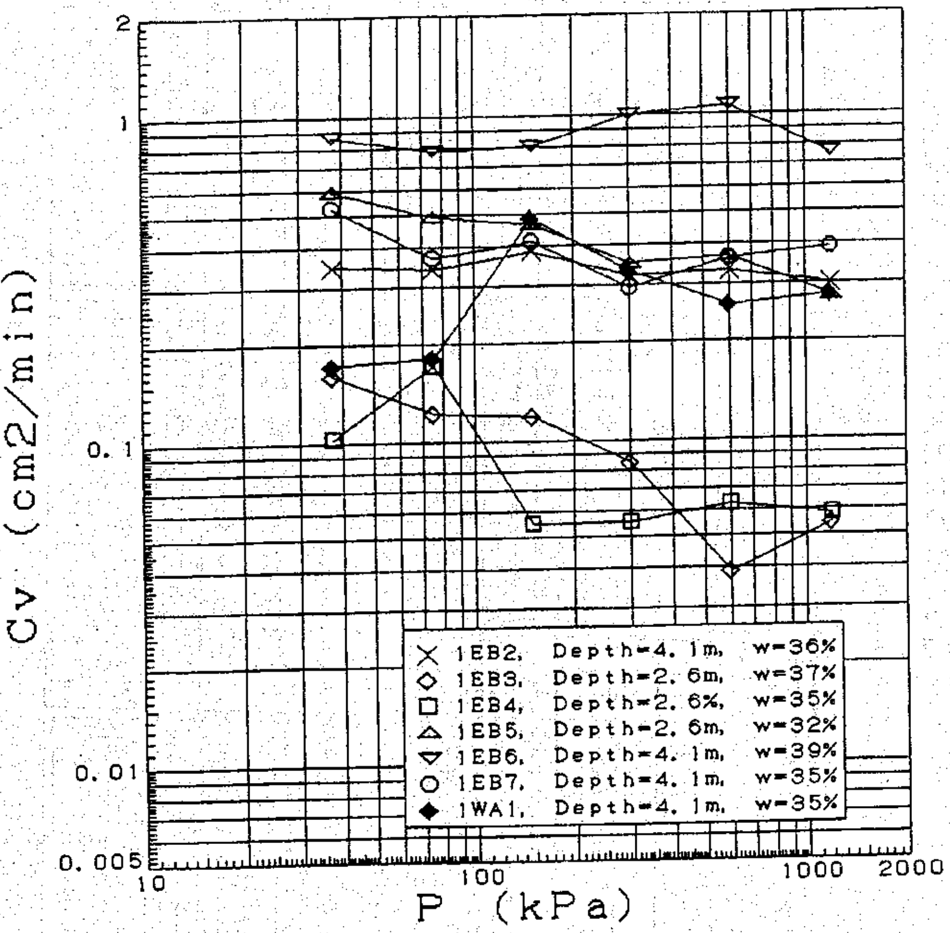


Fig. 3.6.7 log Cv - log P Curves for Inorganic Soils Taken from Shallow Depth

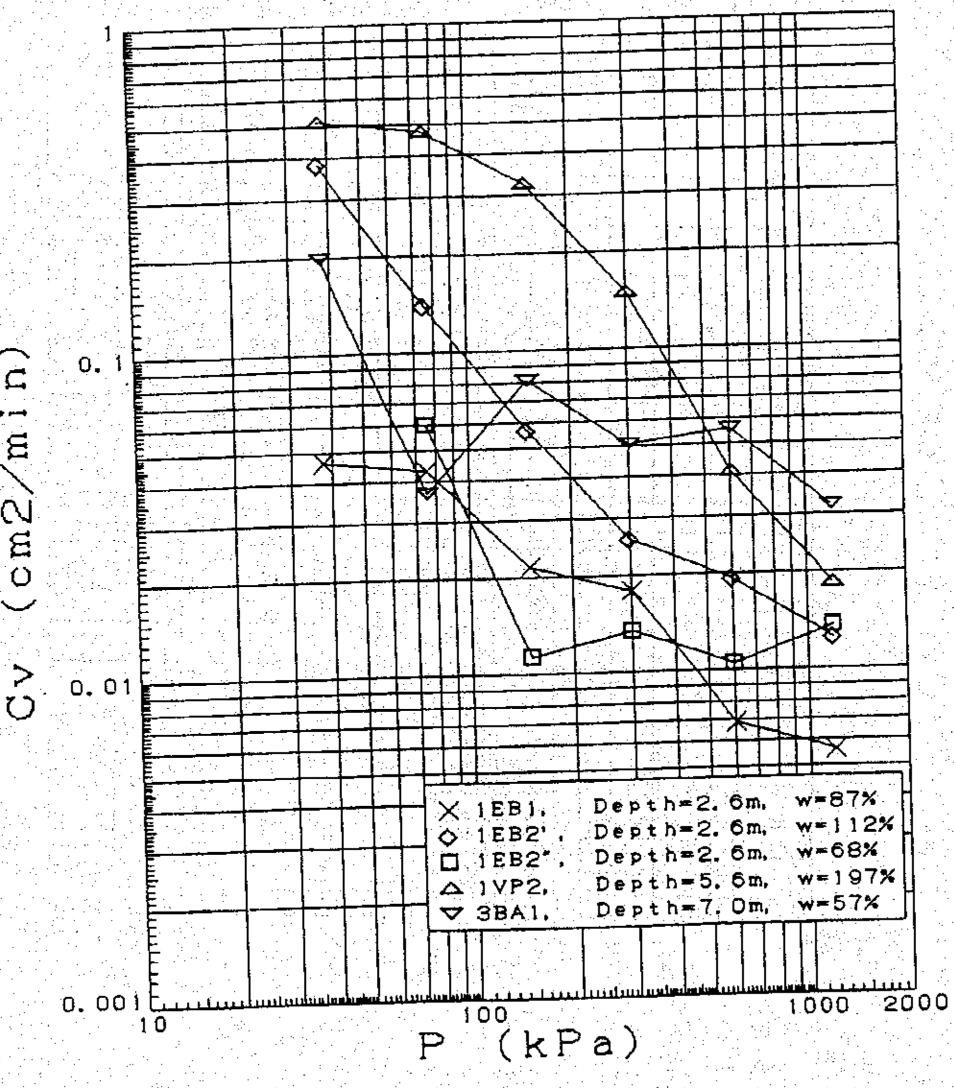


Fig. 3.6.8 log Cv –Log P Curves for Inorganic Soils Taken from Deep Depth

3.6.2 Stability of Embankment

Slope stability of embankment was analyzed using the Simplified Bishop's slip circle method with the PCSTABLE 5M software. Purdue University first developed the software in 1985. Subsequent modifications wee made to the original software that results in the development of the PCSTABLE 5M version in 1989.

(1) Stability of Embankment other than Approach Sections to Bridges and Viaducts

Estimated thickness of the embankment of the Route 1 is less than 4.5m except approach sections to the proposed bridges and viaducts. The embankment of which thickness is less than 4.5m will be constructed safely. Average filling speed should not exceed 5cm / day.
The stability analysis shows that a critical thickness of the embankment is 5m assuming that the embankment is placed on the very soft to soft soil of which untrained shear strength is 20kPa and the minimum factor of the safety (Fs) is 1.2. Excavation for side borrow pits results in instability of the embankment. In the determination of the critical thickness of the embankment, it was assumed that shoulder of the pit was located at 10m away from toe of the embankment slope and its depth was 2m.
Similar to the settlement analysis, the ground model of the stability analysis was prepared for the each section mentioned in Section 3.6.1, and presented in Figures-20 through 31 in Appendix C. The undrained shear strength of each layer was estimated based on Fig.3.6.9 (N-value VS. Undrained Shear Strength), moisture content and type of soil. Unconfined compression test results obtained in other project in Bangladesh were also referred in the estimation. Half of the unconfined compressive strength is equivalent to the undrained shear strength. Strength of the embankment was assumed as follow :

Undrained shear strength (C) : 25 kPa
Internal friction angle (ϕ) : 10°

Undrained shear strengths at different degree of consolidation were estimated from a following equation ;

$$C = Co + \Delta Pe \cdot m \cdot U / 100 \quad \cdots\cdots \text{ Eg. 3.6.3}$$

where,

C = undrained shear strength at a certain degree of consolidation (kPa)
Co = initial undrained shear strength (kPa)

$\triangle Pe$ = effective stress in the ground …… assumed to be (embankment load x influence factor-30) (kPa)

m = rate of strength gain

U = degree of consolidation (%)

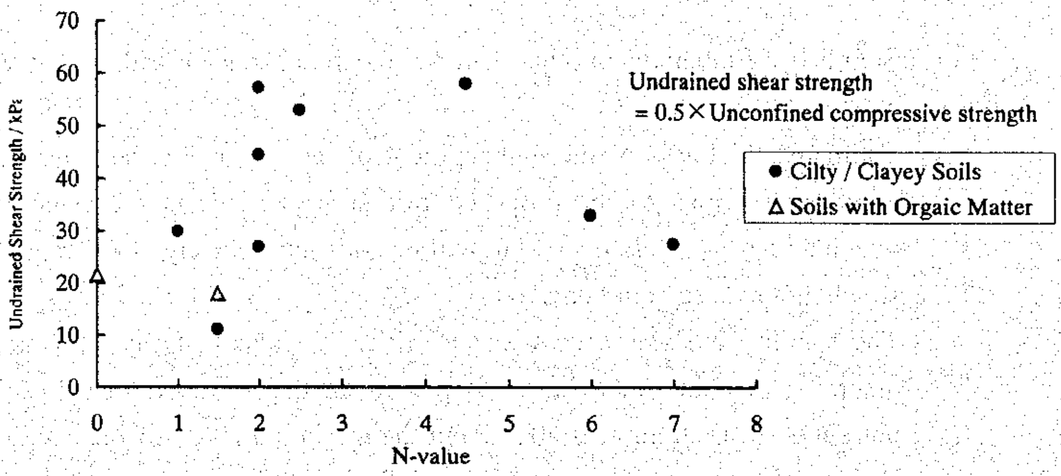


Fig. 3.6.9 N-value VS. Undrained Shear Strength

(2) Stability of Embankment, Approach Sections to Bridges and Viaducts.

The embankment at the approach sections to the proposed bridges and viaducts should be constructed by a slow embankment method with or without counter berm in order to secure stability of the embankment.

Filling speed, length of curing periods, timing of commencement of the pilling, and rate of back fill behind the abutment should be decided based on monitoring results in order to avoid instability of the embankment. Because, in the slow embankment method, the stability of embankment depends on increase of strength of the weak soil by consolidation. If filling speed is too fast, enough strength gain cannot be expected, and this results in failure of the embankment. Monitoring program should consist of following items;

Settlement of embankment :

to be monitored by settlement plates installed at ground surface along the center line and both sides of crest of the embankment.

Displacement of ground :

to be monitored by displacement pegs installed around the embankment

Movement of abutment :

to be monitored by point marked / or installed on abutment

Table 3.6.2 summarizes the stability of the embankment. The ground models used for the stability analysis are presented in Figures-32 through36 in Appendix C. Additional soil investigation may be required when instability of the embankment is detected by the monitoring and / or in order to judge the timing of removal of a part of the fill for the piling preparation.

Table 3.6.2 Summary of Stability of Embankment

Section	Factory of Safety	Countermesure
Hatia Bridge West Approach	1.28	Slow embankment method + Geotextile + Sand mat + Monitoring
Hatia Bridge East Approach	1.20	Slow embankment method + Sand mat + Monitoring
West Approach to Viaduct	1.24	Slow embankment method + Sand mat + Counter berm* + Monitoring
East Approach to Viaduct	1.25	Slow embankment method, Sand mat + Counter berm* + Monitoring
Molonghata Bridge Approaches	1.39	Sand mat + Monitoring

* The counter berm is required only for longitudinal direction of the road.

(a) Hatia Bridge West Approach

The stability of the embankment toward the river is more critical than the transverse direction. The 6.0m thick embankment will be able to be constructed in 12 months, and should be maintained for 11 months for the curing before commencement of preparation for piling at the abutment as shown in Fig. 3.6.10. The figure is an indication only. More stages and longer curing periods may be required. Actual filling speed, length of curing periods, timing of commencement of the pilling, and rate of backfill behind the abutment should be decided based on monitoring results.

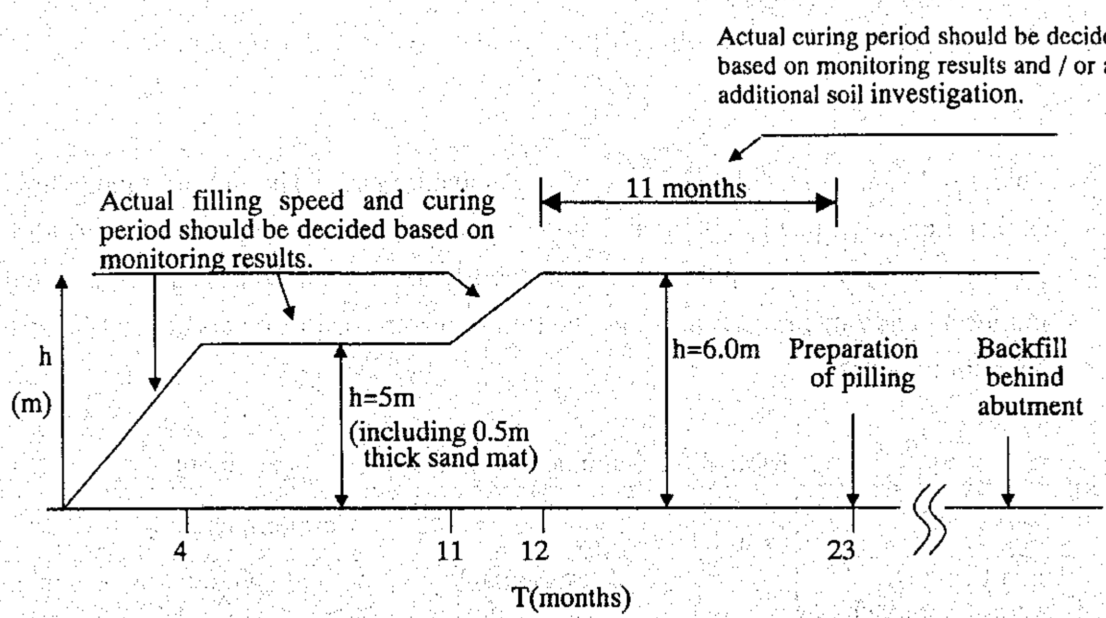


Fig.3.6.10 Filling Schedule for Hatia Bridge West Approach

Figures 3.6.8 and 3.6.11 show longitudinal sections of the fill before and at the pilling and abutment construction, respectively. As shown in Fig.3.6.11, one meter thick fill should be remained for protection of geotextile placed under the sand mat.

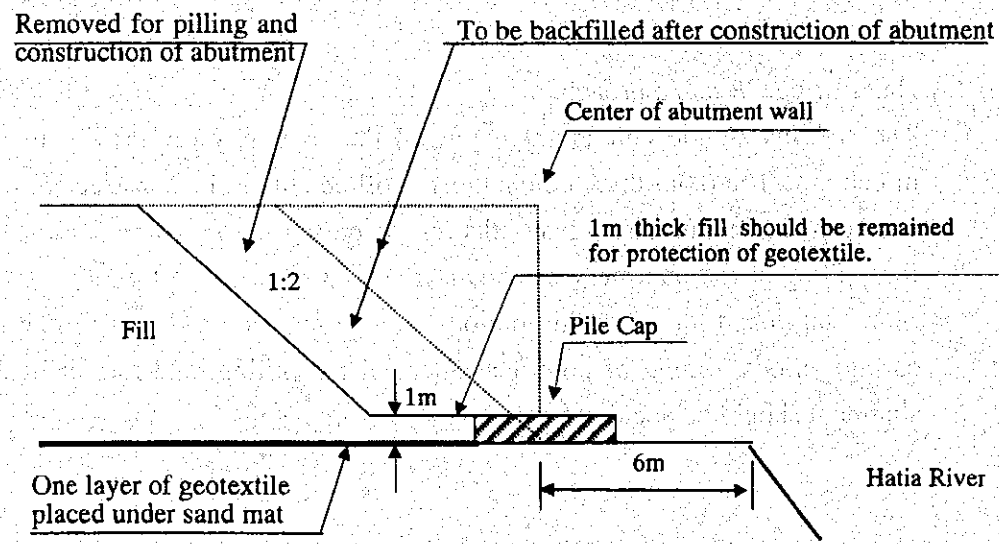


Fig. 3.6.11 Longitudinal Section of Fill at Pilling and Abutment Construction for Hatia Bridge West Approach

(b) Hatia Bridge East Approach

Same to the west bank of the Hatia River, stability of the embankment toward the river is more critical than the transverse direction. The 5.8m thick embankment will be able to be constructed in 7 months, and should be maintained for 5 months for the curing before commencement of preparation for piling at the abutment as shown in Fig. 3.6.12. The figure is an indication only. More stages and longer coring periods may be required. Actual filling speed, length of curing periods, timing of commencement of pilling, and rate of backfill behind the abutment should be decided based on monitoring results. A 0.5m thick sand mat should be placed on the ground surface to improve the rate of strength gain in the weak ground.

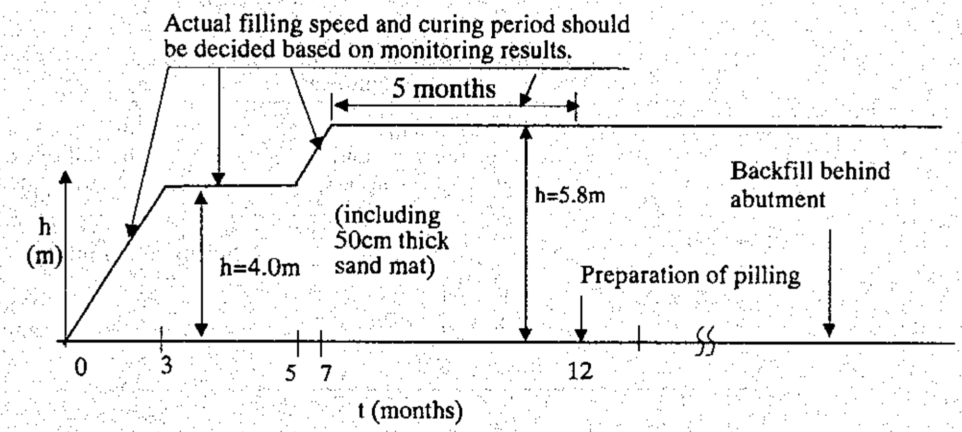


Fig. 3.6.12 Filling Schedule for Approach Section to Hatia Bridge East Approach

Fig.3.6.13 shows a longitudinal section of the embankment before the preparation piling.

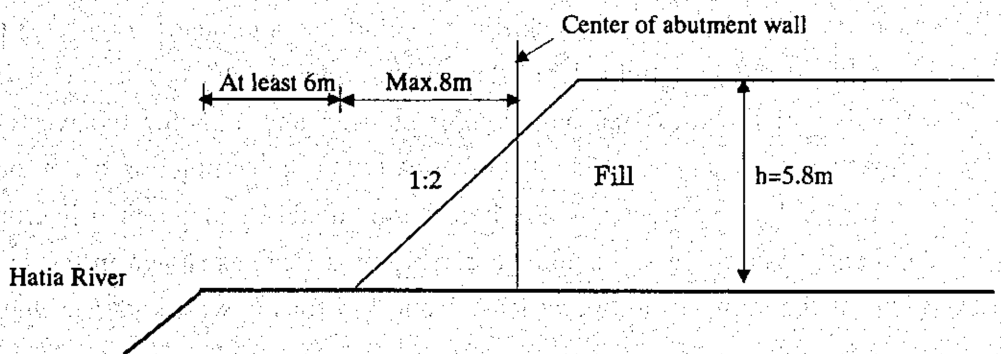


Fig. 3.6.13 Longitudinal Section of Fill before Preparation of Pilling for Hatia Bridge East Approach

(c)West and East Approaches to Viaduct

The 6.5m thick embankment will be able to be constructed in 6 months, and should be maintained for 4 months (one rainy season) for the curing before commencement of preparation of piling. Fig.3.6.14 shows a filling schedule. The figure is an indication only. The stage construction may be required instead of a steady increment of embankment load shown in Fig.3.6.14. Actual filling speed, length of curing periods, timing of commencement of preparation of the piling, and rate of backfill behind the abutment should be decided based on the monitoring results. A 0.5m thick sand mat should be placed on the ground surface to improve the rate of strength gain in the weak ground.

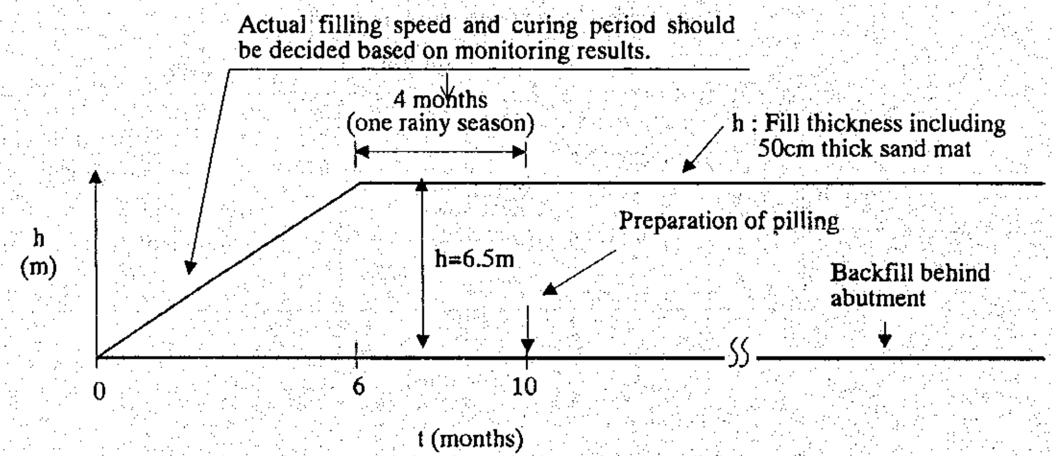


Fig. 3.6.14 Filling Schedule for West and East Approaches to Viaducts

No berms are required for the transverse direction of the embankment. However, the preload fill and counter berm should be placed in front of the abutment location to improve the soft ground around the abutment, and secure the stability of the embankment at the stage of backfilling behind the abutment. Figures 3.6.15 and 3.6.16 show the longitudinal profiles of the embankment. The factor of safety when the fill thickness reaches to 6.5m is 1.24 and 1.25 for the west and east approaches, respectively. The factor of safety at the completion of the backfill is 1.33 for the both sections. A 2m high berm should be maintained in front of the abutment at the piling and abutment construction stage as shown in the Fig.3.6.16. Width of the berm should be at least 5 and 4m for the west and east approaches, respectively.

Placement of sand mat at the ground surface is required to improve the rate of strength gain in the weak ground.

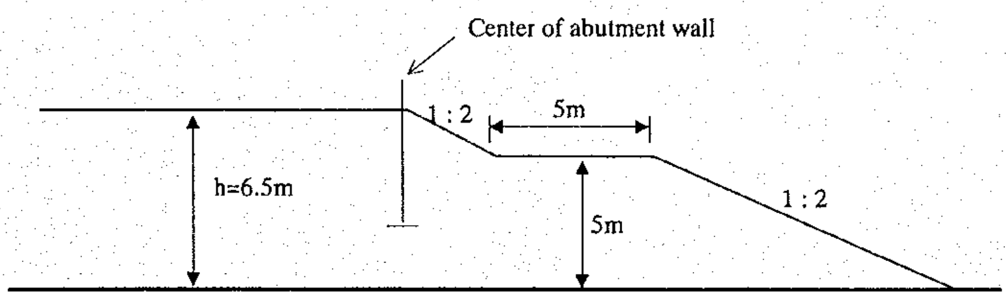


Fig. 3.6.15 Longitudinal Section of Fill before Preparation of Pilling for West and East Approaches to Viaducts

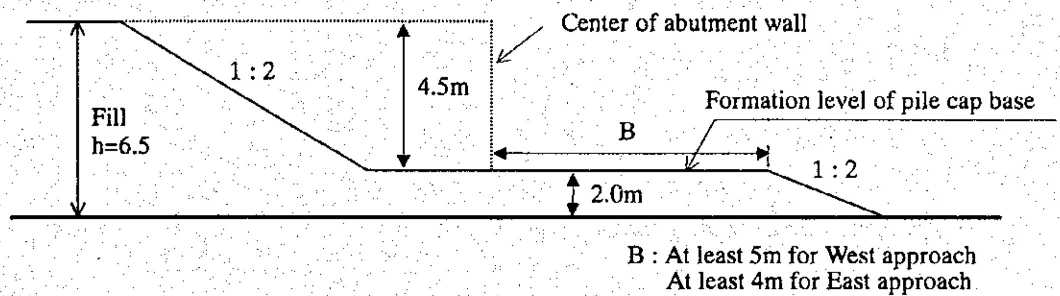


Fig. 3.6.16 Longitudinal Section of Fill at Pilling and Abutment Construction Stage for West and East Approaches to Viaducts

(d)Molonghata Bridge Approaches

The stability analysis shows that a 4.8m thick embankment will be able to be placed safely. The average filling speed should be less than 5cm / day, and should be adjusted based on the monitoring results.

The minimum factor of the safety at this section is 1.39. The ground conditions of this section is better than the other approach sections. The weak zone in the ground at the Molonghata bridge section is thinner than the other approach road sections, and a hard crust of which undrained shear strength is 50 kPa covers this section. However, same to the other approach sections, the piling should be carried out after placement of the fill in order to avoid possible damage to the installed piles by the filling operation.