6.2.4 Study in Terms of Geotechnical Engineering

(1) Study on the Road Banking Material

A proposed place for borrow pit near the route produces the quality banking material because sandy to gravel laterites are distributed.

Each of ten pieces of disturbed samples (CD01 – CD10) taken from the candidate place for borrow pit was compacted to 95% of maximum dry density with the optimum water content and soaked in water for 96 hours. Then, the CBR test was made, and the design CBR was determined to be 6 (average 12.1 minus standard deviation 5.3 = 6.8) on the basis of test result.

The field CBR test and laboratory CBR test with compacted undisturbed/disturbed samples were made for the base course/subgrade of existing national roads (A1, A2, A3, A4, A8, B84, A110, B214, B216). A1, A2, and A3 national roads showed CBR of 6 or more. For A4 national road, the field CBR was 3.7, CBR with undisturbed samples was 5.25, and CBR with disturbed samples was 6.6. In the case of A8 national road, the field CBR was 11.4, CBR with undisturbed samples was 4.4, and CBR with disturbed samples was 21.4. Considering that the data of existing national roads show such degree of variation, the design CBR of 5 is considered rational and proposed for this outline design.

Table 6.1. Results of Labo-CBR tests for Proctor 95% compaction of disturbed soil from borrow pits and Field CBR tests for Subgrade at the shoulder of existing National Roads

Borrow pit location	CBR Value
CD 04	6.4
CD 01	6.925
CD 09	8.425
CD 06	9.875
CD 07	10.25
CD 03	12.05
CD 05	14.375
CD 10	16.75
CD 08	24.05
Average (a)	12.12
Standard deviation (b)	5.28
(a) – (b)	6.84

Field CBR location	Field CBR Value	Proctor CBR Value	UD CBR Value
CF-8 (A4)	3.66	6.625	5.25
CF-10 (B84)	7.72	3.685	8.75
CF-6 (A110)	8.2	21	9.75
CF-4 (A1)	9.3	24	6.25
CF-12 (A8)	11.36	21.43	4.395
CF-2 (A3)	12.2	6.075	24.35
CF-14 (A2)	16.8	7.7	41.75
CF-16 (B216)	18.9	3.48	36.375
(a)	11.02	11.75	17.11
(b)	4.65	8.20	14.03
(a) – (b)	6.37	3.55	3.08

Design CBR	5

(2) Road Fill for the Soft Ground

For the high road fill of 3 m or more in the optimum route, there are geotechnical engineering problems as follows:

1) Ordinary Lowland

Soft flood plain deposit distributed in paddies, streams, and low marsh in the dentritic water system occupying most of routes is extremely uneven and complicated in distribution. For the depth of several meters from the ground surface, deposition of extremely loose silt-mixed sandy soil with the N value of about 0 – 1 and soft sand-mixed clayey soil is often observed. From the result of this field survey boring, standard penetration test, portable cone penetration test, and sampling from the ground surface, the thickness of soft layer with the N value of 0 – 1 or less is estimated to be 1 – 2 m or less on average. It is advised to replace this soft layer with banking material according to the displacement method during construction of fill. Note that this boring survey (BH-03, BH-08, BH-09) and portable cone penetration test (M-02, M-03, M-04, M-05, and M-06) did not show any soft layer with possible consolidation settlement below the surface soft layer. Consolidation settlement of road fill is considered to be negligibly small after replacement of soft layer directly below the ground surface.

For the area around Bolgoda River and Lake to the south of the route and the area to the north of Kelani River, this geotechnical investigation result showed no excessively soft layer. However, the existing data shows that there are certain locations where the soft peat layer is buried in a drowned valley or old backland marsh. It is therefore advised to confirm the situation in the future by the detailed geotechnical investigation.

2) Swamp Near the Northern STA24

From this field survey and analysis of existing data, it was confirmed that the thick soft layer (in particular, the peaty soft ground where substantial consolidation settlement is expected) exists near northern STA24 (Nawanmehara Marsh Area) where boring BH-02 was made. Due consideration must be paid during design and construction of the road filling in this area.

Followings are the description of the ground condition and countermeasures.

a) Boring at BH-02 and laboratory soil test results showed that the extremely soft peat and organic clay layer exists to the thickness of about 8 m. For this Nawanmehara Marsh, the soil investigation was made previously when there was a reclamation plan. According to the existing data obtained from above investigation, the peat layer thickness varies within a range of 2 – 8 m in this marsh. Boring BH-02 made this time was at a point where the soft layer is the thickest, as determined from the previous study on the past data. Fig. 6.2.6 shows thickness contours of soft layers (Pt for peat, Oc for organic clayey soil) extracted from existing data (19 mechanical borings and 15 hand auger borings) by assuming the thickness of soft layer of the island (DUWA) in the middle of marsh, island (ILUKGODA) to the south-west, and northern boundary with the Navy facilities as zero.

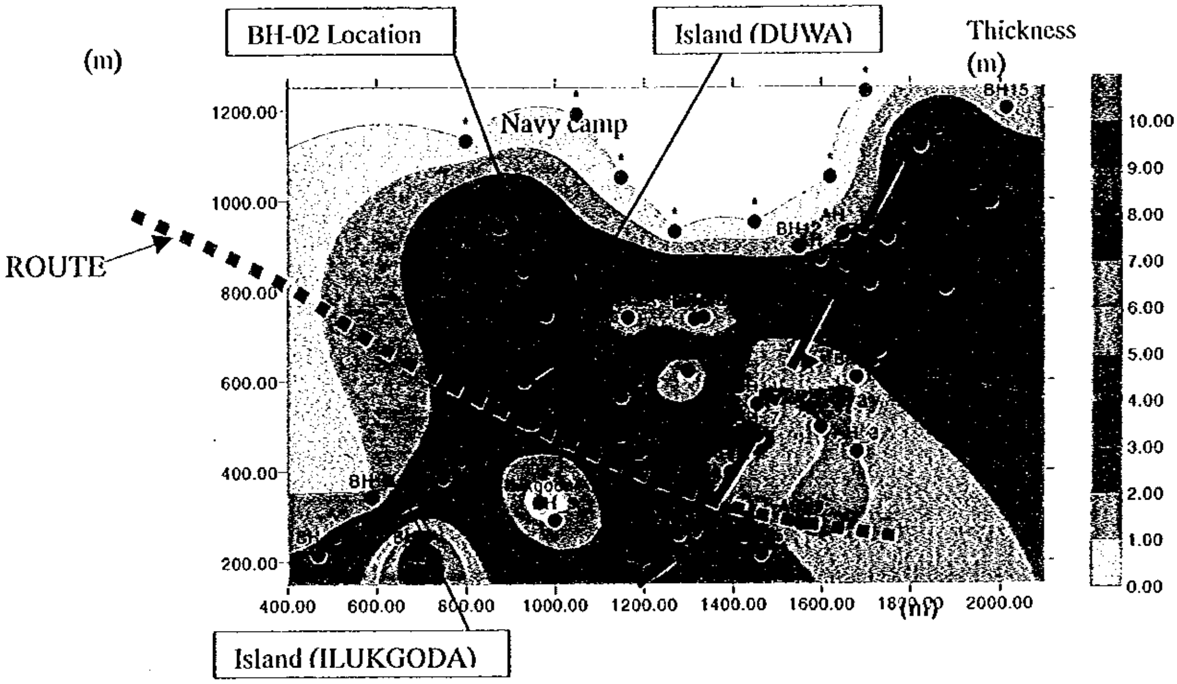
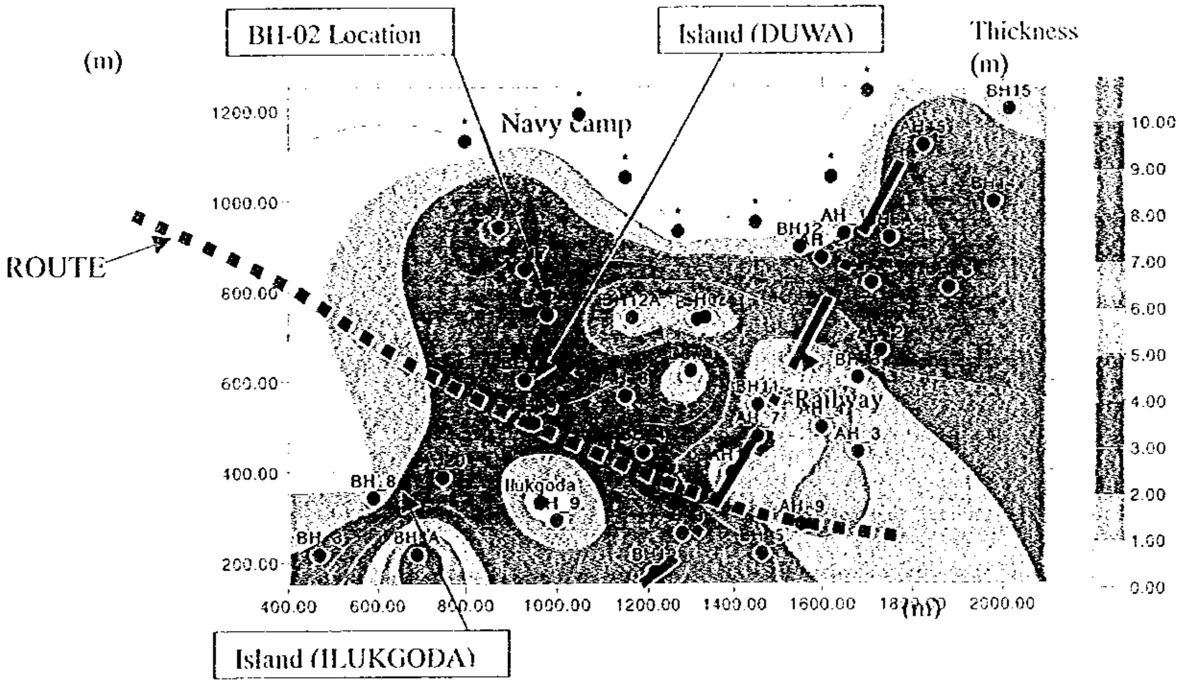


Fig – 6.2.6 Estimation for the thickness of Peat and Organic Clay from existing data at Nawanmehara Marsh (in meters, arbitrative X-Y coordinate, existing data from "WELISARA Landfill – Geotechnical Investigation 1994")

b) The detailed design study by JICA (1992) for old Katunayake highway, in which the south-north route was to run along the railway to the east of the marsh involved several ten borings and detailed soil survey. Fig. 6.2.7 summarizes the consolidation characteristics (e-log P) of peaty/organic clay at BH-02. The figure also shows data extracted for Horape and Ragama areas from existing Katunayake data by using broken lines.

a) Boring at BH-02 and laboratory soil test results showed that the extremely soft peat and organic clay layer exists to the thickness of about 8 m. For this Nawanmehara Marsh, the soil investigation was made previously when there was a reclamation plan. According to the existing data obtained from above investigation, the peat layer thickness varies within a range of 2 - 8 m in this marsh. Boring BH-02 made this time was at a point where the soft layer is the thickest, as determined from the previous study on the past data. Fig. 6.2.6 shows thickness contours of soft layers (Pt for peat, Oc for organic clayey soil) extracted from existing data (19 mechanical borings and 15 hand auger borings) by assuming the thickness of soft layer of the island (DUWA) in the middle of marsh, island (ILUKGODA) to the south-west, and northern boundary with the Navy facilities as zero.



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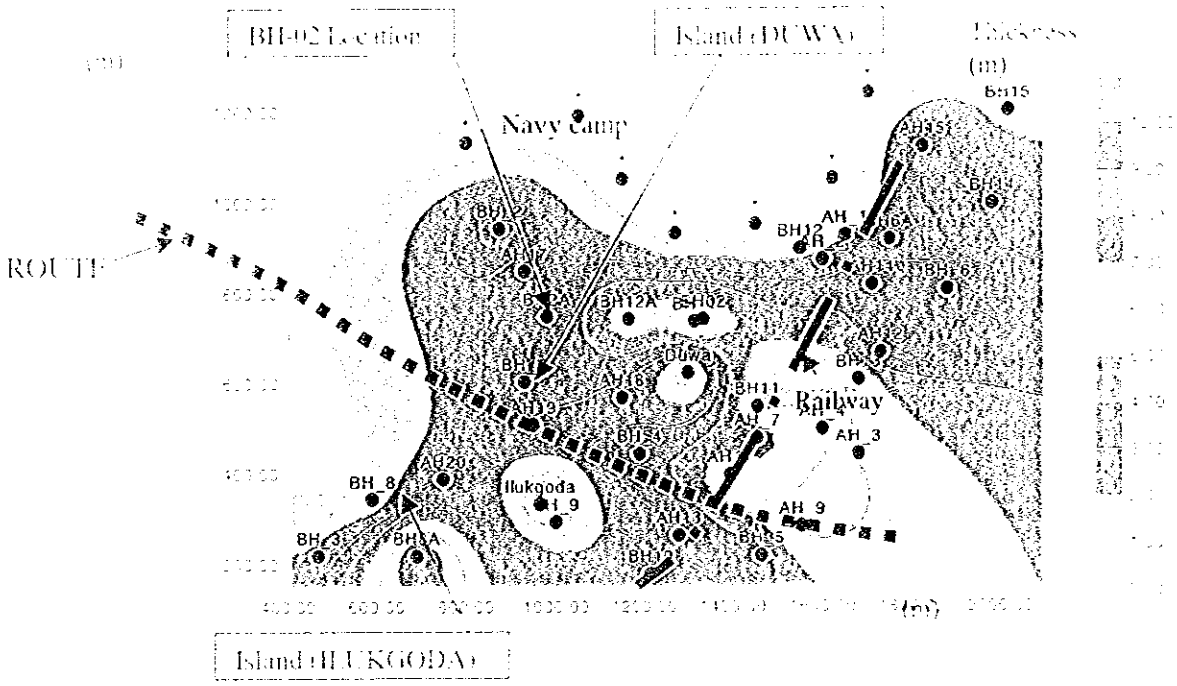


Fig – 6.2.6 Estimation for the thickness of Peat and Organic Clay from existing data at Nawanmehara Marsh (in meters, arbitrative X-Y coordinate, existing data from "WELISARA Landfill – Geotechnical Investigation 1994")

b) The detailed design study by JICA (1992) for old Katunayake highway, in which the south-north route was to run along the railway to the east of the marsh involved several ten borings and detailed soil survey. Fig. 6.2.7 summarizes the consolidation characteristics (e-log P) of peaty organic clay at BH-02. The figure also shows data extracted for Horape and Ragama areas from existing Katunayake data by using broken lines.

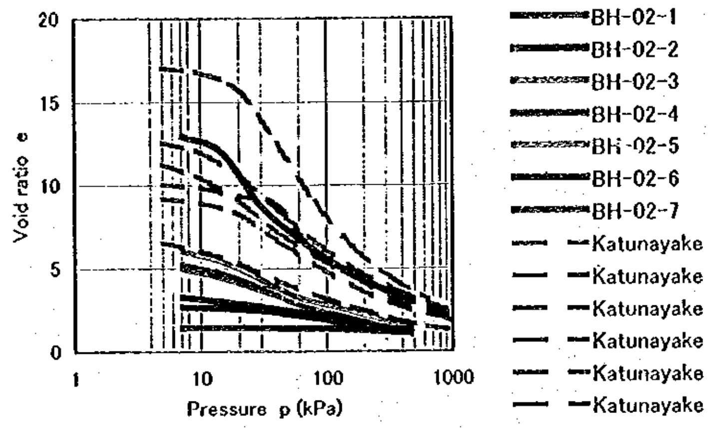


Fig. 6.2.7 e – logP Curve of Alluvial Peat (Ap) at BH-02 and existing data of Katunayake(1992) at Horape – Ragama Area

c) As a result of the laboratory test conducted on the undisturbed samples from BH-02, extremely soft blackish organic clay with the natural water content of 150 – 300% was predominant, except for peat with the natural water content of 550% at the depth of 2 m (BH-02-2). It was known also that the clay mixed with organics with the natural water content of 50 – 100% existed to the depth of 6.3 – 8.3 m under the soft layer. No intervening thin sand layer, etc. could be observed. When compared with the neighboring peat area (old Katunayake highway), peat distributed in this marsh was predominantly organic clay and its consolidation settlement is considered to take more time relative to the peat with high water content.

d) As is estimated from the result of unconfined compression test of BH-02, the undrained strength of the soil is 2.5 – 3.8 kN/m^2 in Cu. In comparison with the existing data of the old Katunayake HW (average of data of the entire section; the peat Ap is Cu = 10 kN/m^2 and organic clay Ao = 15 kN/m^2), the soil here is considerably soft. Though mere comparison between places differing in deposit environment can not be made, this Nawanmehara Marsh is a virgin ground and the ground is characterized by its extreme softness.

e) Judging from the critical bearing capacity of qd = 3.6 Cu = 9 – 14 kN/m^2 of the ground for fill, the critical fill height, Hec becomes Hec = 9 to 14 / 19 = 0.47 – 0.74 m when the unit weight of fill is = 19 kN/m^3. In this situation, the fill may be immediately buried under the ground even if sand mat of about 0.5 m thick is installed.

f) In Sri Lanka, replacement and pre-loading methods have proved successful as a soft ground improvement process. But methods requiring special soil improvement machinery, such as sand compaction pile method or deep mixing method of soil stabilization, have not been applied, and application of such methods requires import of the machinery from the outside. Besides, materials used for ground improvement (quality sand or cement) are expensive. In this way, the applicable soil improvement method is restricted because of the cost.

g) In Sri Lanka, the displacement method or excavation replacement method has often been applied to ordinary roads (e.g., the Japan – Sri Lanka Friendship Road near the National Assembly Hall) that pass through the peat area. As shown in Fig. 6.2.6, the soft layer thickness tends to be thin, with localized thickening, around the marsh. If the thorough soil investigation is made and the route selected with care, it may be possible to bypass the thick soft layer area. Besides, the length of route passing through the soft layer is about 1 – 2 km. Therefore, if replacement is to be made for the peat layer within 4 – 5 m, the replacement method will be the most economic and practical soil improvement method and thus recommended. For replacement, gravely laterite used for road fill may be used. In the case of excavation replacement method, additional application of sheet pile wall or a dewatering method will reduce settlement in the future while enabling satisfactory construction work. On the other hand, the displacement method will suffer deterioration of the work accuracy when the replacement thickness exceeds 1 – 3 m. Namely, more or less residual settlement may occur.

h) If there is enough time allowance for the work, the pre-loading method may be applied. But this method suffers in terms of residual secondary consolidation settlement in the future and associated necessity of road maintenance, so that the replacement method is considered to be more practical.

3) Lowland Marsh Near Abutment of South STA443

Near the eastern abutment BH-10 at a Bolgoda River crossing point, the base rock is covered with soft organic clayey soil with N = 0 to the depth of 0 – 3 m, alluvial clayey soil with N = 2 – 4 to the depth of 3 – 5.8 m, and soft residual clay (completely weathered base rock) with N = 0 – 4 to the depth of 5.8 – 9.5 m. On the other hand, at BH-11 only 27 m to the west, the base rock appears at a depth shallower by about 4 m and there is no residual clay distribution. Since the geological distribution around here is extremely complicated with the base depth varying heavily, detailed soil investigation in the future is indispensable for rational design.

When the 3 – 4 m high fill is to be made on the backside of eastern abutment, the use of pile foundation for abutment may possibly result in substantial displacement of abutment due to lateral flow of the ground. Reliable way is to apply the caisson foundation that allows bearing of the abutment directly on the base rock and to replace the backside soil. It is also recommended to replace the backside soil. Though depending on the ground condition of the marsh behind the abutment (up to the southern connection interchange on the east side), pile foundation may be employed if the lightweight fill method is planned for the backside of the abutment, or bridge structure may be planned passing through this marshy section. For the sand compaction pile method or deep mixing method of soil stabilization that requires the dedicated soil improvement machinery as discussed in above (2), c), the first thing to do is construction of a access road to carry such machinery into the marsh.

Assuming that the ground condition behind the abutment is similar to the BH-10, substantial settlement after fill may be due mainly to organic clayey soil at the depth of 0 – 3 m. It is advised therefore to apply the displacement method to such organic clayey soil. When the fill height is 3 – 4 m, filling is considered difficult because of small shearing strength of residual clay at the depth of 5.8 – 9.5 m unless the width of the counterweight fill or slope of filling is substantially large.

Distribution and faces of weathered soil on top of bedrock are complicated, and it is desirable to carry out detailed geotechnical investigation.

(3) Stability of the Road Fill for the Soft Ground

Stability calculation was conducted by circular arc method for original ground and improved ground by replacement method at above-mentioned 6.2.4 (2) 2) Swamp near the northern STA24. Fig.6.2.9 – 6.2.25 show the conditions of analysis and the results of calculations. The conditions and results were summarized in Table 6.2.1. These results mean as follows;

Original ground	:	Maximum height of filling must be less than 1.5 m
3m of filling height	:	Replacement thickness must be 6 m

4 - 6m of filling height : Replacement thickness must be more than 7 m

7 - 9m of filling height : All of soft layers have to be replaced

Table 6.2.1 Summary of stability calculations for road fill on very soft ground

Ground condition	Filling height (m)	Applied Replacement method			Safety factor Fs	Reference Figure No
		Thickness of replacement (m)	Average width of replacement (m)	Replacement quantities (m^3/m)		
Original ground	1				1.437	6.2.9
	1.5				1.001	6.2.10
	2				0.776	6.2.11
	3				0.557	6.2.12
Soil improvement Replacement method or Displacement method	3	1	B+28.8	1x(B+28.8)	0.559	6.2.13
	3	2	B+27.8	2x(B+27.8)	0.594	6.2.14
	3	3	B+26.8	3x(B+26.8)	0.629	6.2.15
	3	4	B+31.8	4x(B+31.8)	0.792	6.2.16
	3	5	B+30.8	5x(B+30.8)	1.015	6.2.17
	3	6	B+30.8	6x(B+30.8)	1.171	6.2.18
	4	6	B+34.4	6x(B+34.4)	1.052	6.2.19
	5	6	B+38.0	6x(B+38.0)	0.959	6.2.20
	5	7	B+32.0	7x(B+32.0)	1.205	6.2.21
	6	7	B+35.6	7x(B+35.6)	1.205	6.2.22
	7	7	B+39.2	7x(B+39.2)	1.059	6.2.23
	8	7	B+42.8	7x(B+42.8)	0.994	6.2.24
	9	7	B+46.4	7x(B+46.4)	0.976	6.2.25

Note: B = Roadway width (m)

Although, it is difficult to fill up to 1.5 meters without soil improvement, consolidation settlement was calculated for the original ground condition using the test results showed in Fig.6.2.7 for the references. The calculated consolidation settlements for center of the road fill are shown in Fig.6.2.27 and Fig.6.2.28

Case 1: Initial ground condition

Height of road fill	H	= 1 (m)
Slope angle of road fill	V:H	= 1:1.8
Minimum safety factor	Fs min	= 1.437
Center of circular arc	X	= 0.00 (m)
	Y	= 5.00 (m)
Radius	R	=11.20 (m)
Moment of resistance	Mr	=1419.7 (kN·m)
Moment of slide	Mo	= 988.0 (kN·m)

No of layer	Saturated unit weight (kN/m³)	Wet unit weight (kN/m³)	Internal friction angle (degree)	Cohesion (kN/m²)
1	10.8	10.8	0	3
2	12.1	12.1	0	5
3	15.8	15.8	0	11
4	19	19	35	100
5	19	19	35	1
6	18	18	25	0

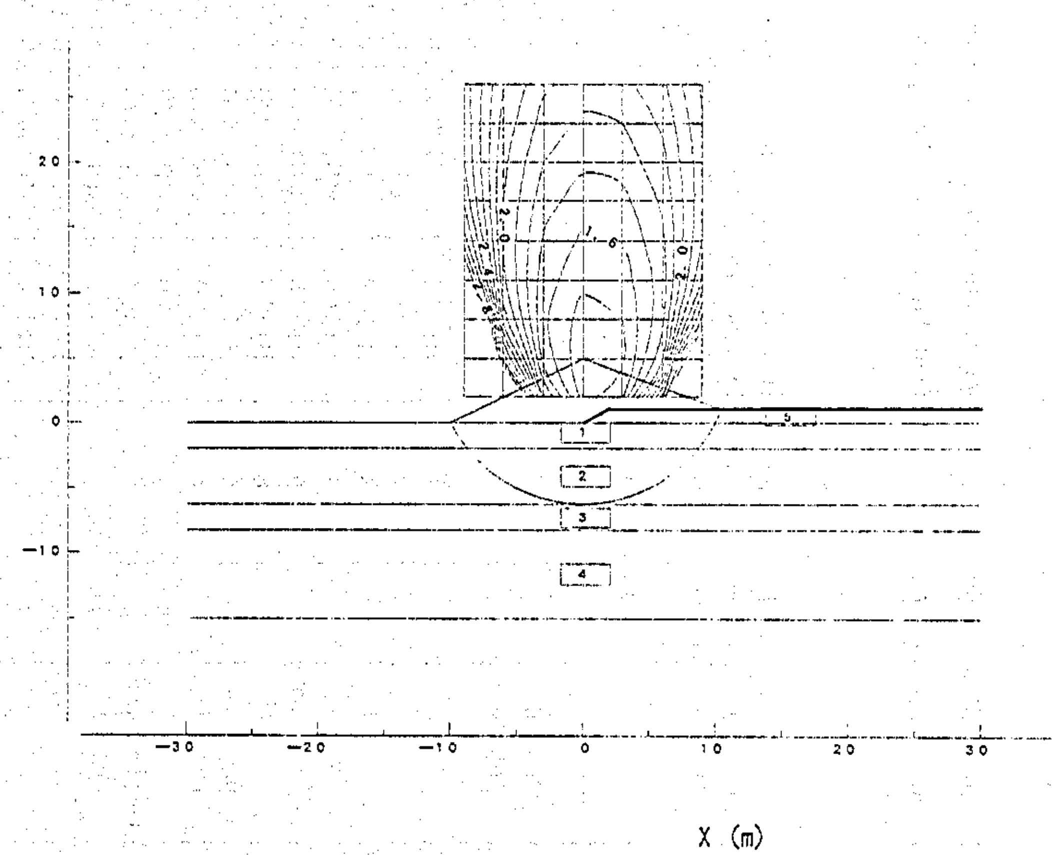


Fig. 6.2.9 Minimum safety factor circular arc and equi-safty factor diagram

Case 2: Initial ground condition

Height of road fill	H	= 1.5 (m)
Slope angle of road fill	V:H	= 1:1.8
Minimum safety factor	Fs min	= 1.001
Center of circular arc	X	= 0.00 (m)
	Y	= 5.00 (m)
Radius	R	=11.20 (m)
Moment of resistance	Mr	=1525.7 (kN·m)
Moment of slide	Mo	=1524.6 (kN·m)

No of layer	Saturated unit weight (kN/m³)	Wet unit weight (kN/m³)	Internal friction angle (degree)	Cohesion (kN/m²)
1	10.8	10.8	0	3
2	12.1	12.1	0	5
3	15.8	15.8	0	11
4	19	19	35	100
5	19	19	35	1

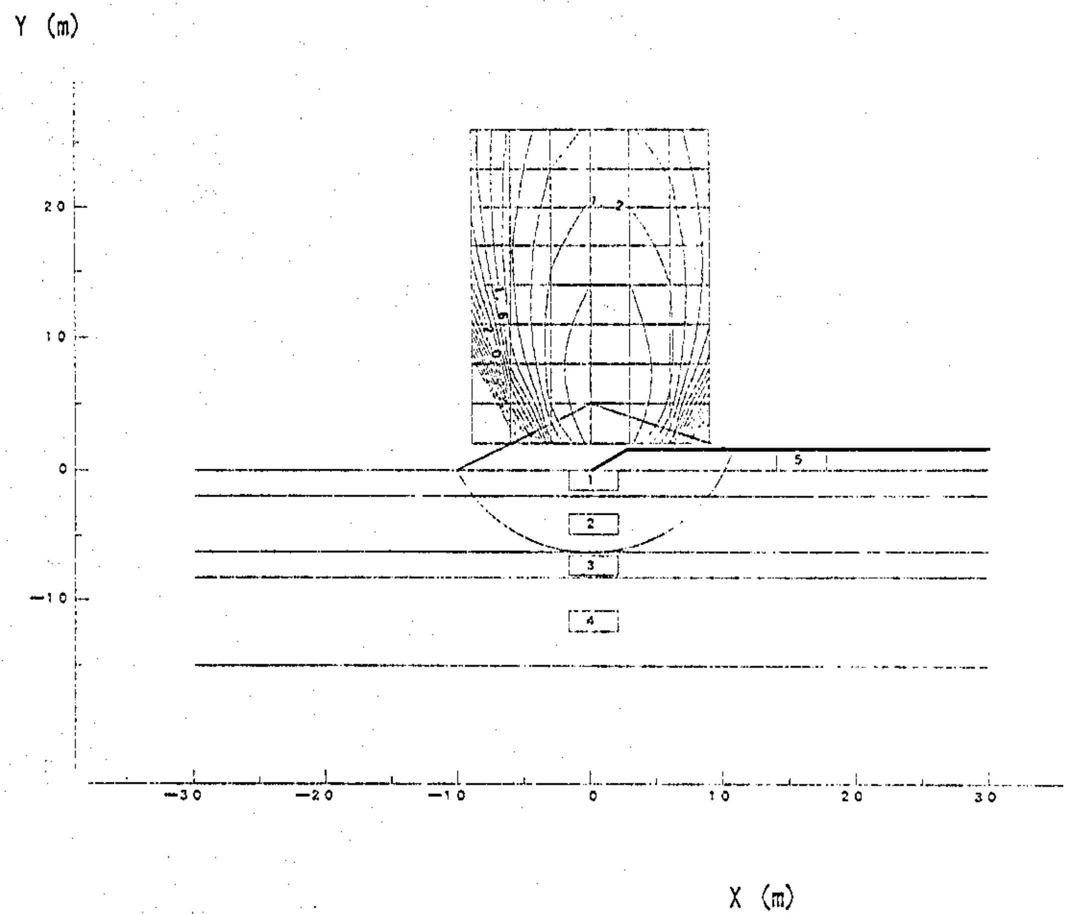


Fig.6.2.10 Minimum safety factor circular arc and equi-safty factor diagram

Case 3: Initial ground condition

Height of road fill	H	= 2 (m)
Slope angle of road fill	V:H	= 1:1.8
Minimum safety factor	Fs min	= 0.776
Center of circular arc	X	= 3.00 (m)
	Y	= 5.50 (m)
Radius	R	=11.80 (m)
Moment of resistance	Mr	=1712.6 (kN·m)
Moment of slide	Mo	=2206.1 (kN·m)

No of layer	Saturated unit weight (kN/m^3)	Wet unit weight (kN/m^3)	Internal friction angle (degree)	Cohesion (kN/m^2)
1	10.8	10.8	0	3
2	12.1	12.1	0	5
3	15.8	15.8	0	11
4	19	19	35	100
5	19	19	35	1

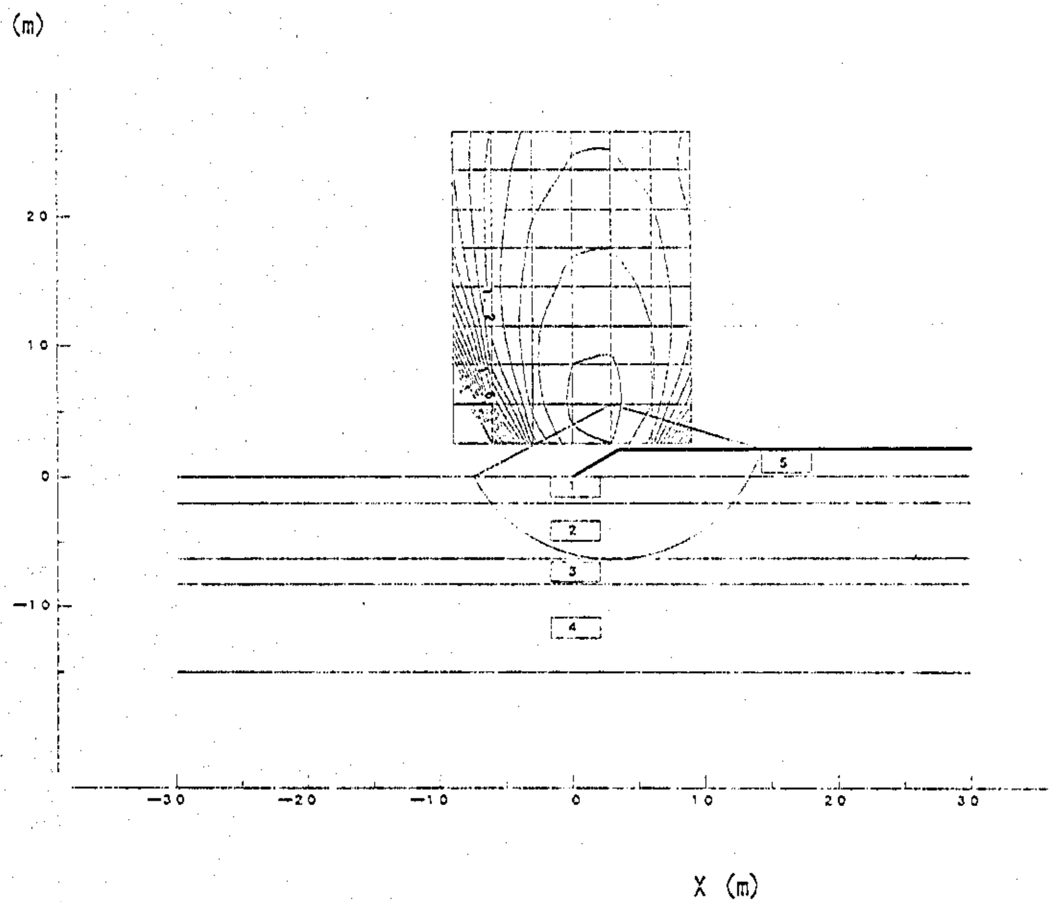


Fig.6.2.11 Minimum safety factor circular arc and equi-safty factor diagram

Case 4: Initial ground condition

Height of road fill	H	= 3 (m)
Slope angle of road fill	V:H	= 1:1.8
Minimum safety factor	Fs min	= 0.557
Center of circular arc	X	= 3.00 (m)
	Y	= 6.50 (m)
Radius	R	=12.80 (m)
Moment of resistance	Mr	=2153.4 (kN·m)
Moment of slide	Mo	=3863.0 (kN·m)

No of layer	Saturated unit weight (kN/m^3)	Wet unit weight (kN/m^3)	Internal friction angle (degree)	Cohesion (kN/m^2)
1	10.8	10.8	0	3
2	12.1	12.1	0	5
3	15.8	15.8	0	11
4	19	19	35	100
5	19	19	35	1

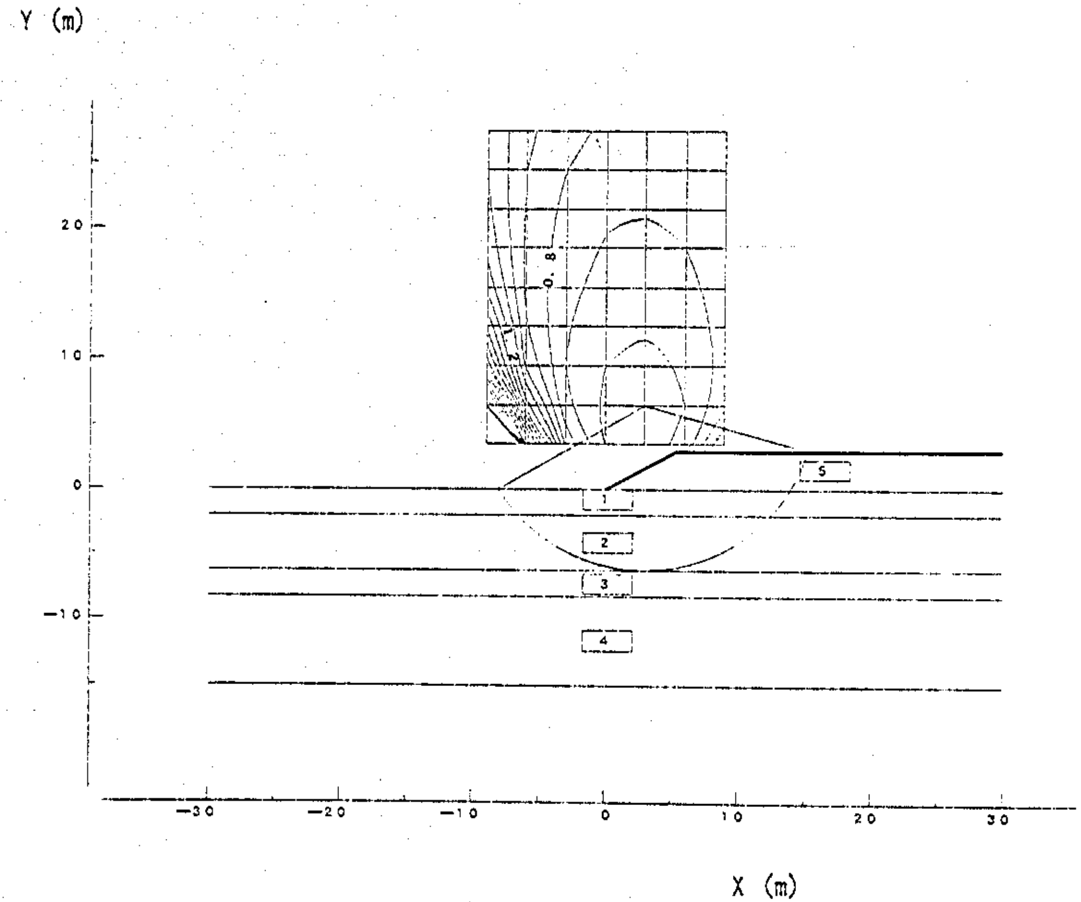
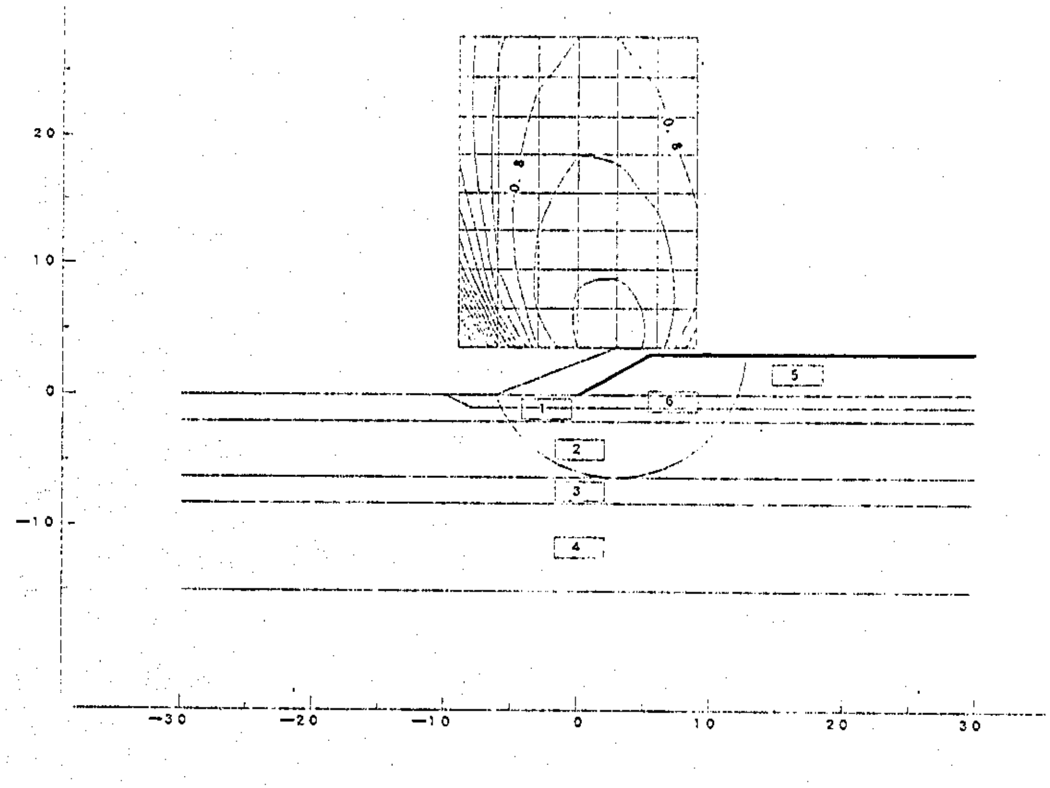


Fig.6.2.12 Minimum safety factor circular arc and equi-safty factor diagram

Improvement case 1:	Replacement thickness of 1m		
Height of road fill	H	= 3	(m)
Slope angle of road fill	V:H	= 1:1.8	
Minimum safety factor	Fs min	= 0.559	
Center of circular arc	X	= 3.00	(m)
	Y	= 3.50	(m)
Radius	R	= 9.80	(m)
Moment of resistance	Mr	=1413.8	(kN·m)
Moment of slide	Mo	=2529.4	(kN·m)

No of layer	Saturated unit weight (kN/m³)	Wet unit weight (kN/m³)	Internal friction angle (degree)	Cohesion (kN/m²)
1	10.8	10.8	0	3
2	12.1	12.1	0	5
3	15.8	15.8	0	11
4	19	19	35	100
5	19	19	35	1
6	18	18	25	0

Y (m)



X (m)

Fig.6.2.13 Minimum safety factor circular arc
and equi-safty factor diagram

Improvement case 2:	Replacement thickness of 2m	
Height of road fill	H	= 3 (m)
Slope angle of road fill	V:H	= 1:1.8
Minimum safety factor	Fs min	= 0.594
Center of circular arc	X	= 3.00 (m)
	Y	= 3.50 (m)
Radius	R	= 9.80 (m)
Moment of resistance	Mr	=1501.9 (kN·m)
Moment of slide	Mo	=2529.4 (kN·m)

No of layer	Saturated unit weight (kN/m³)	Wet unit weight (kN/m³)	Internal friction angle (degree)	Cohesion (kN/m²)
1	10.8	10.8	0	3
2	12.1	12.1	0	5
3	15.8	15.8	0	11
4	19	19	35	100
5	19	19	35	1
6	18	18	25	0

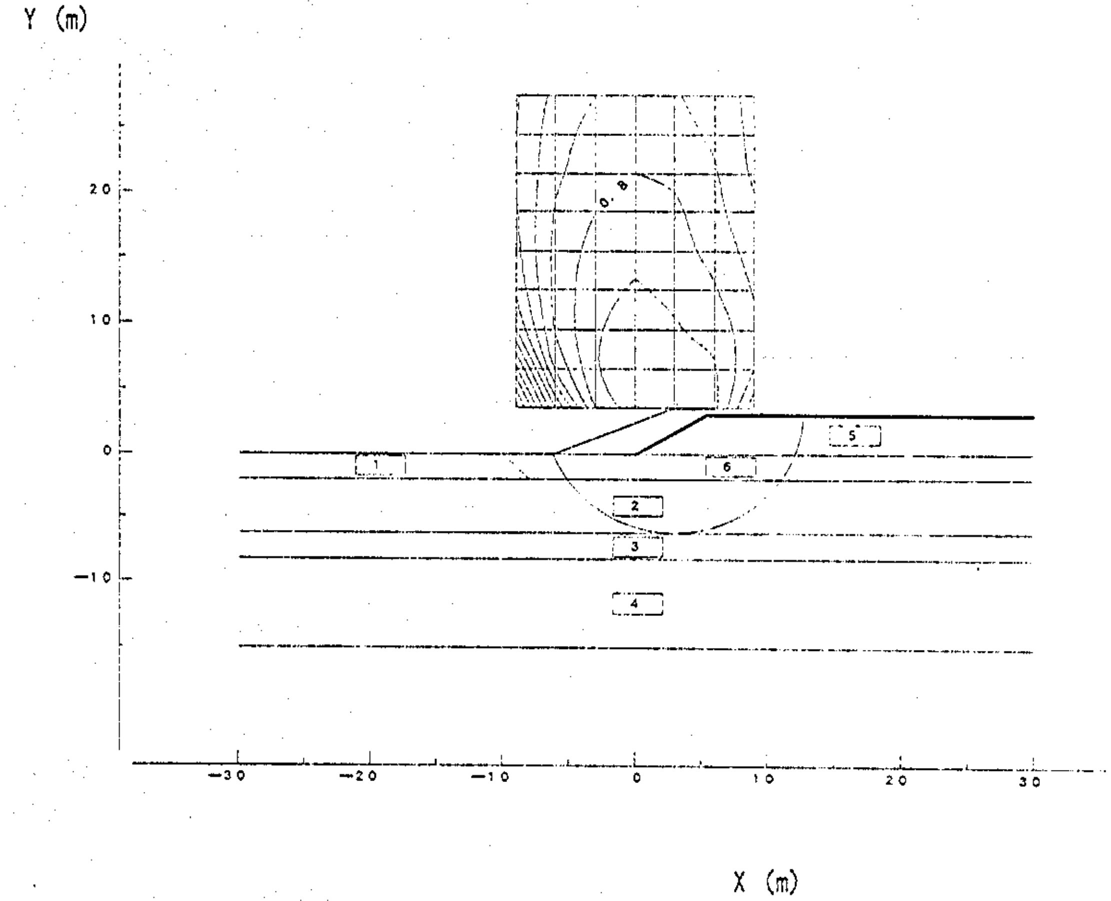


Fig.6.2.14 Minimum safety factor circular arc
and equi-safty factor diagram

Improvement case 3:	Replacement thickness of 3m	
Height of road fill	H	= 3 (m)
Slope angle of road fill	V:H	= 1:1.8
Minimum safety factor	Fs min	= 0.629
Center of circular arc	X	= 0.00 (m)
	Y	= 0.50 (m)
Radius	R	= 6.80 (m)
Moment of resistance	Mr	= 654.3 (kN·m)
Moment of slide	Mo	=1040.0 (kN·m)

No of layer	Saturated unit weight (kN/m³)	Wet unit weight (kN/m³)	Internal friction angle (degree)	Cohesion (kN/m²)
1	10.8	10.8	0	3
2	12.1	12.1	0	5
3	15.8	15.8	0	11
4	19	19	35	100
5	19	19	35	1
6	18	18	25	0

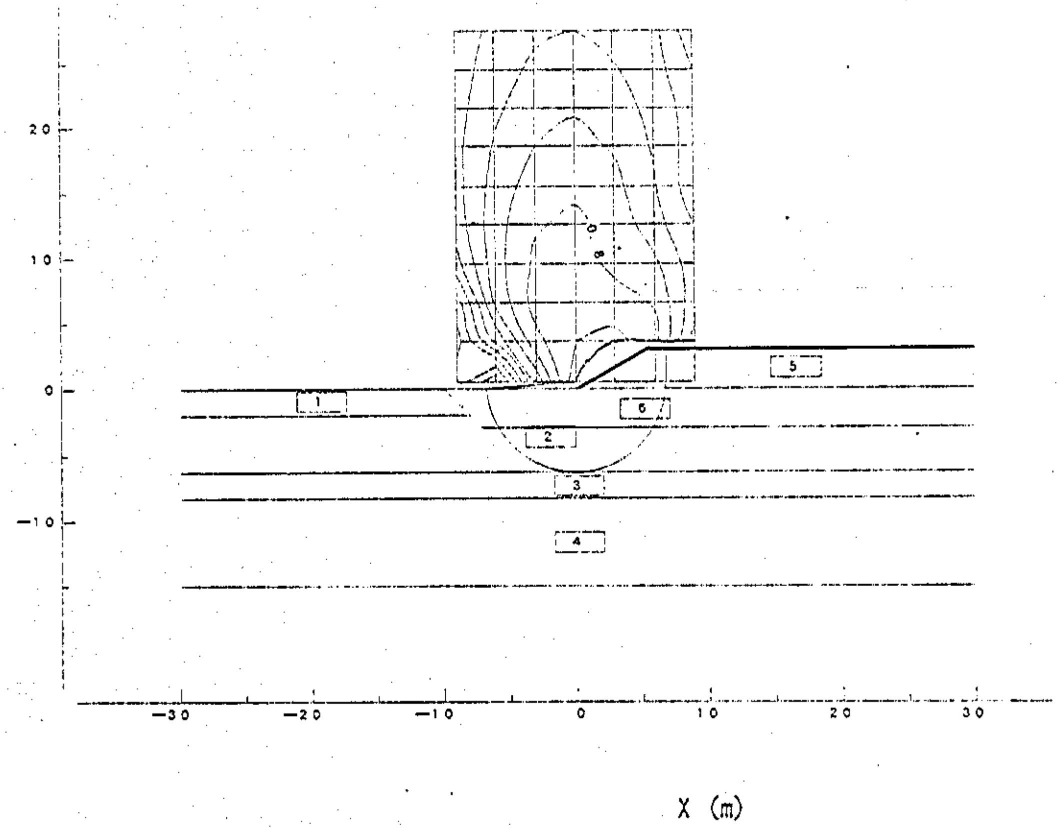


Fig.6.2.15 Minimum safety factor circular arc
and equi-safty factor diagram

Improvement case 4:	Replacement thickness of 4m	
Height of road fill	H = 3 (m)	
Slope angle of road fill	V:H = 1:1.8	
Minimum safety factor	Fs min = 0.792	
Center of circular arc	X = 0.00 (m)	
	Y = 0.50 (m)	
Radius	R = 6.80 (m)	
Moment of resistance	Mr = 823.5 (kN·m)	
Moment of slide	Mo = 1040.0 (kN·m)	

No of layer	Saturated unit weight (kN/m^3)	Wet unit weight (kN/m^3)	Internal friction angle (degree)	Cohesion (kN/m^2)
1	10.8	10.8	0	3
2	12.1	12.1	0	5
3	15.8	15.8	0	11
4	19	19	35	100
5	19	19	35	1
6	18	18	25	0

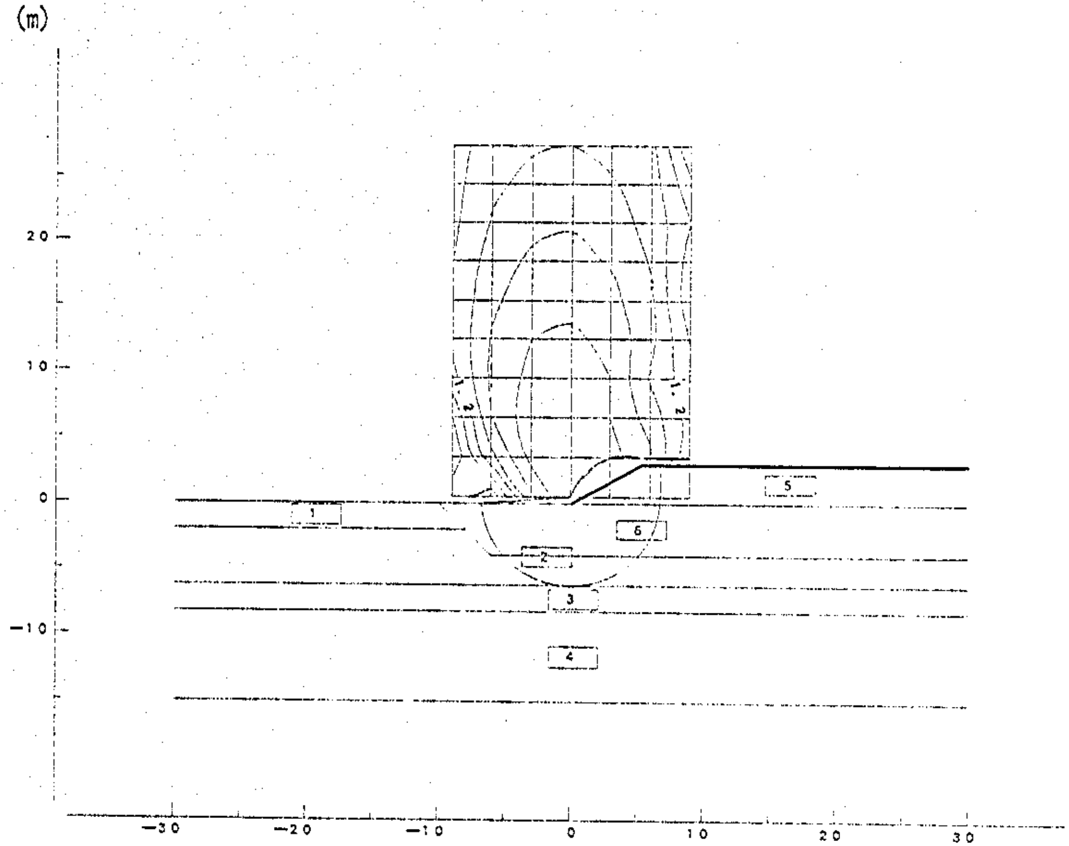


Fig.6.2.16 Minimum safety factor circular arc
and equi-safty factor diagram

Improvement case 5: Replacement thickness of 5m

Height of road fill	H	= 3 (m)
Slope angle of road fill	V:H	= 1:1.8
Minimum safety factor	Fs min	= 1.015
Center of circular arc	X	=-3.00 (m)
	Y	= 6.50 (m)
Radius	R	=14.80 (m)
Moment of resistance	Mr	=5847.9 (kN·m)
Moment of slide	Mo	=5762.6 (kN·m)

No of layer	Saturated unit weight (kN/m³)	Wet unit weight (kN/m³)	Internal friction angle (degree)	Cohesion (kN/m²)
1	10.8	10.8	0	3
2	12.1	12.1	0	5
3	15.8	15.8	0	11
4	19	19	35	100
5	19	19	35	1
6	18	18	25	0

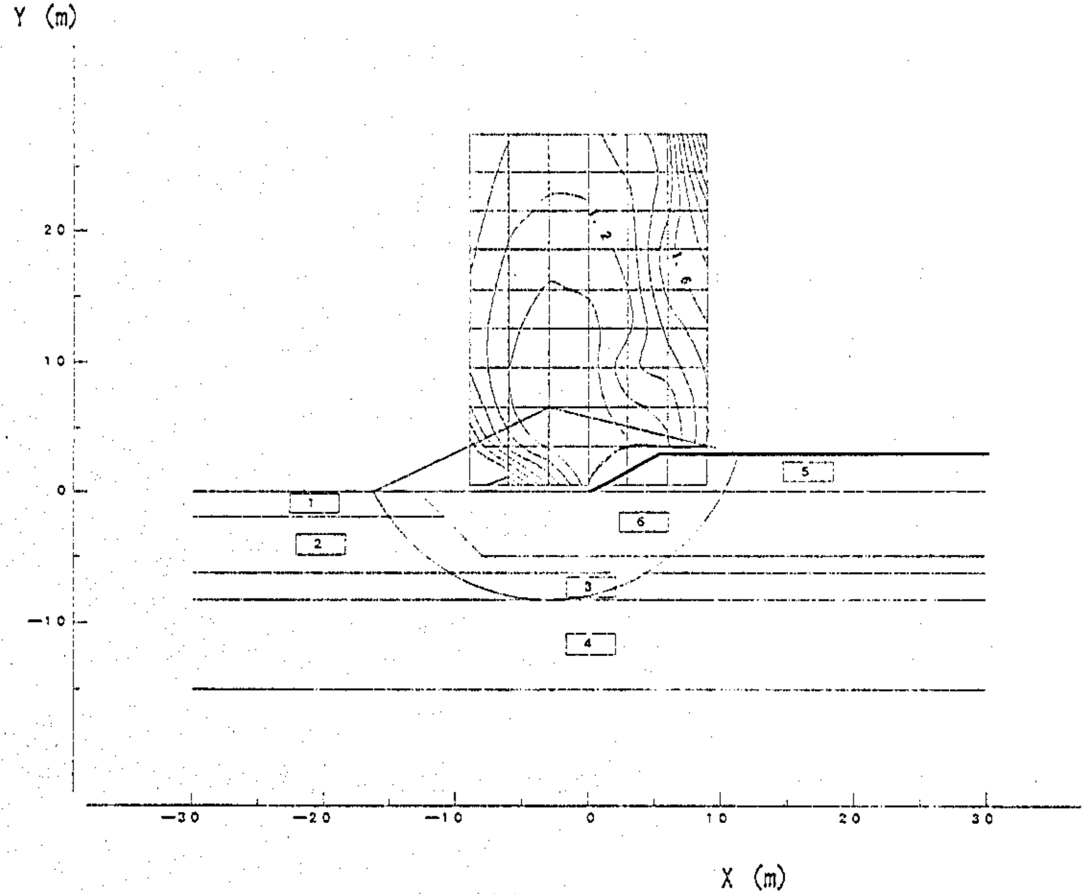


Fig.6.2.17 Minimum safety factor circular arc and equi-safty factor diagram

Improvement case 6: Replacement thickness of 6m

Height of road fill	H	= 3 (m)
Slope angle of road fill	V:H	= 1:1.8
Minimum safety factor	Fs min	= 1.171
Center of circular arc	X	=-3.00 (m)
	Y	= 6.50 (m)
Radius	R	=14.80 (m)
Moment of resistance	Mr	=6935.4 (kN·m)
Moment of slide	Mo	=5923.7 (kN·m)

No of layer	Saturated unit weight (kN/m³)	Wet unit weight (kN/m³)	Internal friction angle (degree)	Cohesion (kN/m²)
1	10.8	10.8	0	3
2	12.1	12.1	0	5
3	15.8	15.8	0	11
4	19	19	35	100
5	19	19	35	1
6	18	18	25	0

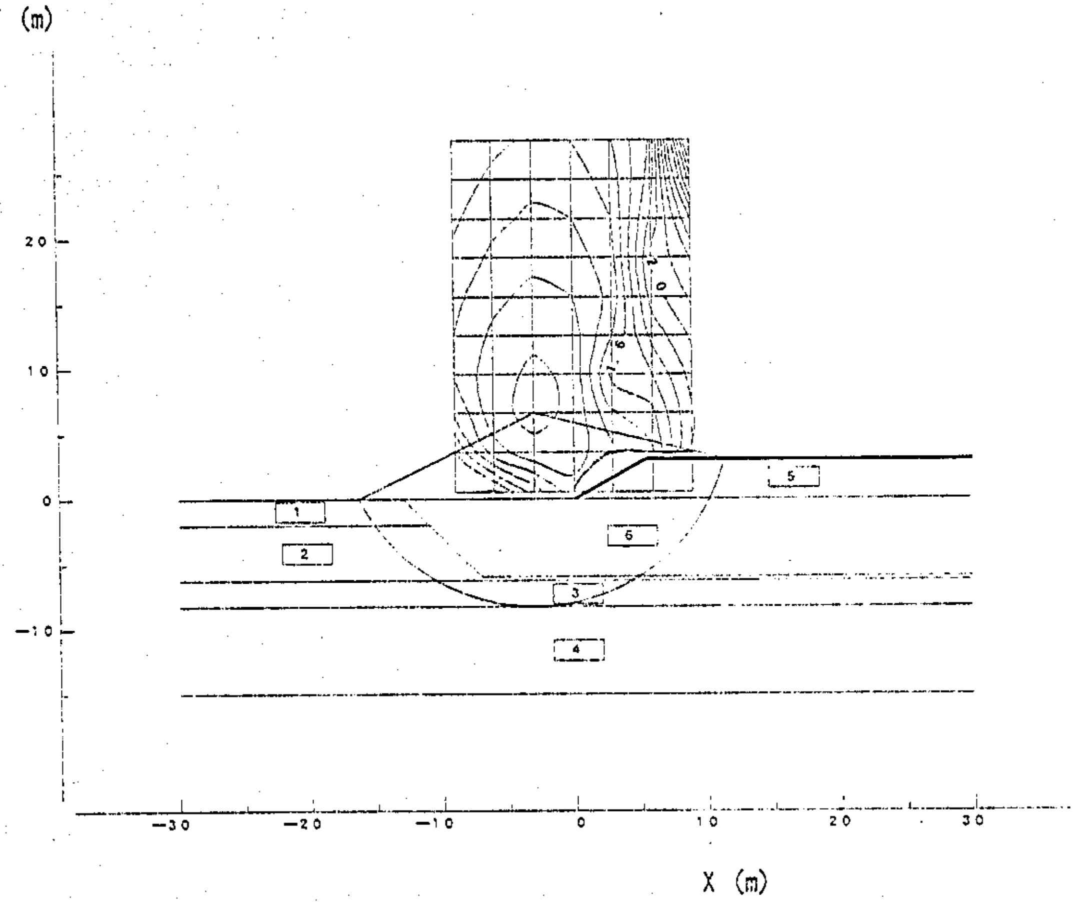


Fig.6.2.18 Minimum safety factor circular arc and equi-safty factor diagram

Improvement case 7: Replacement thickness of 6m

Height of road fill	H	= 4 (m)
Slope angle of road fill	V:H	= 1:1.8
Minimum safety factor	Fs min	= 1.052
Center of circular arc	X	= 0.00 (m)
	Y	= 6.50 (m)
Radius	R	=14.80 (m)
Moment of resistance	Mr	=7651.6 (kN·m)
Moment of slide	Mo	=7272.0 (kN·m)

No of layer	Saturated unit weight (kN/m³)	Wet unit weight (kN/m³)	Internal friction angle (degree)	Cohesion (kN/m²)
1	10.8	10.8	0	3
2	12.1	12.1	0	5
3	15.8	15.8	0	11
4	19	19	35	100
5	19	19	35	1
6	18	18	25	0

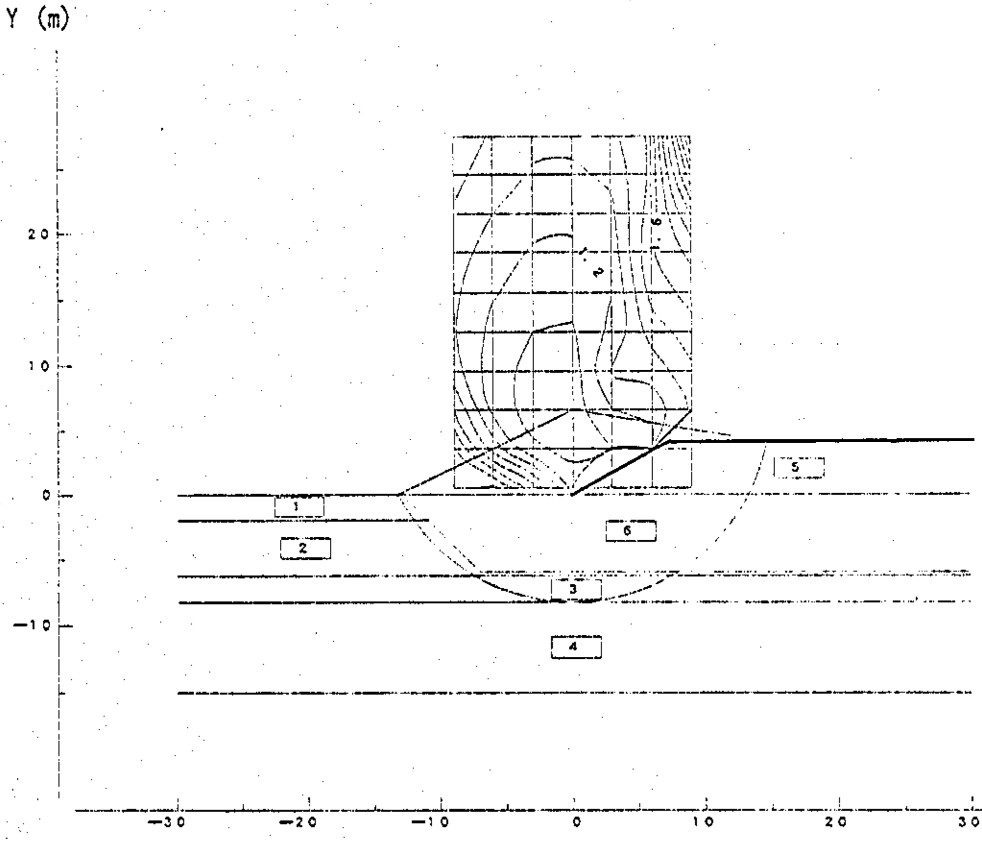


Fig.6.2.19 Minimum safety factor circular arc and equi-safty factor diagram

Improvement case 8:	Replacement thickness of 6m
Height of road fill	H = 5 (m)
Slope angle of road fill	V:H = 1:1.8
Minimum safety factor	Fs min = 0.959
Center of circular arc	X = 0.00 (m)
	Y = 6.50 (m)
Radius	R =14.80 (m)
Moment of resistance	Mr =8329.9 (kN·m)
Moment of slide	Mo =8687.9 (kN·m)

No of layer	Saturated unit weight (kN/m³)	Wet unit weight (kN/m³)	Internal friction angle (degree)	Cohesion (kN/m²)
1	10.8	10.8	0	3
2	12.1	12.1	0	5
3	15.8	15.8	0	11
4	19	19	35	100
5	19	19	35	1
6	18	18	25	0

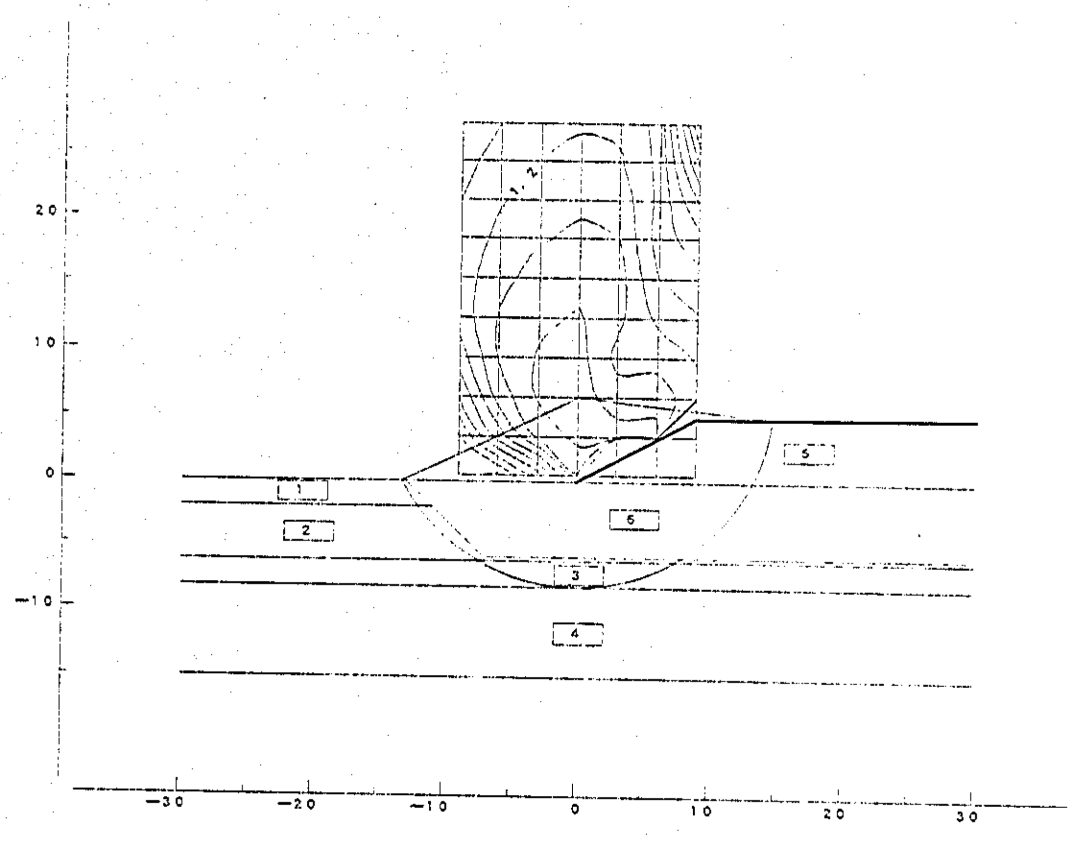


Fig.6.2.20 Minimum safety factor circular arc
and equi-safty factor diagram

Improvement case 9: Replacement thickness of 7m

Height of road fill	H	= 5 (m)
Slope angle of road fill	V:H	= 1:1.8
Minimum safety factor	Fs min	= 1.205
Center of circular arc	X	=-3.00 (m)
	Y	= 9.50 (m)
Radius	R	=17.80 (m)
Moment of resistance	Mr	=13789.4(kN·m)
Moment of slide	Mo	=11441.0(kN·m)

No of layer	Saturated unit weight (kN/m³)	Wet unit weight (kN/m³)	Internal friction angle (degree)	Cohesion (kN/m²)
1	10.8	10.8	0	3
2	12.1	12.1	0	5
3	15.8	15.8	0	11
4	19	19	35	100
5	19	19	35	1
6	18	18	25	0

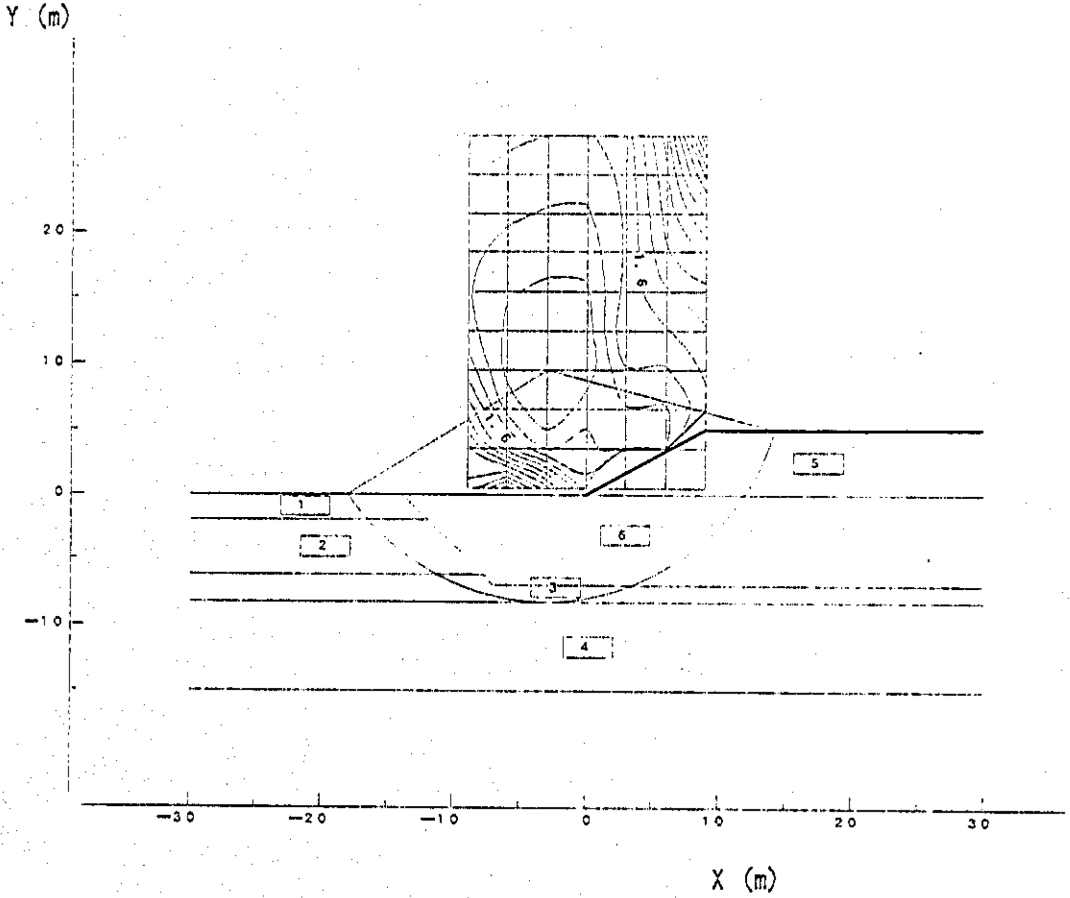


Fig.6.2.21 Minimum safety factor circular arc
and equi-safty factor diagram

Improvement case 10: Replacement thickness of 7m

Height of road fill	H	= 6 (m)
Slope angle of road fill	V:H	= 1:1.8
Minimum safety factor	Fs min	= 1.205
Center of circular arc	X	=-3.00 (m)
	Y	= 9.50 (m)
Radius	R	=17.80 (m)
Moment of resistance	Mr	=14401.4(kN·m)
Moment of slide	Mo	=12714.4(kN·m)

No of layer	Saturated unit weight (kN/m³)	Wet unit weight (kN/m³)	Internal friction angle (degree)	Cohesion (kN/m²)
1	10.8	10.8	0	3
2	12.1	12.1	0	5
3	15.8	15.8	0	11
4	19	19	35	100
5	19	19	35	1
6	18	18	25	0

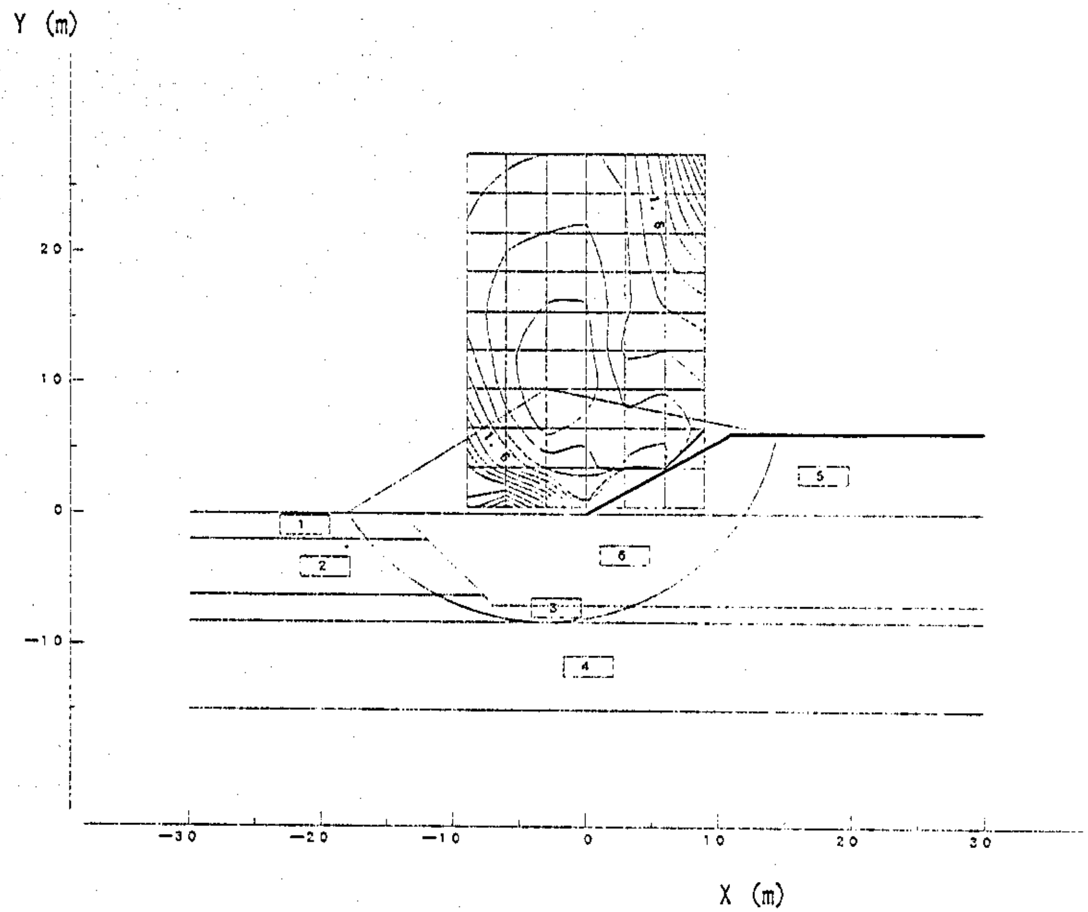


Fig.6.2.22 Minimum safety factor circular arc and equi-safty factor diagram

Improvement case 11: Replacement thickness of 7m

Height of road fill	H	= 7 (m)
Slope angle of road fill	V:H	= 1:1.8
Minimum safety factor	Fs min	= 1.059
Center of circular arc	X	= 6.00 (m)
	Y	= 3.50 (m)
Radius	R	=11.80 (m)
Moment of resistance	Mr	=8719.2(kN·m)
Moment of slide	Mo	=8236.3(kN·m)

No of layer	Saturated unit weight (kN/m³)	Wet unit weight (kN/m³)	Internal friction angle (degree)	Cohesion (kN/m²)
1	10.8	10.8	0	3
2	12.1	12.1	0	5
3	15.8	15.8	0	11
4	19	19	35	100
5	19	19	35	1
6	18	18	25	0

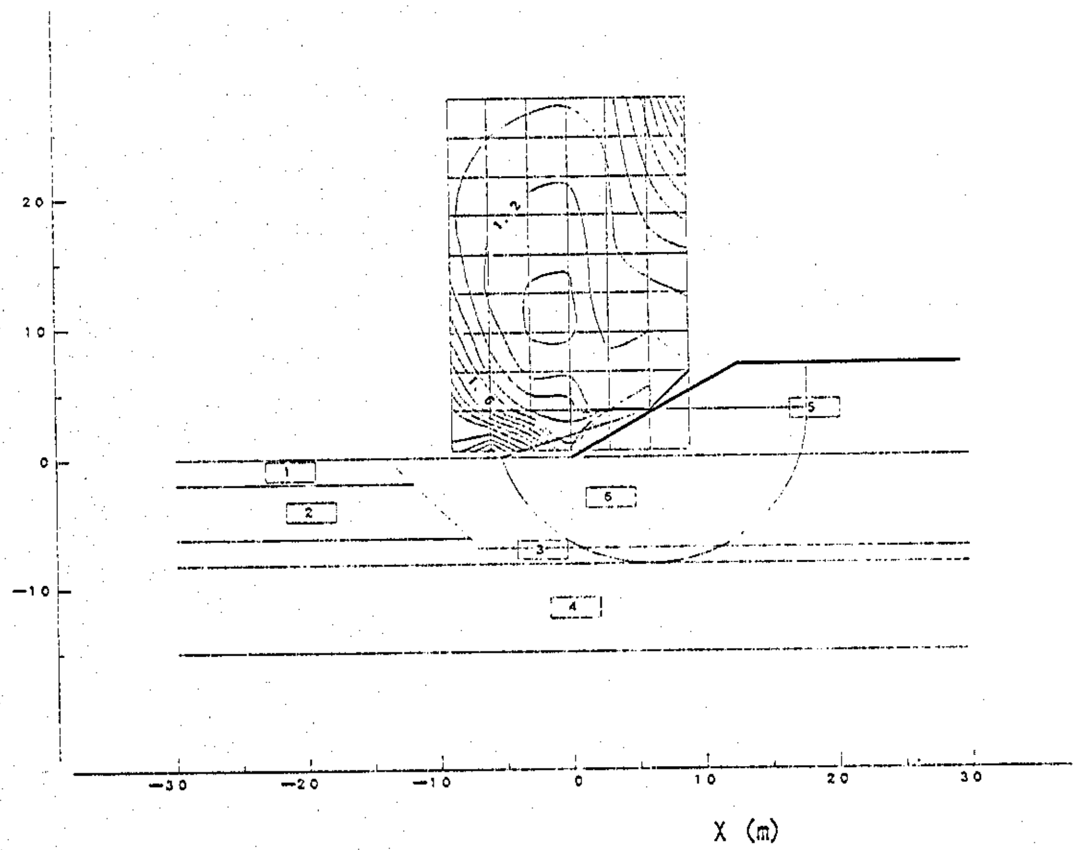


Fig.6.2.23 Minimum safety factor circular arc and equi-safty factor diagram

Improvement case 12: Replacement thickness of 7m

Height of road fill	H	= 8 (m)
Slope angle of road fill	V:H	= 1:1.8
Minimum safety factor	Fs min	= 0.994
Center of circular arc	X	= 6.00 (m)
	Y	= 3.50 (m)
Radius	R	=11.80 (m)
Moment of resistance	Mr	=8967.0(kN·m)
Moment of slide	Mo	=9022.0(kN·m)

No of layer	Saturated unit weight (kN/m³)	Wet unit weight (kN/m³)	Internal friction angle (degree)	Cohesion (kN/m²)
1	10.8	10.8	0	3
2	12.1	12.1	0	5
3	15.8	15.8	0	11
4	19	19	35	100
5	19	19	35	1
6	18	18	25	0

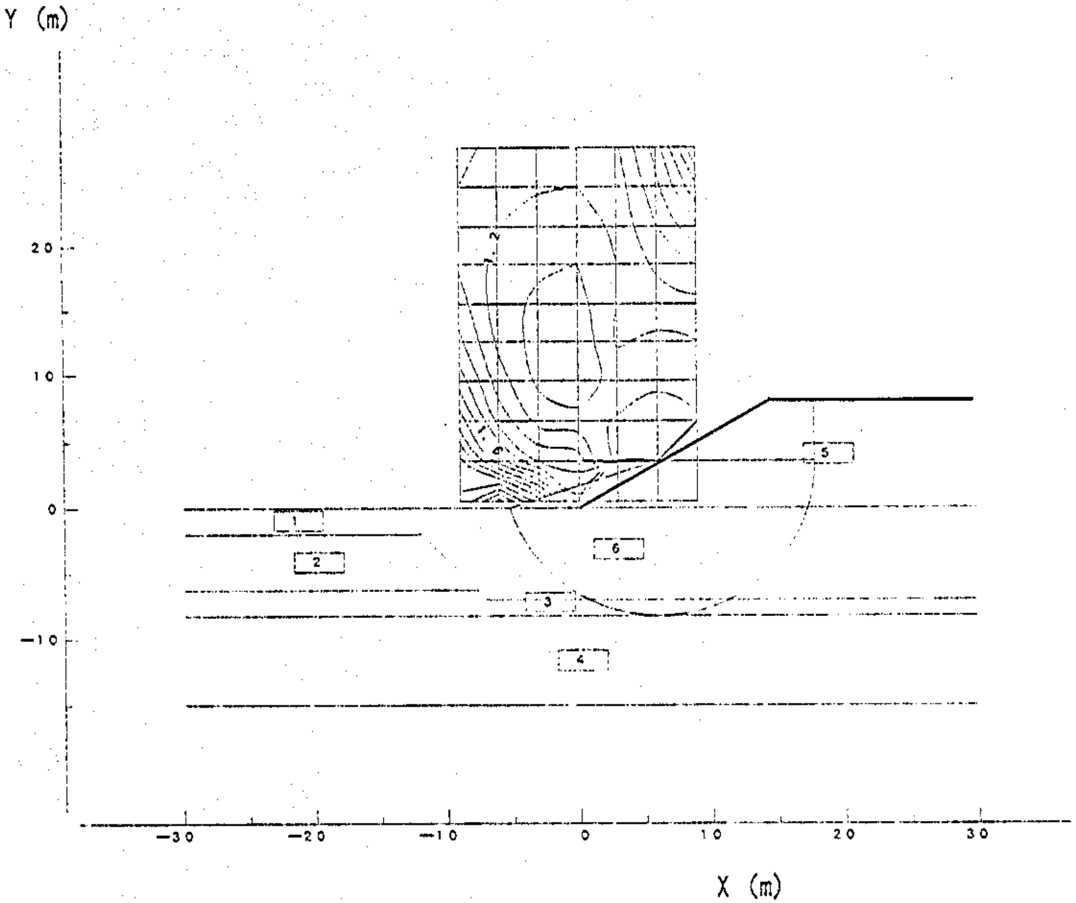


Fig.6.2.24 Minimum safety factor circular arc and equi-safty factor diagram

Improvement case 13: Replacement thickness of 7m

Height of road fill	H	= 9	(m)
Slope angle of road fill	V:H	= 1:1.8	
Minimum safety factor	Fs min	= 0.976	
Center of circular arc	X	= 6.00	(m)
	Y	= 3.50	(m)
Radius	R	=11.80	(m)
Moment of resistance	Mr	=8855.7	(kN·m)
Moment of slide	Mo	=9075.7	(kN·m)

No of layer	Saturated unit weight (kN/m^3)	Wet unit weight (kN/m^3)	Internal friction angle (degree)	Cohesion (kN/m^2)
1	10.8	10.8	0	3
2	12.1	12.1	0	5
3	15.8	15.8	0	11
4	19	19	35	100
5	19	19	35	1
6	18	18	25	0

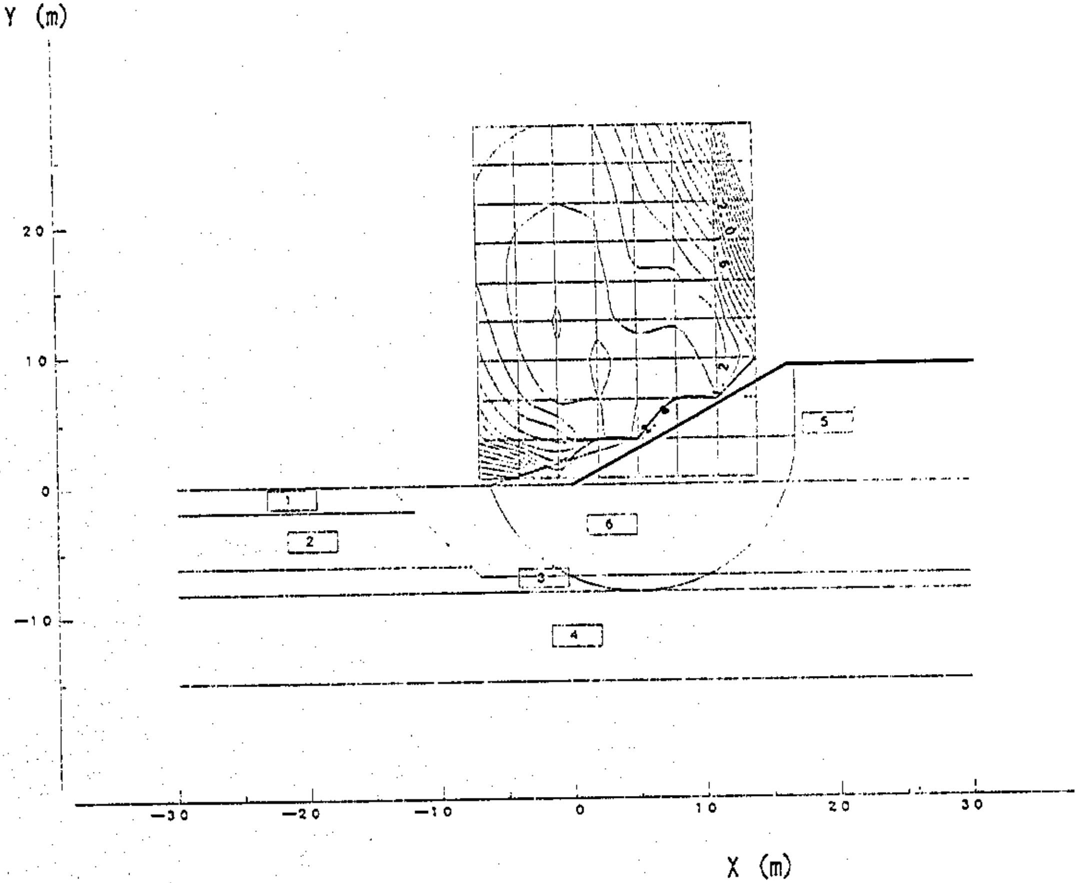


Fig.6.2.25 Minimum safety factor circular arc and equi-safty factor diagram

(4) Stability of the Cut Slope

Fig. 6.2.26 shows a case study concerning the relationship between the cut slope angle and limit height of lateritic soil. According to this study, the slope angle should be less than 45 degrees for the slope height of more than 17.5 m although this may vary depending on the type of soil.

This must be confirmed by means of the actual geological and geotechnical investigation in the future. As shown in Photo 6.2.9, it is common that the thickness of weathered soil over the bedrock varies from several meters to more than ten meters, and it is no easy matter to excavate the hard fresh bedrock for 10 m or more. In particular, vibration and noise must be taken into account for the rock grinding method near the A1 road in the dense residential area. Detailed survey is most desirable for rational design on the required land width, the design of an over-bridge, and determination of construction method. Since the slope stability of weathered soil slope on the bedrock depends greatly on the geological structure (dip slope, geological boundary, fault, fractured zone) and groundwater conditions, thorough survey must be made to ensure the final design that takes into account the required drainage plan, slope protection, etc.

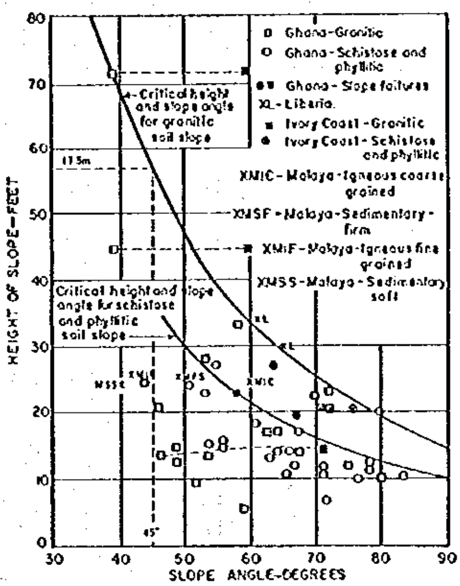


Fig. 6.2.26 Critical Height and Slope Angle for Latritic Soils

[After Liauteaud, 1970 Laterite Soil Engineering P511]

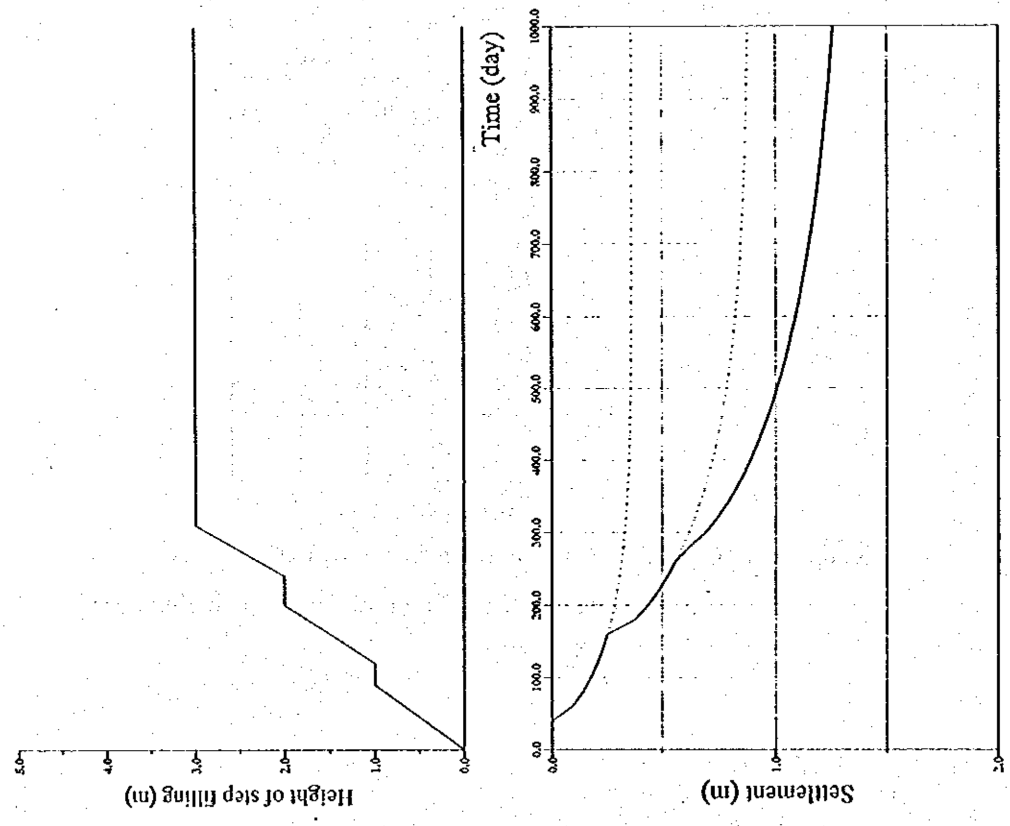


Figure 6.2.27 Consolidation settlement at center of road fill with time in relation to the step filling for original ground at BH-02

3 meters of height of filling

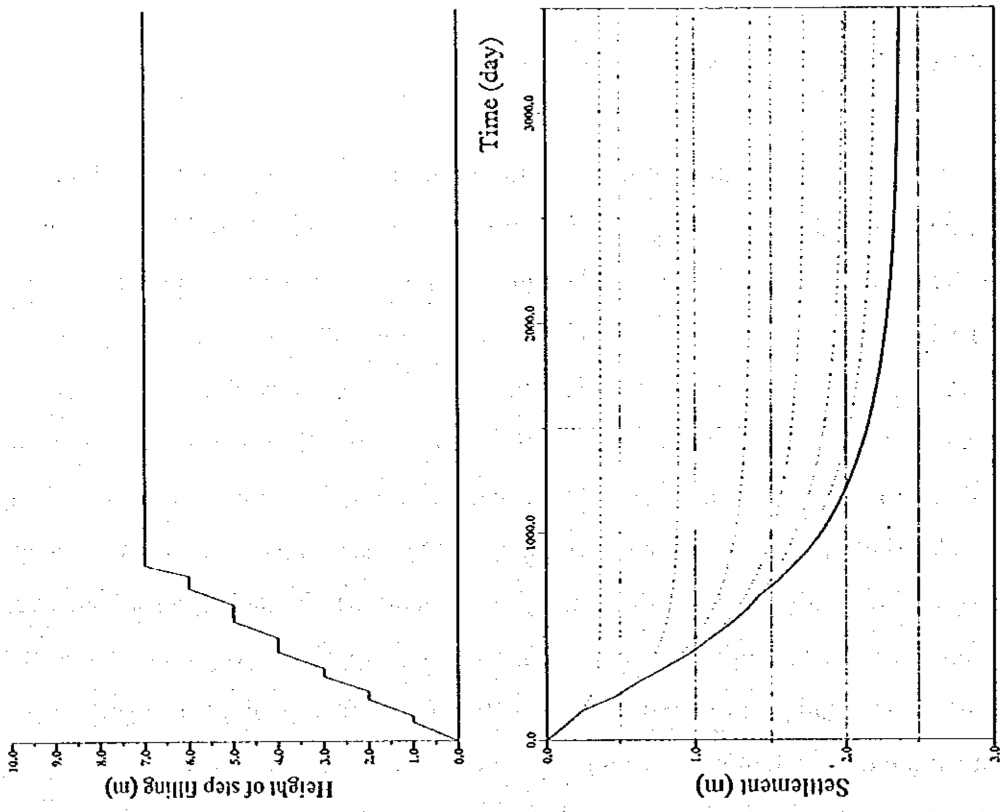


Figure 6.2.28 Consolidation settlement at center of road fill with time in relation to the step filling for original ground at BH-02

7 meters of height of filling

(5) Study on the Bearing Stratum of Structural Foundation

For the foundation of structures (bridges, etc.) to be planned on the route, either the shallow foundation or pile or caisson foundation will be considered depending on the scale of structure and the depth of bearing stratum because the bedrock and its weathered portion appears at a relatively shallow depth.

Large structures should be embedded firmly to the bearing strata with standard penetration test N value N > 30 – 50 or more. As the thickness of weathered portion of bedrock often varies suddenly and the depth of bearing strata is not uniform, it is essential to carry out boring survey for each structure to design the pile length rationally.

In the case of pile foundation, the end bearing capacity of a pile and skin friction varies depending on the work method (including the work reliability). Accordingly, the ordinary method that has achieved successful result in Sri Lanka must be referenced. For the bearing capacity of each bearing strata, the following vertical bearing capacity of the ground can be expected by sufficient penetration to the bearing layer. As the allowable bearing capacity of foundation in the vertical direction is determined from the relationship with the concrete strength used in this project, it is desirable to attempt rational foundation design appropriate to the importance and scale of a structure.

1) The method to calculate the bearing capacity of the ground in the vertical direction can be determined as follows,

a) Pile Foundation

Table 6.2.2 Ultimate end bearing capacity of pile, qd (N/mm^2)

Soil of bearing layer / Type of pile	Gravel and sand layers, and weathered bedrock (N>30)	Hard clayey soil layer	Base rock (N>50) or fresh rock
Driven piles	Db/B > 5: qd = 0.3N Db/B < 5: qd = 0.06 N Df/B – Open end steel piles qd = 0.1 N (1 + 0.4 Df/B) - Closed end piles Note that the upper limit of N value is 40.		21
Cast-in-place piles or PHC piles according to the pile installation by inner excavation (concrete placement method)	3	3 qu	15
PHC piles according to the pile installation by inner excavation (cement milk spray stirring method)	0.15 N: Sand layer 0.2 N : Gravel layer		

Notes N ; SPT N value
Db ; Penetration depth into the bearing layer(m)
B ; Pile diameter (m)
qu ; Unconfined compressive strength (N/mm^2)

Table 6.2.3 Ultimate frictional resistance of pile peripheral surface, f (N/mm^2)

Soil of bearing layer / Type of pile	Gravel and sand layers, and weathered bedrock	Clayey soil layer
Driven piles	N/500 (≦10)	qu/2 or N/100 (≦15)
Cast-in-place piles	N/200 (≦20)	qu/2 or N/100 (≦15)
Pile installation by inner excavation	N/1000 (≦5)	qu/4 or N/200 (≦10)

b) Spread Foundation and Caisson Foundation

To calculate the ultimate bearing capacity of spread foundation in the vertical direction, the cohesion "c" and angle of shear resistance " ϕ " of the ground should first be calculated from the result of plate loading test. Then, the bearing capacity is calculated using a statics equation that takes the eccentric inclination into account:

$$Qu = Ae\,(\alpha k\, c\, Nc + k\, q\, Nq + 0.5\,\gamma 1\, \beta Bc\, N\gamma) \qquad \text{Equation (6.2.1)}$$

For the sandy soil, the following estimate equation may be used with the N value of standard penetration test and angle of shear resistance " ϕ ":

$$=15 + \sqrt{15N} \leqq 45^{\circ} \quad (N \geqq 5) \qquad \text{Equation (6.2.2)}$$

For the cohesion "c" of clayey soil, either 1/2 of unconfined compressive strength qu or the triaxial compressive strength cu of undisturbed sample may be used.

The upper limit of the maximum subgrade reaction for both the bearing strata and bedrock is as follows.

Table 6.2.4 Upper limit of maximum subgrade reaction in ordinary condition, Qu (N/mm^2)

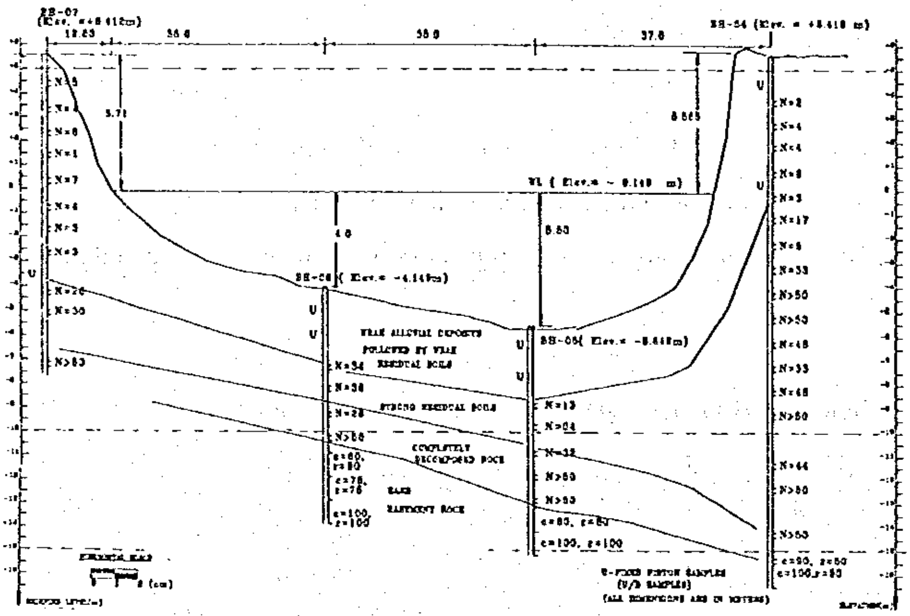
Soil of bearing layer	Max. subfgade reaction, Qu	Guideline
Gravel ground	0.7	N > 70
Sand ground	0.4	N > 40
Clayey soil graound	0.2	N > 20

Table 6.2.5 Upper limit of maximum subgrade reaction of bedrock, Qu (N/mm^2)

Type of bearing bedrock		Max. subgrade reaction in ordinary condition	Guideline	
			qu (N/mm^2)	RQD
Hard rock	A few fissures	2.5	10 or more	20 or more
	Many fissures	1.0		20 or less
Soft rocks and weathered rocks		0.6	1 or more	0

2) Bearing Capacity for Each Structure

a) Kelani River Crossing Point



Fig－6.2.29 Geological cross section (Provisional)

(Section: Kelani River crossing point)

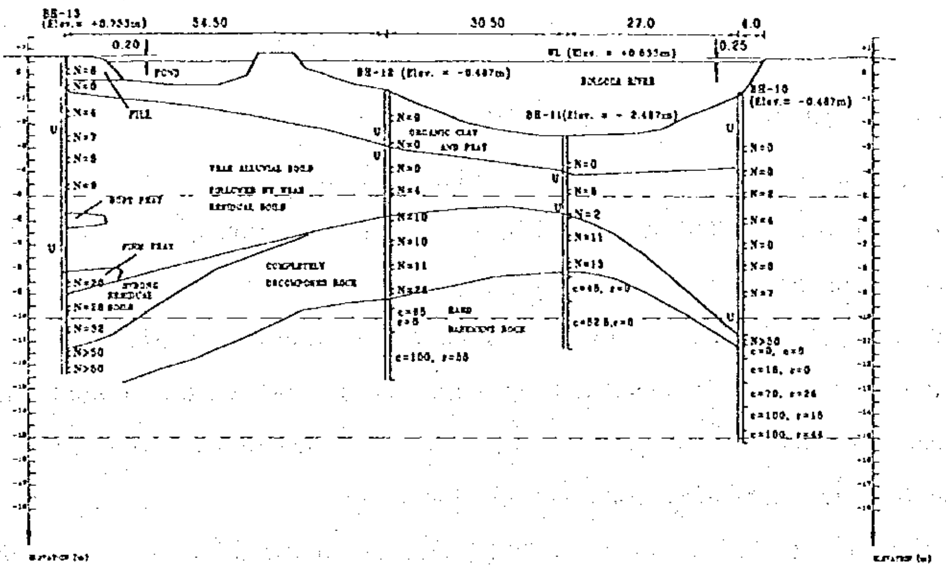
i) North-side abutment, BH-04: The sandy layer at the elevation of –5.0 m (GL – 10.2 m) or deeper with N > 50 is considered appropriate to bear the pile foundation.

ii) South-side abutment, BH-07: The weathered bedrock at the elevation of –7.4 m (GL – 12.8 m) or deeper with N > 50 is considered appropriate to bear the pile foundation.

iii) North-side pier, BH-05: The weathered bedrock at the elevation of –12 m (GL – 6.3 m) or deeper with N > 50 is considered appropriate to bear the pile foundation.
The caisson foundation using the fresh bedrock at the elevation of –13.3 m (GL – 7.7 m) or deeper as a bearing layer may also be considered for comparison.

iv) South-side pier, BH-06: The weathered bedrock at the elevation of –10.5 m (GL – 6.3 m) or deeper with N > 50 may become the bearing layer of pile foundation. In the case of driven piles, further geotechnical investigation is necessary in the future because the fresh bedrock appears at the elevation of –11.0 m (GL – 6.8 m) or deeper that prevents penetration of piles. Besides, excessive inclination or unevenness of the bedrock surface may cause slip of pile end.
The caisson foundation using the fresh bedrock at the elevation of –11.0 m (GL – 6.8 m) or deeper as a bearing stratum may also be considered for comparison.

b) Bolgoda River Crossing Point



Fig−6.2.30 Geological cross section (Provisional)
(Section: Bolgoda River crossing point)

i) East-side abutment, BH-10: The caisson foundation using the weathered bedrock at the elevation of −10.0 m (GL − 9.5 m) or deeper as a bearing layer is recommended. In the case of pile foundation, due consideration must be made on ground improvement because of possibility that the abutment may develop substantial displacement due to lateral flow of the ground caused by fill on the backside as described in 6.2.4 (3), 3).

ii) West-side abutment, BH-13: The weathered rock at the elevation of −12.5 m (GL − 11.8 m) or deeper with N > 50 is considered appropriate as a bearing layer of pile foundation.

iii) East-side pier, BH-11: The weathered rock at the elevation of −9.5 m (GL − 7 m) or deeper with N > 50 is considered appropriate as a bearing layer of pile foundation.

iv) West-side pier, BH-12: The weathered bedrock at the elevation of −9.5 m (GL − 9 m) or deeper with N > 50 may become the bearing layer of pile foundation. In the case of driven piles, detail geotechnical investigation is necessary in the future because the fresh bedrock appears at the elevation of −10.5 m (GL − 10 m) or deeper that prevents penetration of piles. Besides,

excessive inclination or unevenness of the bedrock surface may cause slip of RC pile end.
The caisson foundation using the fresh bedrock at the elevation of –10.5 m (GL – 10 m) or deeper as a bearing stratum may also be considered for comparison.

c) Interchanges

i) A3 national road, BH-01: The sand layer at the elevation of –10.5 m (GL-11 m) or deeper is considered appropriate as a bearing stratum of pile foundation.

ii) A1 national road, BH-03: The weathered bedrock at the elevation of +4.7 m (GL – 4 m) or deeper is considered appropriate as a bearing stratum of shallow foundation.

iii)A4 national road, BH-08: The weathered bedrock at the elevation of +0.5 m (GL – 10 m) or deeper with N > 45 or more is considered appropriate as a bearing stratum of pile foundation.

iv) A8 national road, BH-09: The weathered bedrock at the elevation of +8.3 m (GL – 9 m) or deeper is appropriate as a bearing stratum for pile foundation.

v) A2 national road, BH-15: The sandy layer at the elevation of +0 m (GL – 3,3 m) or deeper with N > 20 – 30 is considered appropriate as a bearing stratum for spread foundation. Note that dewatering must also be made during the work because the groundwater level is at GL – 0.6 m (elevation + 2.7 m). The sand layer at the elevation of –3.7 m (GL – 7 m) or deeper with N > 35 or more may also be considered as a bearing stratum for pile foundation.

6.2.5 Study on the Maximum Flood Run-Off of Kelani and Bolgoda Rivers

The tropical climate of Sri Lanka is characterized by two rainfall patterns; south-western monsoon and north-eastern monsoon. Normally, the precipitation is highest during both monsoon seasons south-western monsoon season is from May to September (so-called the YALA season) and the north-eastern monsoon season is from December to February (so-called the MAHA season). In Sri Lanka, the hydrological year begins with October and ends with September, the next year. (The 1999 hydrological year began with October, 1998 and ended with September, 1999.)
As shown in Fig. 6.2.31, the climate division of Sri Lanka includes three climates according to the annual precipitation pattern; heavy rain, medium, and light rain. The inland water of the heavy rain climate area occurs practically during two monsoon seasons:

(1) Kelani River

This river is located in the heavy rain climate area (Fig. 6.2.32). The monthly average flow rate during a period from 1972 to 1997 is as shown in Fig. 6.2.35 and Fig.6.2.36. Details of observation stations are shown in Fig. 6.2.33 and Table 6.2.7. Fig. 6.2.34 shows estimated flood profiles of Kelani river studied by the Irrigation Department of Sri Lanka.

Fig. 6.2.36 shows the monthly average run-off at the Hanwella station. During a period from 1972 to 1997, the maximum flood run-off occurred at 2349.46 m3/sec in October 29, 1973.

Table 6.2.9 shows the occurrence probability of flood run-off estimated on the basis of the annual maximum flow rate during a period from 1972 to 1997 by using the Gumbel distribution.

Note that the elevation of the under-surface of the girder of Kaduwela bridge located about 1.3 km upstream from the crossing point was confirmed to be 9.377 m during this survey.

(2) Bolgoda River

The Bolgoda Lake, as shown in Fig. 6.2.38, is a slack catchment area within a relatively narrow range, which is located in a watershed defined by the Kalu River system on the north and Kelani River system on the south. This lake has two major catchment areas (north and south lakes) that are connected by the Bolgoda River. The north lake opens to the sea via the Panadura estuary. The south lake connects to the sea at Pinwatta via a slender artificial channel known as Talpitiya Canal that is closed almost throughout the year due to sedimentation. The Bolgoda River runs from the north-eastern end of south lake toward the north lake, and the crossing point of this selected route is located several 100 m from the north-eastern end of the south lake.

As shown in Fig. 6.2.39, many tributaries, such as Panape River, Kapu River, etc. merge before flowing into the Bolgoda River. In addition, the Maha river and agricultural water channels connect to the south lake from the southern end via two small rivers known as the Madubokka River and Hungurilla River. The drainage area of this south lake is about 160 km^2.

The average water depth of Bolgoda Lake is about 2 m, with the tide level ranging from 1 cm to 75 cm. As shown in Fig. 6.2.38, there are many hydrological observation stations around this lake. But it is difficult to determine the occurrence probability of flood run-off according to the ordinary method because data to analyze the flood level has not been obtained.

(3)Interview Result on the Flood Level

According to the result of interview, the flood level was confirmed to be the elevation + 5.4 m at the crossing point of Kelani River and the elevation + 1.0 - +2.0 m at the crossing point of the Bolgoda River.

Table 6.2.6 Result of interview with residents on the flood level along the route

Position	Station	Maximum flood water level elevation (m)	Year when the flood occurred, etc.
BH-01	STA-3	+1.0 (GL+0.3)	1989
BH-04	STA160	+6.3 (GL+0.9)	1948,Bank of Kelani river
BH-09	STA424	+3.0 (GL+0.95)	1989, A-8
BH-14	STA485.5	+1.5 (GL+0.5)	1948, Paddy by Bolgoda lake
Boat yard	STA444	+1.9 (GL+1.0)	1948, Thotupola
M-01	STA42	(GL+1.5)	1948
M-02	STA148	(GL+2.2)	1948
M-03	STA307	(GL+0.5)	1989
M-04	STA357	(GL+0.3)	1989
CU-09	STA180	(GL+0.3)	1989
CF-09	STA338	(GL+0.6)	1989
GPS-9912		(GL+0.5)	1989

Table 6.2.7 Average annual rainfall of meteorological stations in Kelani river catchments area

METEOROLOGICAL STATION		LOCATION		ELEV.: (M)	AVG. ANNUAL RAINFALL (MM)	PERIOD
CODE	NAME	LAT.	LONG.			
M023	ANNEFIELD	065227N	803759E	1311	2858	1943-1985
M031	ARSLENEA	065725N	802921E	457	4560	1948-1982
M061	BLAIR ATHOL	065121N	803442E	1140	3153	1948-1973
M069	CAMPION	064648N	801147E	1524	2361	1943-1985
M103	DIGALLE ESTATE	065731N	801746E	122	4233	1949-1985
M105	DIWELA ESTATE	071243N	802301E	244	2643	1949-1971
M115	DUNEDIN	070232N	801607E	122	4014	1949-1985
M209	INGOYA ESTATE	070035N	801551E	305	4904	1949-1985
M268	KELLIE ESTATE	070611N	802600E	954	4761	1949-1984
M314	LIMYAGALA GROUP	065557N	802141E	259	4258	1949-1985
M318	LUCCOMBE ESTATE	065222N	803154E	1097	4083	1943-1976
M341	MALIBODA	065327N	802526E	274	5643	1949-1984
M430	NORTON BRIDGE	065456N	803106E	893	4907	1943-1985
M431	NORWOOD (NEW VALLEY)	065038N	803559E	1122	2870	1943-1985
M482	PINDENIYA	071007N	801726E	76	3115	1949-1973
M539	SOUTH WANARAJAH	065153N	803456E	1140	2713	1949-1981
M579	UDAHENKANDE	065548N	801955E	1140	4421	1952-1968
M626	WEWLITALAWA	070314N	802257E		5534	1949-1985

Table 6.2.8 Mean monthly rainfall of meteorological stations in Kelani river catchments during period specified in Table 6.2.7

STATION CODE	MEAN MONTHLY RAINFALL												
	OCT	NOV	DEC	JAN	FEB	MAR	APR	MAY	JUN	JUL	AUG	SEP	YEARLY TOTALS
M023	308.6	260.9	167.7	78.0	84.0	180.6	266.0	287.3	360.0	329.0	275.5	256.1	2853.6
M031	565.6	364.6	181.7	74.3	93.2	159.6	315.7	581.4	676.8	609.1	483.6	506.2	4611.8
M061	328.5	229.3	165.7	77.4	74.1	180.1	288.7	325.9	437.0	379.0	300.7	322.8	3109.2
M069	310.7	297.4	198.2	112.2	98.3	166.9	257.8	227.5	205.6	166.4	150.9	174.8	2362.2
M103	540.0	450.2	220.9	127.0	143.6	283.2	460.7	528.3	426.0	343.9	285.7	425.8	4235.2
M105	400.1	298.4	173.6	90.4	94.4	167.8	271.5	245.3	264.0	257.9	156.4	223.4	2643.2
M115	516.4	466.3	219.1	109.7	146.0	278.7	441.1	469.0	403.7	322.1	255.3	392.2	4019.6
M209	615.7	403.6	204.5	106.5	116.2	221.7	383.0	641.9	642.3	547.1	453.5	573.4	4909.4
M268	597.2	492.0	253.0	87.0	106.0	197.5	355.0	555.6	617.0	526.4	506.8	506.9	4800.4
M314	550.0	390.4	184.2	115.0	122.9	241.1	433.4	573.6	501.8	399.4	330.4	414.5	4256.7
M318	484.4	283.7	183.9	82.1	81.4	152.3	251.9	433.8	622.1	563.5	505.4	411.6	4086.1
M341	725.9	475.7	251.2	126.4	134.9	280.6	461.5	733.0	742.2	550.2	495.9	593.0	5570.3
M430	573.6	374.5	195.6	84.6	109.0	169.1	280.0	515.4	765.7	669.8	608.0	566.2	4911.6
M431	311.6	265.6	169.2	74.4	79.6	224.3	311.1	297.0	346.7	299.3	242.5	265.9	2887.3
M482	457.2	391.8	181.8	95.9	114.7	248.2	344.3	357.1	313.8	218.4	179.4	287.5	3189.9
M539	284.4	226.9	133.4	58.1	72.0	169.1	262.9	304.8	348.9	304.5	265.7	274.2	2704.7
M579	577.6	417.9	242.1	108.1	164.1	292.8	415.6	518.8	482.9	384.4	326.6	430.8	4361.6
M626	751.4	552.9	246.6	107.3	136.0	298.6	480.8	670.2	677.6	519.5	480.0	571.7	5492.6

Table 6.2.9 Caluculated floods for different return periods using Gumbel distribution at Hanwella

Return Period (T) years	K_T	Discharge (Q_T) m³/s	Standard Error ($se\ Q_T$) m³/s	(Q_T)/ ($se\ Q_T$)%
50	2.592	2681.26	371.38	13.85
100	3.140	2983.50	418.58	14.03
150	3.454	3156.34	468.55	14.84
200	3.679	3280.35	494.07	15.06

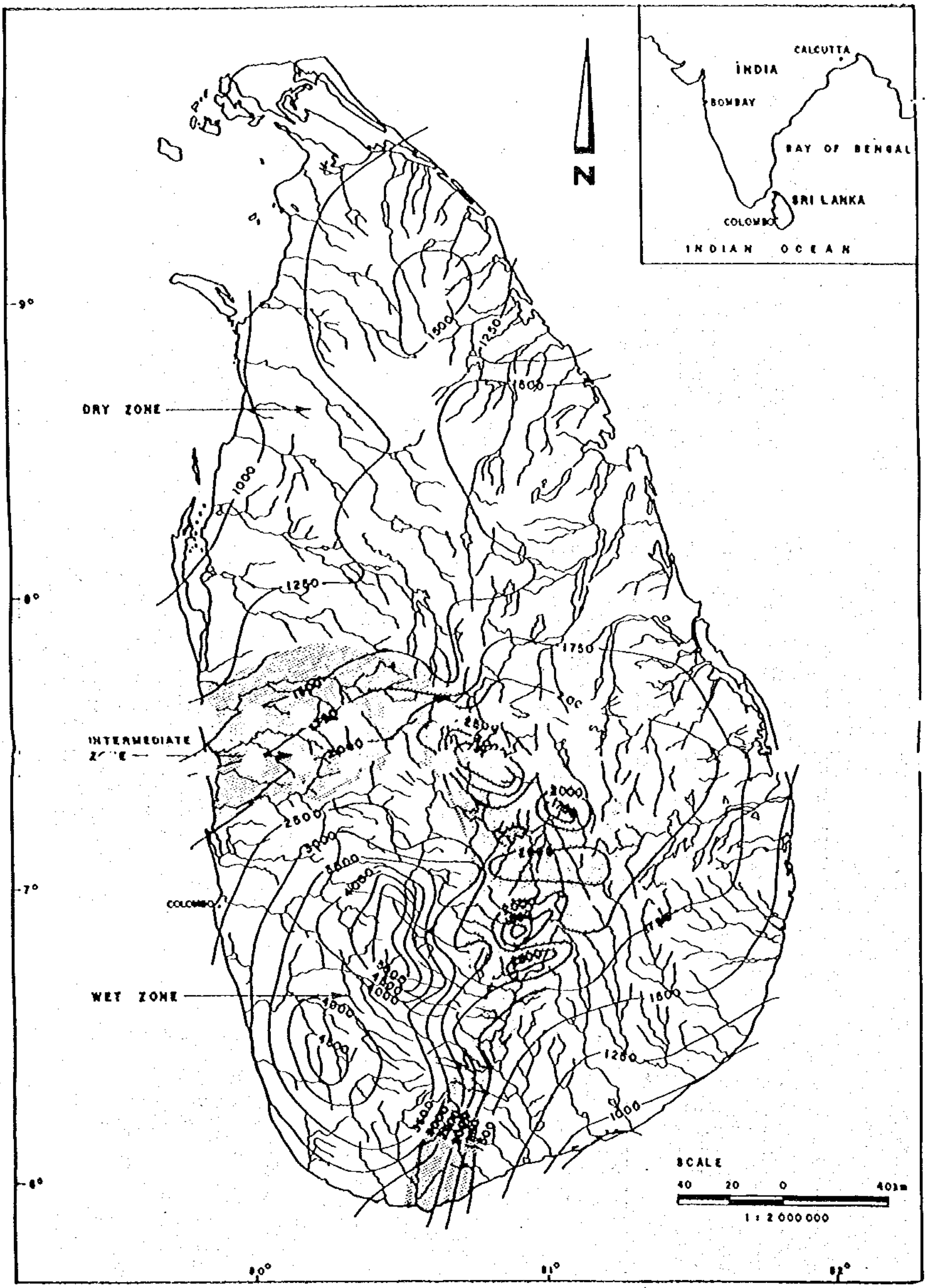


Figure 6.2.31 Climatic zones and annual isohyets (mm)

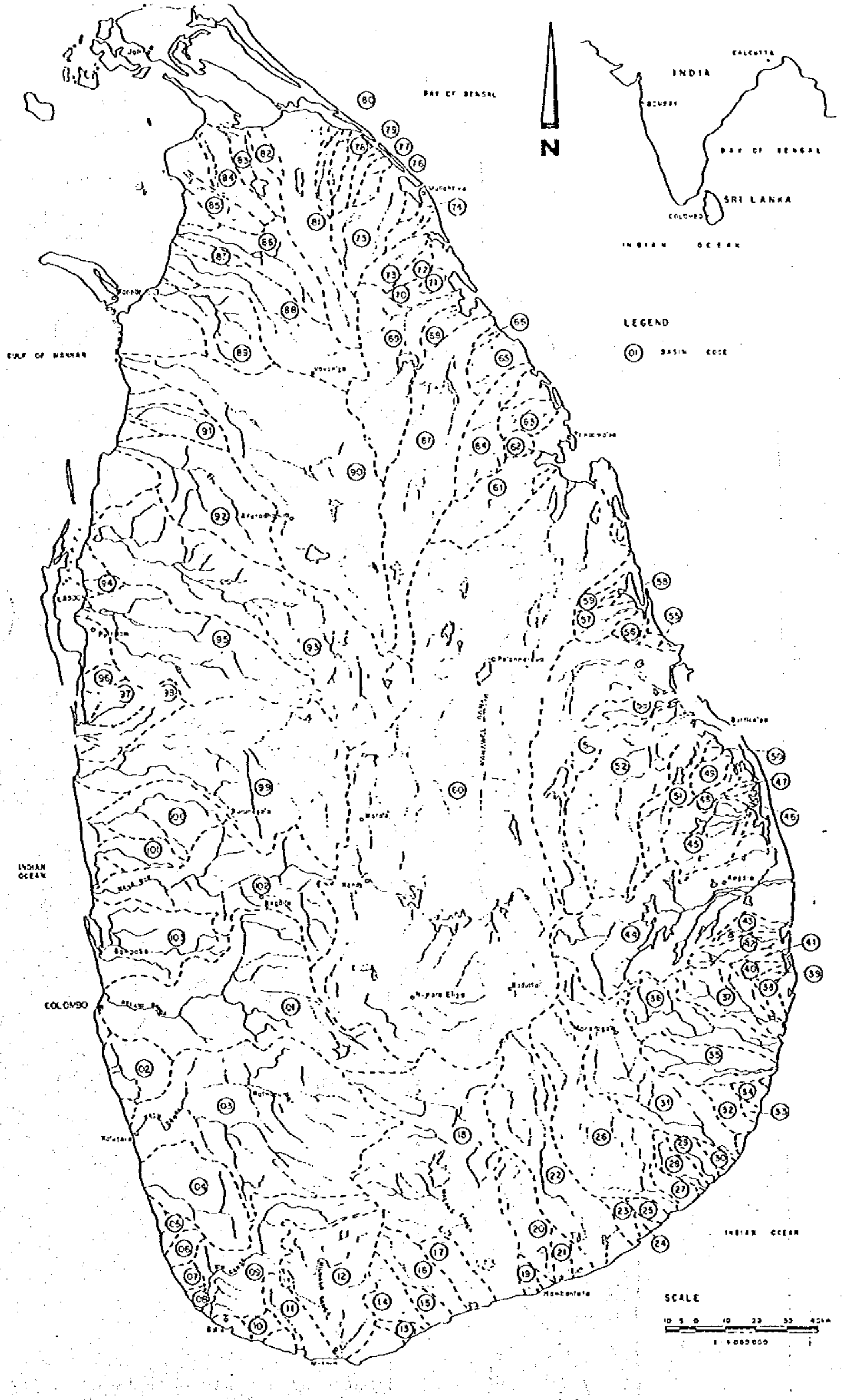


Figure 6.2.32 Geographical location of Kelani river

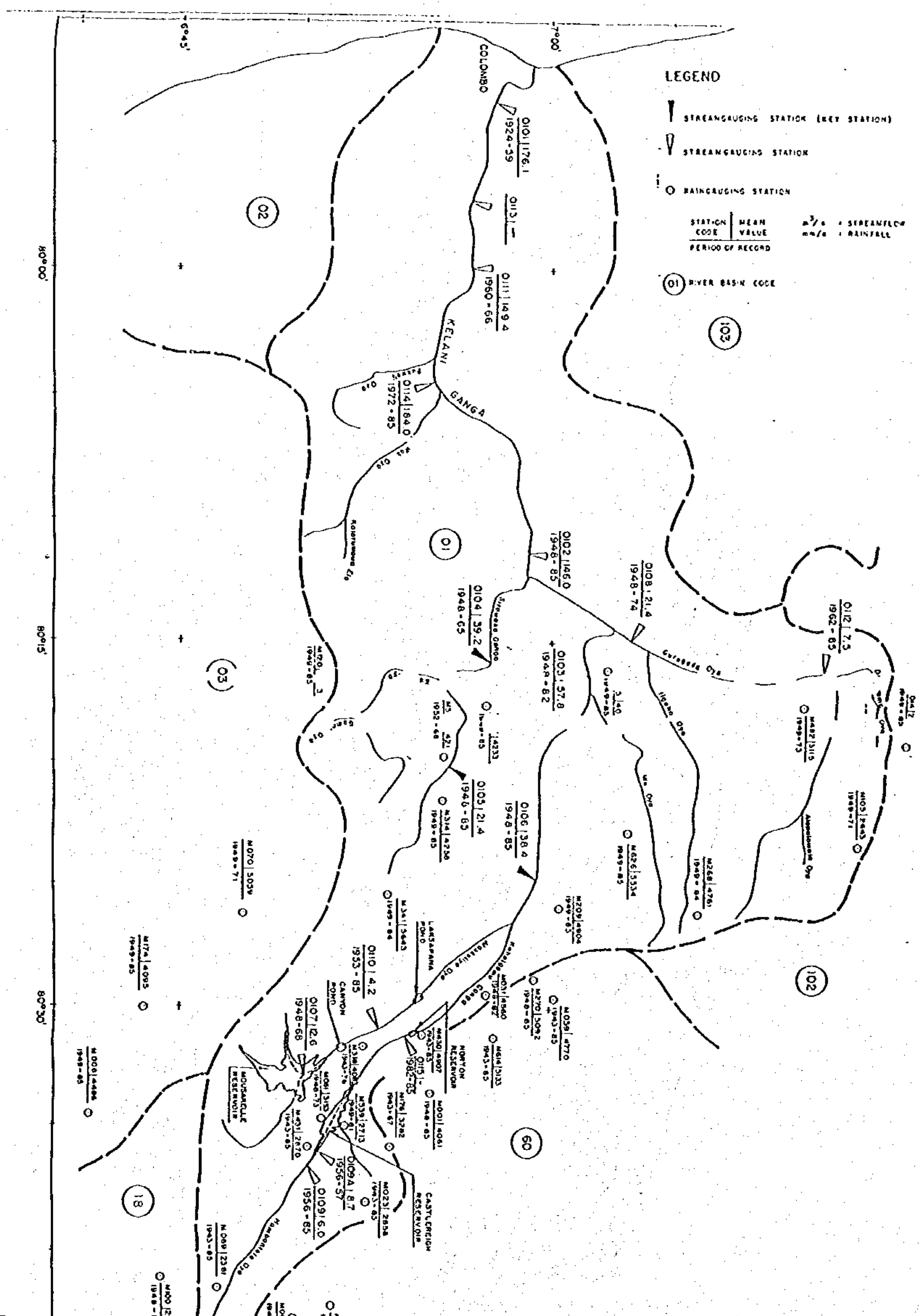


Figure 6.2.33 Meteorological stations and hydrometric stations of Kelani river basin

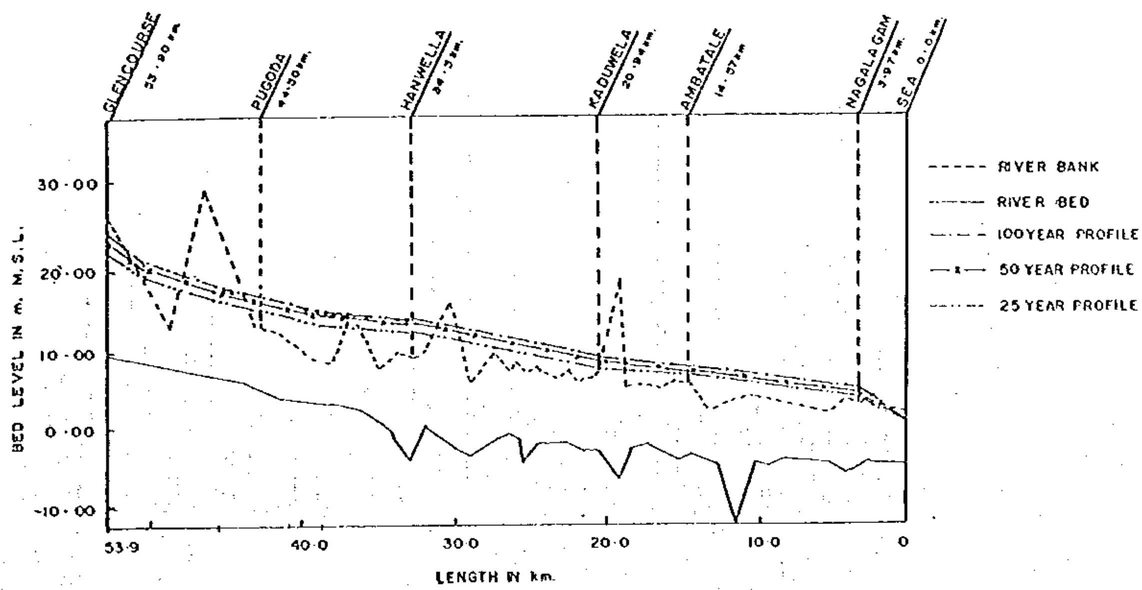


Figure 6.2.34 Flood profiles of Kelani river

Surveyed in 1990

Source from Hydrology Division, Irrigation Department of Sri Lanka

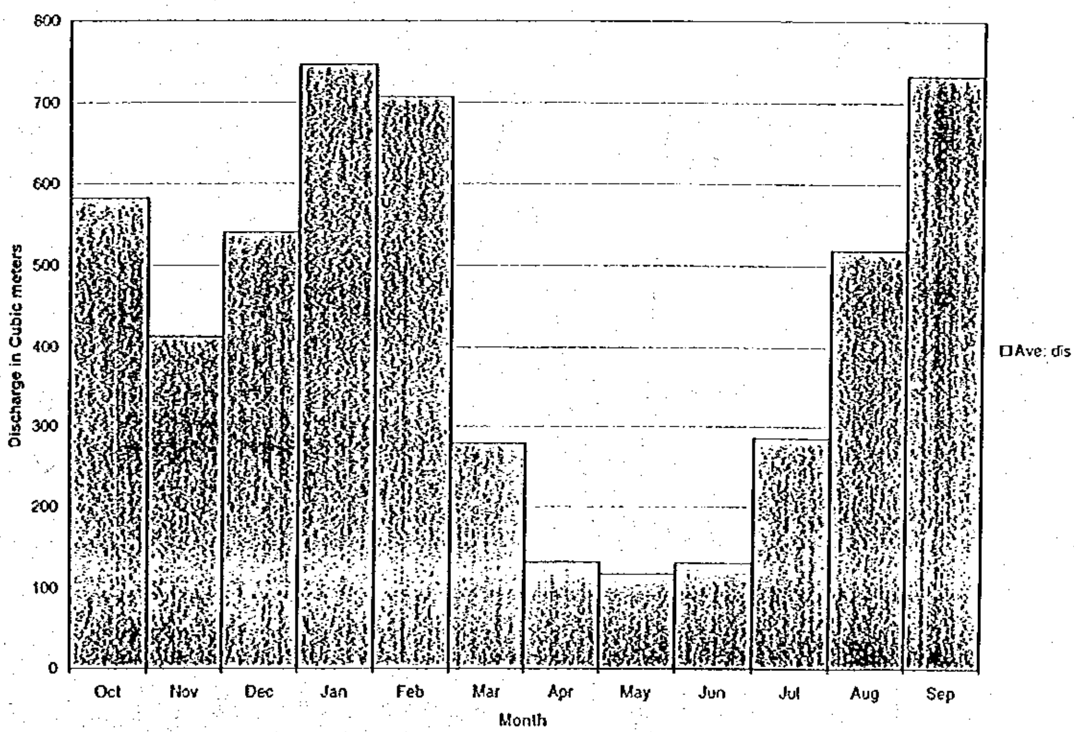


Figure 6.2.35 Mean monthly stream flow pattern at Hanwella gauging station during 1972 to 1997

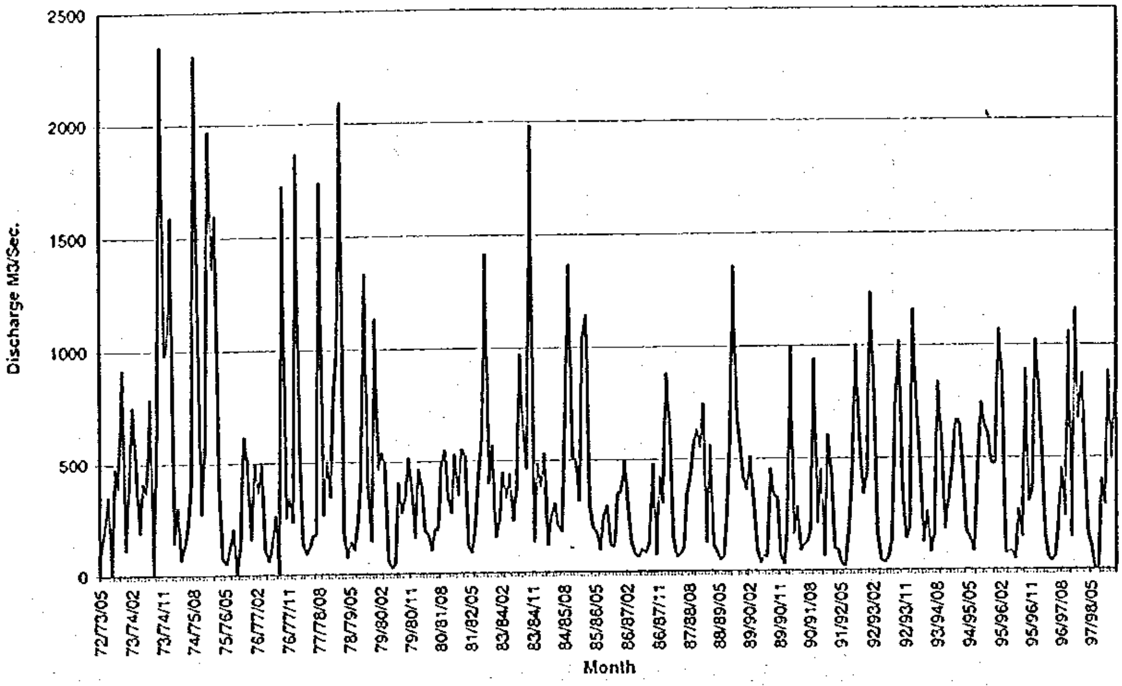


Figure 6.2.36 Mean monthly discharge at Hanwella

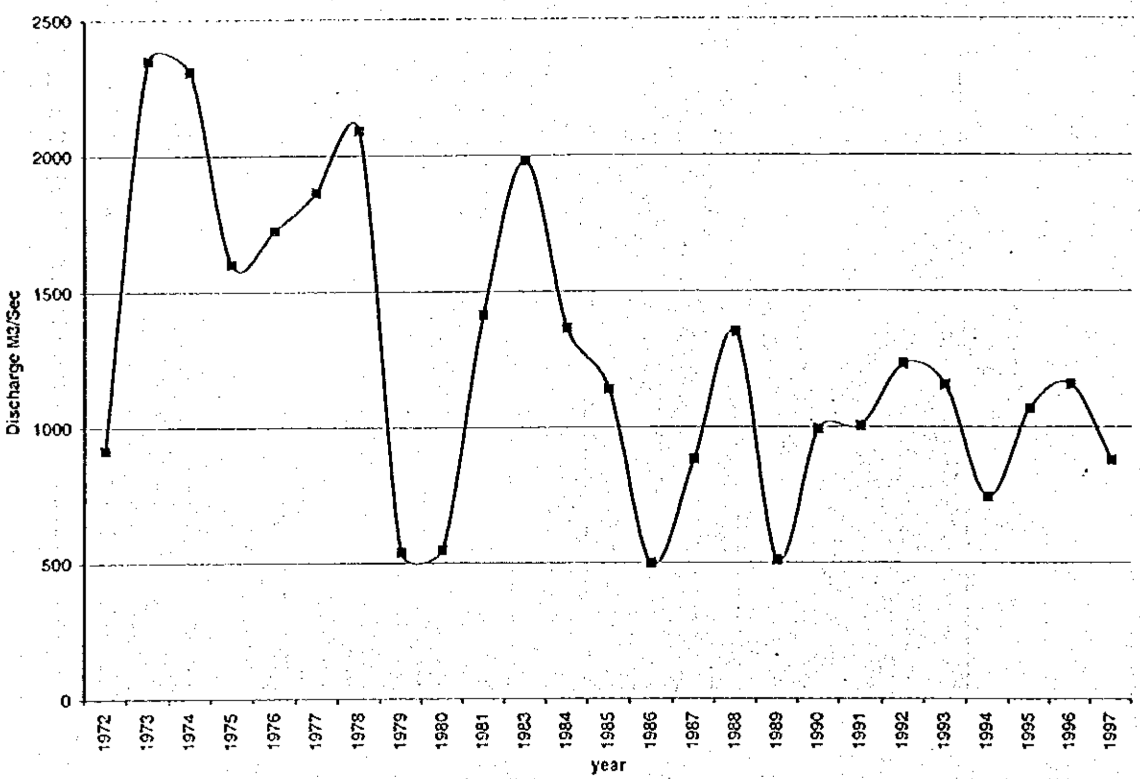
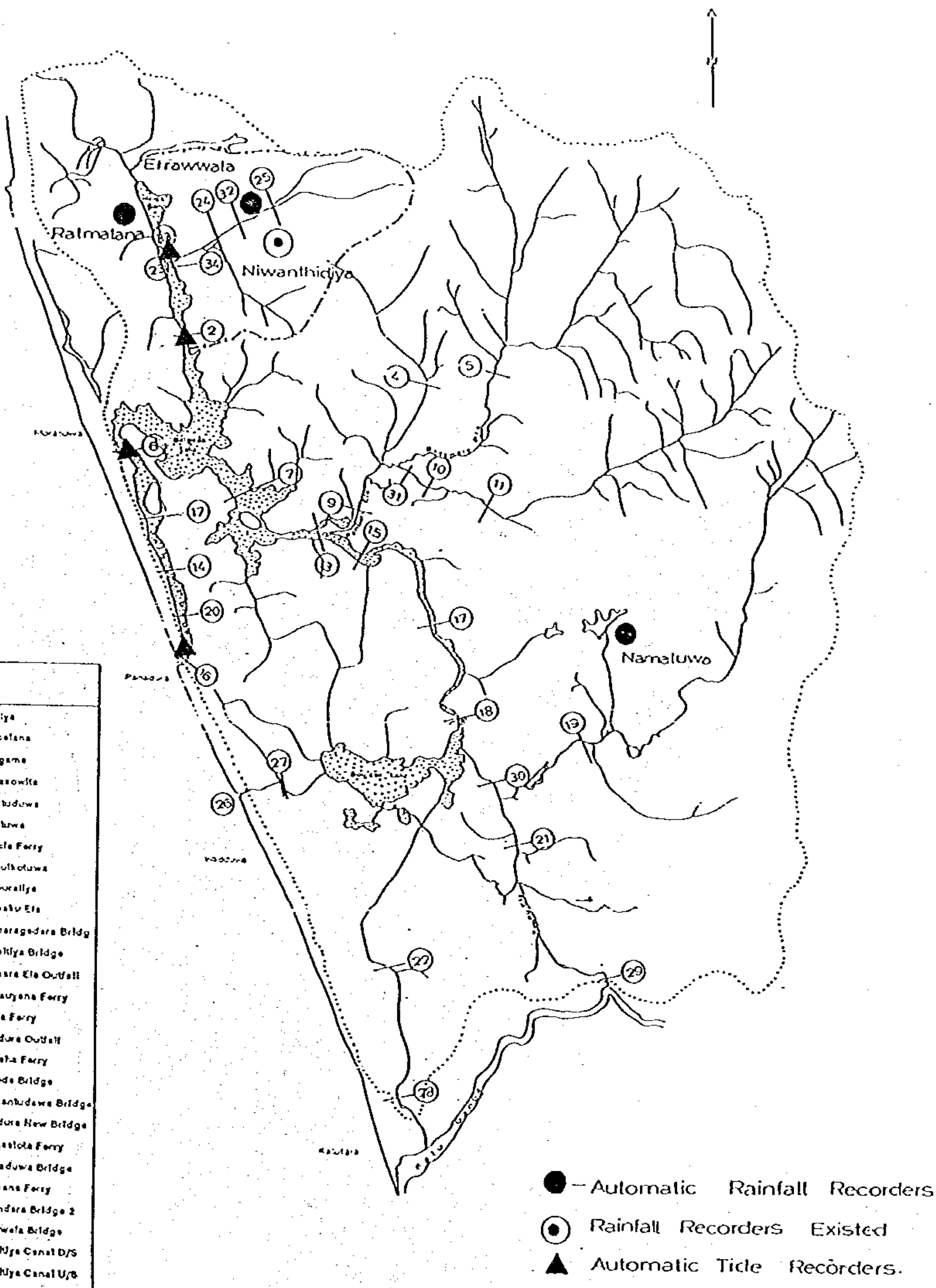


Figure 6.2.37 Annual maximum discharge at Hanwella



St. No.	Name
1	Attidiya
2	Koopelana
3	Olyagama
4	Polgasowita
5	Kahatuduwa
6	Moratuwa
7	Diggala Ferry
8	Kimbulkotuwa
9	Jamburaliya
10	Wannaku Ela
11	Bambaragedara Bridg
12	Halapitiya Bridge
13	Rambara Ela Outfall
14	Egodauyana Ferry
15	Hirana Ferry
16	Panadura Outfall
17	Rukgaha Ferry
18	Bolgoda Bridge
19	Kimmantudawa Bridge
20	Panadura New Bridge
21	Paragastota Ferry
22	Waskaduwa Bridge
23	Borupana Ferry
24	Bokundara Bridge 2
25	Erawwala Bridge
26	Thalpitiya Canal D/S
27	Thalpitiya Canal U/S
28	Dediyawala Aluth Ela
29	Keppu Ela Lock Gate
30	Gelfuda Bridge
31	Palagama Anicut
32	Bokundara Bridge 2
33	Outlet 1-Test Basin
34	Outlet 2-Test Basin

Figure 6.2.38 Details of hydrometric stations of Bolgoda basin

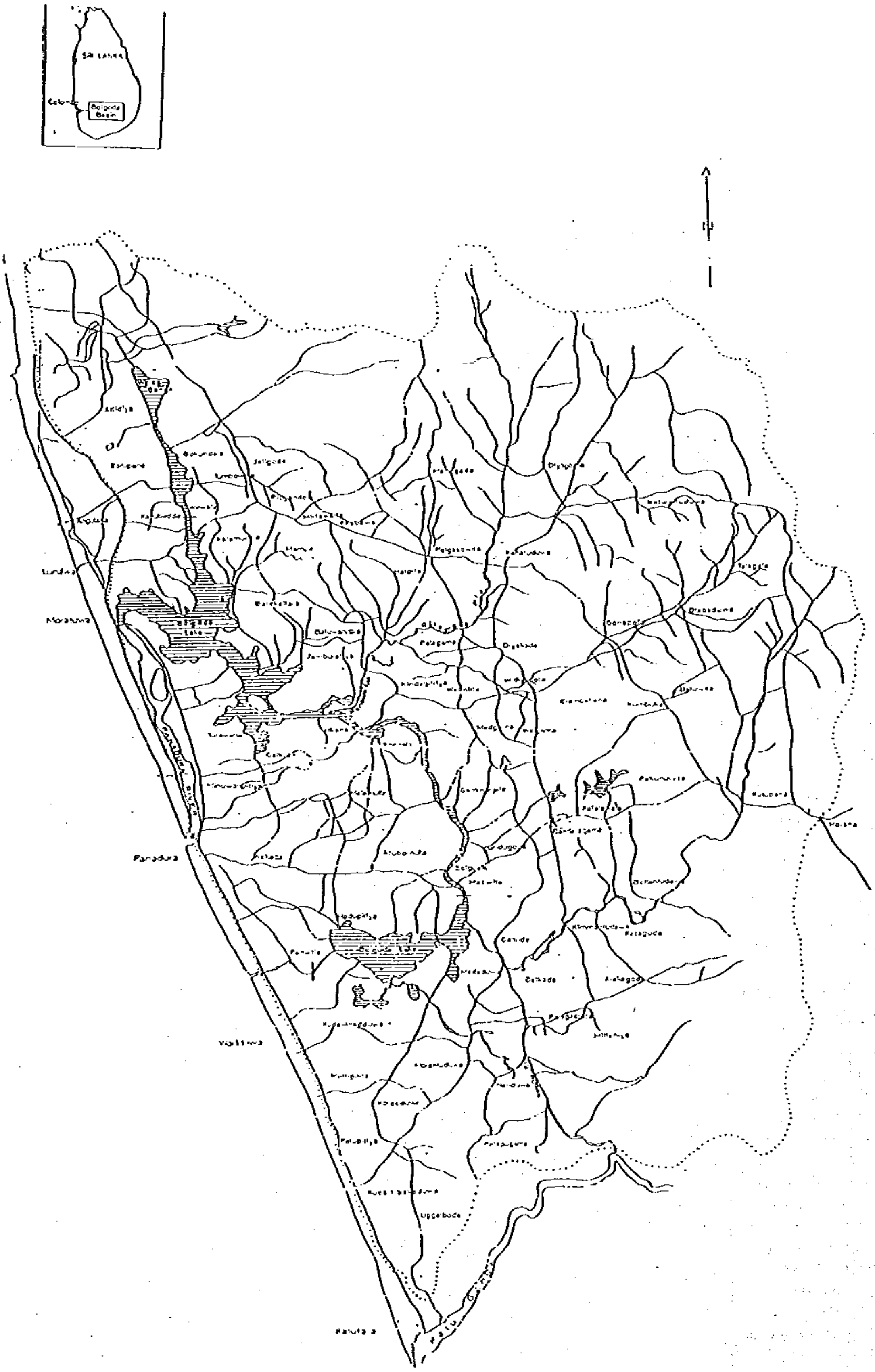


Figure 6.2.39 Catchments area of Bolgoda basin

CHAPTER 7

THE OCH CONCEPT

CHAPTER 7 THE OCH CONCEPT

7.1 Objective

The objective of this chapter is to describe the concept, functions, and physical structure of the Outer Circular Highway (OCH).

7.2 Concept for the OCH

One of the biggest transportation issues facing Colombo is the lack of an orbital or outer ring road. The purpose of such a road in the CMR would be to encourage the development of current or future growth centers, to connect radial routes, and lastly to divert through traffic from the center of the city. This is in line with the stance of the Urban Development Authority (UDA) and the Sri Lankan Government to shift some of the city's core urban functions and population to the outer suburbs in order to reduce congestion and control urban sprawl. In regards to this goal, the OCH is crucial, since it connects many of the present and future growth centers that the UDA envisions in Colombo, which would attract traffic away from the center of Colombo and reduce congestion by creating a multi-core urban structure instead of a uni-core structure (see Fig. 7.1).

Moreover, the OCH could act as a 'Green Belt' by delineating the area inside the ring as a place for planned concentrated urban growth and the area outside the ring as a green area; thereby, serving as a barrier to uncontrolled growth and the concomitant problems of congestion, increased transportation costs, rising automobile emissions, etc. It should be noted that uncontrolled growth also makes the systematic planning of infrastructure and facilities difficult; thereby, preventing efficient public investment. In fact, many cities in the UK such as London, as well as an increasing number of cities in the US (e.g., Seattle), have required the establishment of green belts to prevent sprawl. Finally, the OCH would provide more convenience for CMR residents by improving the inter-connectivity of all the major radial routes of the network, which would include the linking up of the Katunayake Expressway and the Southern Transport Corridor.

Based on the needs of the CMR described above, one of the major functions of the OCH will be the servicing of relatively short-distance trips between existing and future growth centers and the connection of existing radial routes. Given this, it does not seem advisable for the OCH to be a fully-controlled facility like the Katunayake Expressway. On the other hand, it is crucial for the OCH to maintain relatively high speeds if it is to properly fulfill the functions mentioned above. It is suggested, therefore, that the OCH be constructed to a standard somewhere between an arterial road and an expressway.

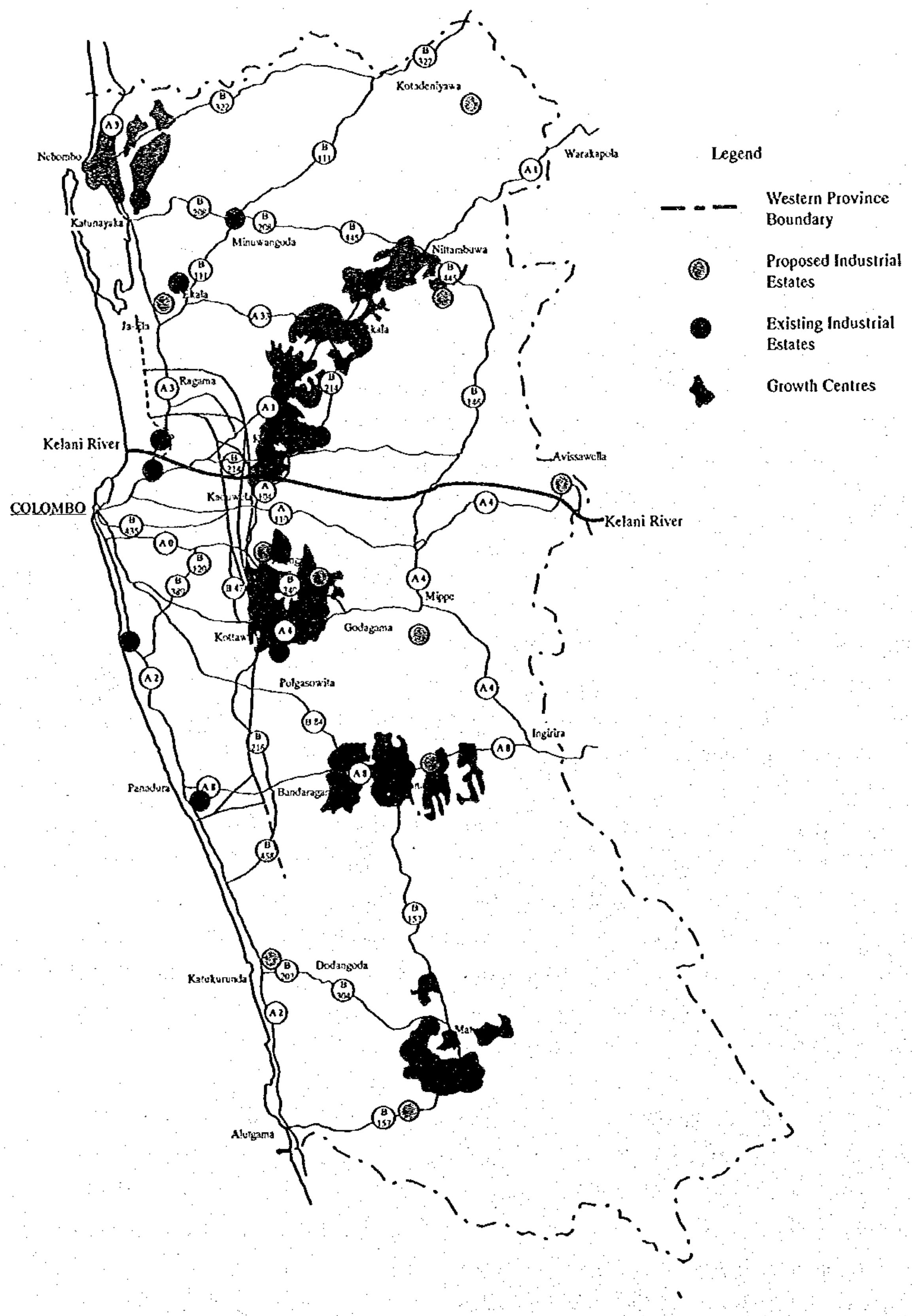


Fig. 7.1 Future Urban & Industrial Development and the OCH Alignments

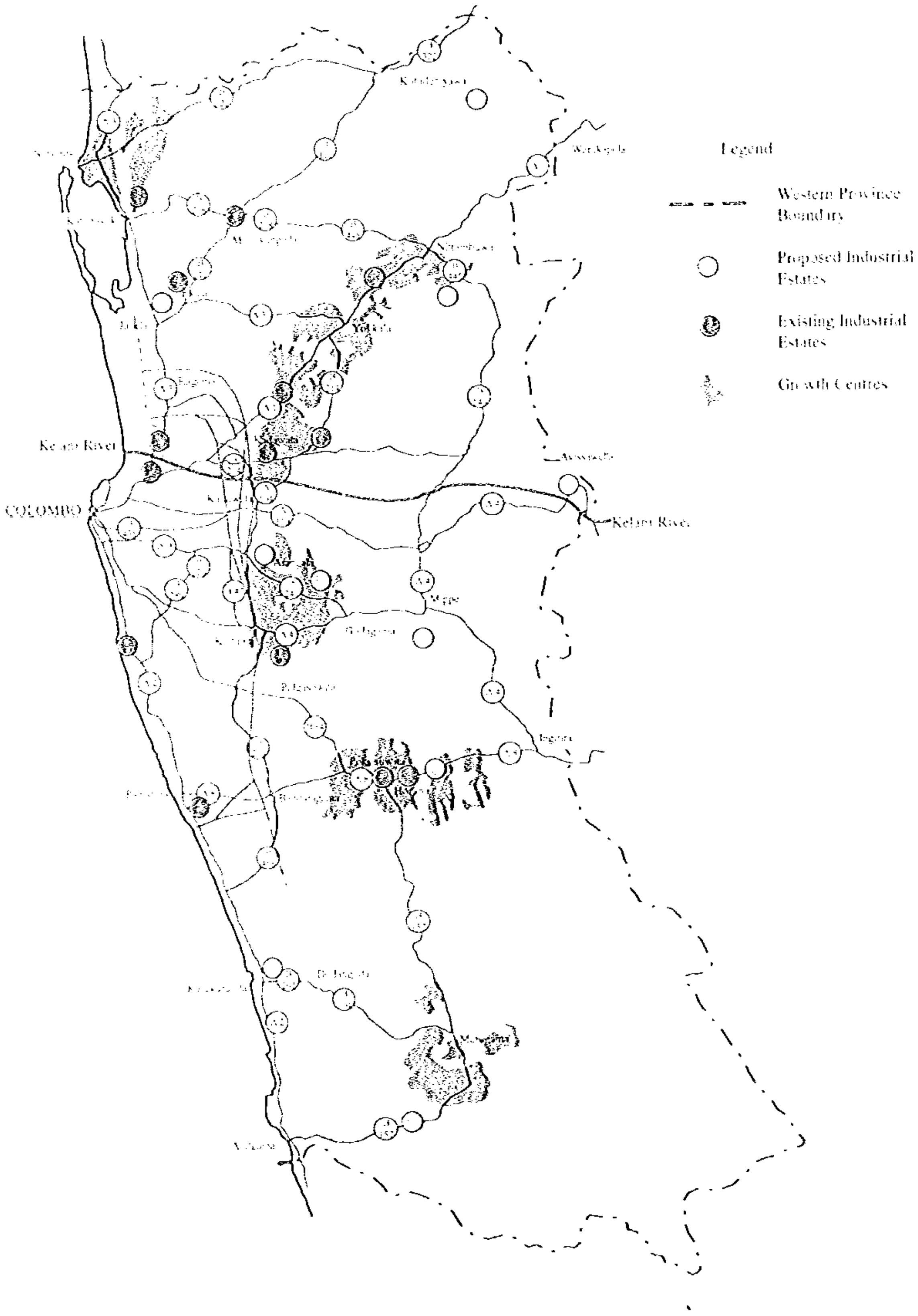


Fig. 7.1 Future Urban & Industrial Development and the OCH Alignments

7.3 Functions of the OCH

Given the above suggestion, it is realistically impossible for the OCH to be a toll road, since a toll road must be a fully-controlled facility in order to collect tolls from users. This, of course, would defeat the main purpose of the OCH, which is to provide quick and easy access within Colombo. As Tab. 7.1 shows below, a partially-controlled facility with levels of service between an arterial road and an expressway (e.g., a toll way) would have the functions needed to provide the access required together with sufficient travel speeds for the people of the CMR. To ensure that travel speeds are maintained at acceptable levels, buses would not be allowed to stop on the OCH facilities and bus stops would be constructed at or near interchanges.

Tab. 7.1 The OCH Types of Urban Roads and Their Functions

Type Of Road	Access Function	Traffic Function	Spatial Function	Design Speed
Freeway (fully-controlled facility)	×	◎	○	High Speed ↓
Partially-controlled facility	○	◎	○	
Arterial	◎	◎	◎	
Collector Road	◎	◎	◎	
Local Street	◎	○	◎	Low Speed

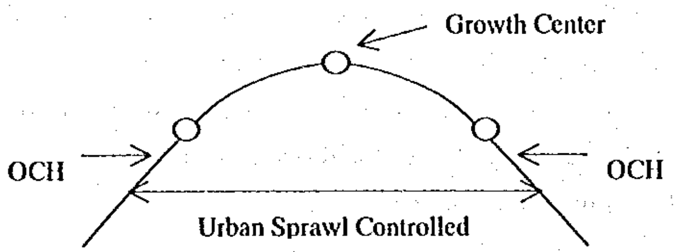
Source: Adapted from The Planning & Design of Urban Roads, Traffic Engineering Society, Tokyo, 1988.
◎,○,× indicates the level of importance in that order.

Another important reason for not assigning the function of a toll road to the OCH is that it would result in fewer drivers using it, producing greater congestion throughout urban Colombo. Even if tolls were set at a low level, the social cost in terms of waiting times at toll booths would be larger than the actual tolls collected. In fact, under non-hypercongested conditions, the toll paid by users is greater than the valuation of their savings in time. Moreover, those who are forced off to an inferior route or mode would also be worse off. In addition, the OCH does not meet two important criteria for becoming a toll road. They are as follows:

(1) There must be a realistic alternative route to a toll road: No such alternative orbital route exists in Colombo. Although, it is possible to use minor local roads to reach one's destination.

(2) A toll road should not be built when the main purpose is development: One of the purposes of the OCH is to encourage the development of both residential and industrial areas in the outer suburbs of CMR away form the city core.

Finally, the OCH has the crucial function of controlling urban sprawl by connecting strategically designated growth centers and providing the necessary transportation infrastructure to support them, as in accordance with the Colombo Metropolitan Regional Structure Plan (the CMRSP). The purpose of the CMRSP is to create a sustainable urban structure, which the Outer Circular Highway can play a crucial role in achieving, as shown in the conceptual sketch below.



7.4 Physical Structure for Outer Circular Highway

In order to satisfy the above-mentioned functions of the Outer Circular Highway (OCH), which include servicing trips between growth centers, connecting existing radial routes, and rerouting north-south through traffic, it is of great importance that the OCH maintain relatively high speeds as well as provide good access. Based on this, it recommended that the physical structure of the OCH have the characteristics listed below.

① Be partially-controlled: To allow for OCH access only at pre-established interchanges, with fences and bus bays at the appropriate locations.

② Be separated by a median or green area: To secure the safety of OCH users.

③ Be grade-separated: To ensure high levels of operation and the smooth flow of traffic.

④ Be equipped with a frontage road: To ensure sufficient access to the OCH as well as maintain current road and community integrity.

Based on the above concept for the OCH, the geometric design criteria for the OCH should be determined. In addition, connectivity with other projects such as the Colombo-Katunayake Expressway and the Southern Transport Corridor should be taken into account. At present, both of these projects are being undertaken separately and they each seem to be applying different geometric standards. The unification or coordination of these standards should be taken into account as well when designing the OCH.

CHAPTER 8

SELECTION OF HIGHWAY ALIGNMENT

CHAPTER 8 SELECTION OF HIGHWAY ALIGNMENT

8.1 Highway Sections

The Outer Circular Highway is approximately 40 to 50kilometers in length and is divided into three (3) highway sections as shown in Fig. 8.1 to simplify the selection of alternative alignment from a view point of terrain and land use. The details of these sections are as follows:

Section 1

This section starts from the proposed Colombo – Katunayake Expressway junction to the Colombo – Ratnapura trunk road (Route A4) approximately 2 kilometers to the north, crossing the Colombo – Puttalam Road (Route A3), Colombo – Kandy Road (Route A1) and the Kelani River on the way.

Most of this area cover lateritic high ground and rolling terrain where the land is utilized for residential, commercial and industrial purpose.

Section 2

This section runs from the Colombo – Ratnapura trunk road approximately 2 kilometers to the north and ends at the proposed Southern Transport Corridor junction ,which will intersect with the Route A4 and Route B84 (Colombo – Horana Road).and traverse paddy lands.

Section 3

This section extends from the proposed Southern Transport Corridor junction to the Colombo – Galle Road (Route A2) approximately 3 kilometers south of Panadura. It will be located in the north edge of Bolgoda Lake where is marsh.

8.2 Alternative Highway Alignments Method for Selecting

A flow chart illustrating the steps used to select the method for determining the most appropriate highway alignment is shown Fig. 8.2 and is summarized as follows:

Step 1

Determination of Possible Highway Alignments

All possible alternative alignments for consideration are located within a 10-km wide belt . The determination is based on discussions with the RDA, from the results of previous studies, and from land acquisition requirements..

Selection of Preferred Possible Highway Alignments

The possible alignments are broken down into segments with alpha-numeric designations (e.g. , a1-c1) which are then screened for suitability based on the concept of urban sustainability. Here, urban sustainability consists of three factors : economic, social and environmental sustainability. These preferred possible alignments are to be evaluated further as described in Step 2.

Step 2

Selection of Most Appropriate Highway Alignment

Alternative alignments comprising segments are compared using a rating System.

All of the parameters broken down into major categories (Engineering, Cost) are assigned an importance factor (The total for all categories must equal 100) to reflect the significance of each parameter and the relative importance among those categories.

The best alternative alignment segments is selected based on the sum of the points and judgement of an environmental view point.

It is proposed to the RDA for the economic and finance feasibility study in Step 3.

Step 3

Recommended Highway Alignment

Analyses of the economic benefits and financial return on investment are performed.

If the results are acceptable, the most appropriate highway alignment is considered to be in the recommended highway alignment and the feasible implementation plan is drawn up taking into consideration the financial capabilities of the project, its appropriate phasing, and the most cost-effective work packaging and work breakdown.

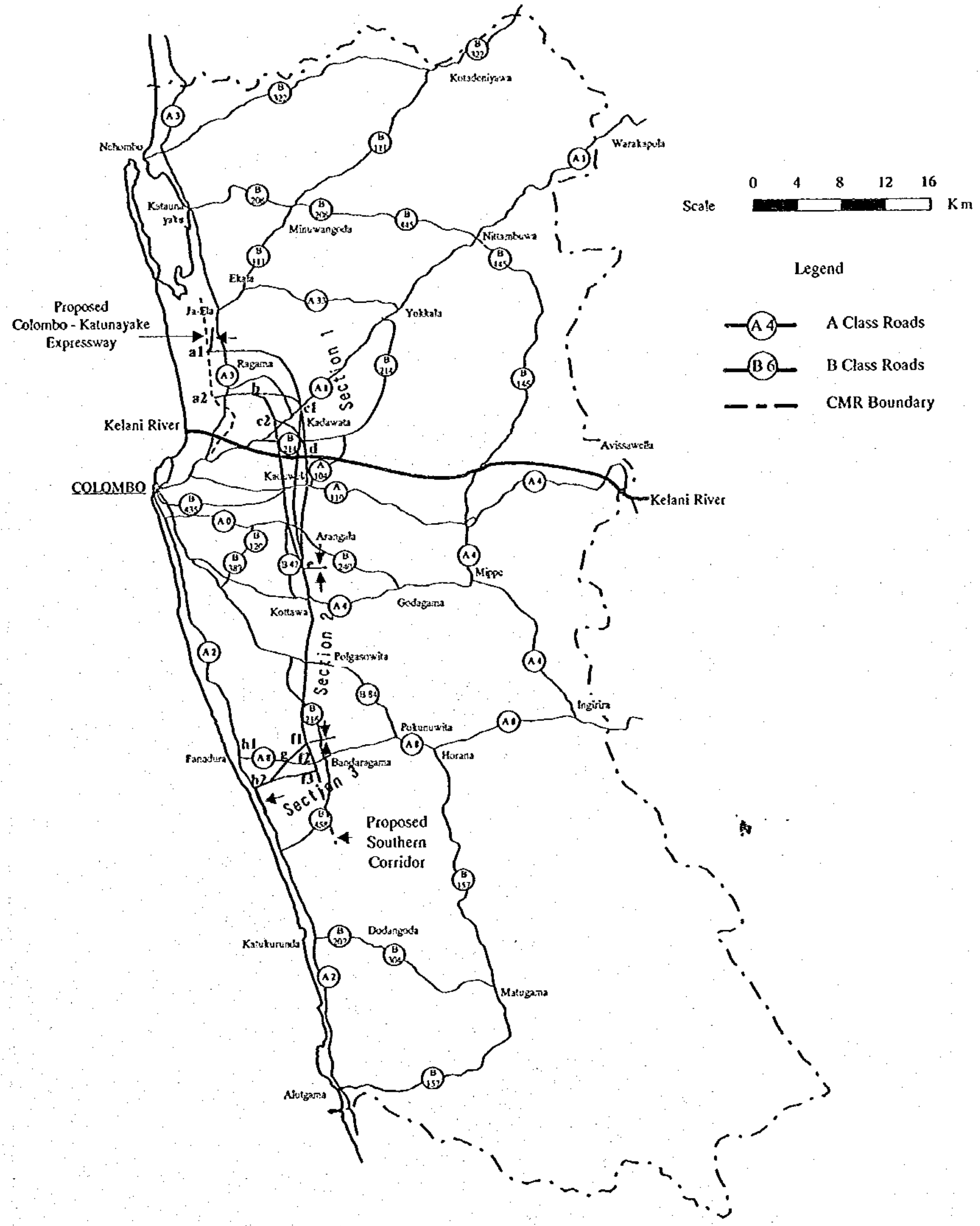


Fig 8.1 Highway Sections

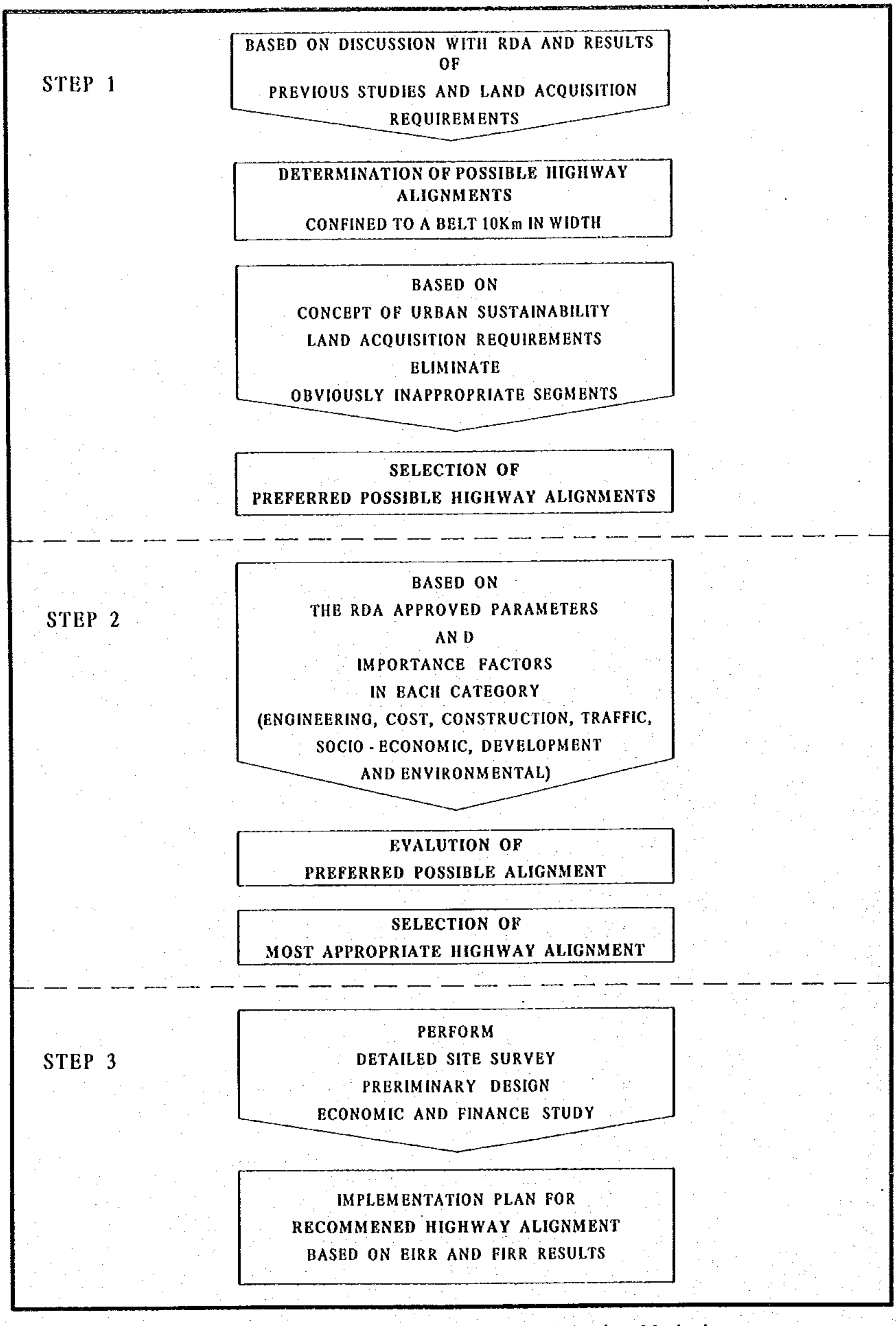


Fig.8.2 Flow Chart for Alignment Selection Method