CHAPTER 3. ROAD DESIGN

3.1 GEOMETRIC DESIGN

3.1.1 Design Standard

The Bago River Bridge was classified as a Main Arterial Road in Urban Area with 60 km/h design speed. Table 3.1.1 shows the design standards for the project road and the applied value in the design.

Table 3.1.1 Geometric Design Standards Applied to the Project

Design Element	Design Standard	Design Value in the Project
Design Speed	60 km/h	60 km/h
Radius of Curve Desirable Minimum Minimum Absolute Minimum	 200 m 150 m 120 m	320 m
Minimum Curve Length Desirable Minimum	 700/θ* m 100 m	150.231 m
Minimum Length of Transition Curve	50 m	51.200 m
Minimum Radius to Omit Transition Curve Desirable Minimum	 1,000 m 500 m	2,000 m
Maximum Grade Desirable Absolute Maximum	 5% 7%	3.000%
Minimum Vertical Curve Radius Crest Desirable Absolute Minimum Sag Desirable Absolute Minimum	 2,000 m (K=20) 1,400 m (K=14) 1,500 m (K=15) 1,000 m (K=10)	 4,400 m 1,900 m
Minimum Length of Vertical Curve	50 m	50 m
Normal Cross Slope	2.0%	2.0%
Superelevation Radius of Curve 120 ≤ R < 150 150 ≤ R < 190 190 ≤ R < 230 230 ≤ R < 270 270 ≤ R < 330 330 ≤ R < 420 420 ≤ R < 560 560 ≤ R < 800 800 ≤ R < 2000	 10% 9% 8% 7% 6% 5% 4% 3% 2%	
Minimum Radius of Curve without Superelevation	2,000 m	2,000 m
Maximum Compound Grade	10.5%	6.2%
Minimum Sight Distance Stopping Sight Distance Passing Sight Distance for Dual 1-lane Road Only Desirable Minimum	 75 m 350 m 250 m	 94.008 m not applicable
Vertical Clearance	5.000 m	5.000 m/5.500 m

Source: ASEAN Highway Standards and Japanese Road Structure Ordinance
*Remark *: θ is an intersecting angle. When θ is less than 2°, θ is applied as 2°.*

The Project was planned to have approach roads from the Star City Area to the project road, and between the intersection of Shukhinthar Mayopat Road with Thanlyin Chin Kat Road and the toll plaza of the project road. These approach roads were designed applying the design standards for ramps. Table 3.1.2 shows the design standards and the design value in the Project.

Table 3.1.2 Geometric Design Standards of Ramps

Design Element	Design Standard	Design Value in the Project
Ramp Design Speed	30 km/h	30 km/h
Radius of Curve Desirable Minimum Absolute Minimum	 30 m 20 m	 58 m
Minimum Parameter of Transition Curve	20 m	50 m
Minimum Radius to Omit Transition Curve	140 m	140 m
Maximum Grade Desirable Absolute Maximum	 9.0% 10.0%	 5.479%
Vertical Curve Minimum Vertical Curve Radius Crest Sag Minimum Vertical Curve Length	 250 m 250 m 25 m	 1000 m 1200 m 30 m
Normal Cross Slope	2.0%	2.0%
Superelevation Radius of Curve R < 50 50 ≤ R < 70 70 ≤ R < 90 90 ≤ R < 130 130 ≤ R < 160 160 ≤ R < 210 210 ≤ R < 280 280 ≤ R < 400 400 ≤ R < 800	 10% 9% 8% 7% 6% 5% 4% 3% 2%	
Maximum Combined Grade	12.0%	10.537%
Minimum Stopping Sight Distance	30 m	41.689 m

Source: Japanese Road Structure Ordinance

The design of the entry point of the approach road from the Star City Area into the Bago Bridge through lanes (on-ramp) was carried out referring to the design standards for ramp terminal. Table 3.1.3 gives the design standards and design value in the Project.

Table 3.1.3 Geometric Design Standards of Ramp Terminals

Design Element	Design Standard	Design Value in the Project
Through Lanes' Design Speed	60 km/h	60 km/h
Off-ramp Minimum Radius of Curve at the Nose Section Parameter of transition curve at the nose section Desirable Minimum	 100 m 50 m	 not applicable not applicable

Design Element	Design Standard	Design Value in the Project
Absolute Minimum	40 m	
Vertical Curve of Ramps near Nose Section Vertical Curve Radius Crest Curve Sag Curve	 450 m 450 m	 1,800 m -
Length of Speed-Change Lane Deceleration Lane Standard Length of Deceleration Lane /1 Standard Length of Taper /2 Divergence Angle /3 Acceleration Lane Standard Length of Acceleration Lane /1 Standard Length of Taper /2	 70 m 45 m 1/15 ~ 1/20 120 m 45 m	 not applicable 144 m (150 m) 54 m (104 m)

Adjustment Factor for Speed-Change Lane Length by the Through Lane's Vertical Grade				
Average Grade of Through Lane (%)	$0 < i \leq 2$	$2 < i \leq 3$	$3 < i \leq 4$	$4 < i$
Factor for Descending Deceleration Lane	1.00	1.10	1.20	1.30
Factor for Ascending Acceleration Lane	1.00	1.20	1.30	1.40

Source: Japanese Road Structure Ordinance

Remark /1: excluding taper

/2: for parallel type speed-change lane design

/3: for tapered type speed-change lane design

As the acceleration lane and taper of the approach road from the Star City Area to the Bago River Bridge are located in the +2.5% vertical alignment section, the adjustment factor of 1.20 shall be applied to the ascending acceleration lane and taper lengths. Thus the required lengths are calculated as follows:

Adjusted acceleration length= 120×1.2 = 144 m

Adjusted taper length = 45×1.2 = 54 m

3.1.2 Typical Cross Section

The project road was designed as a dual two-lane highway with 3.50 m wide carriageways, except for the flyover section above Thanlyin Chin Kat Road where the project road is a dual one-lane highway.

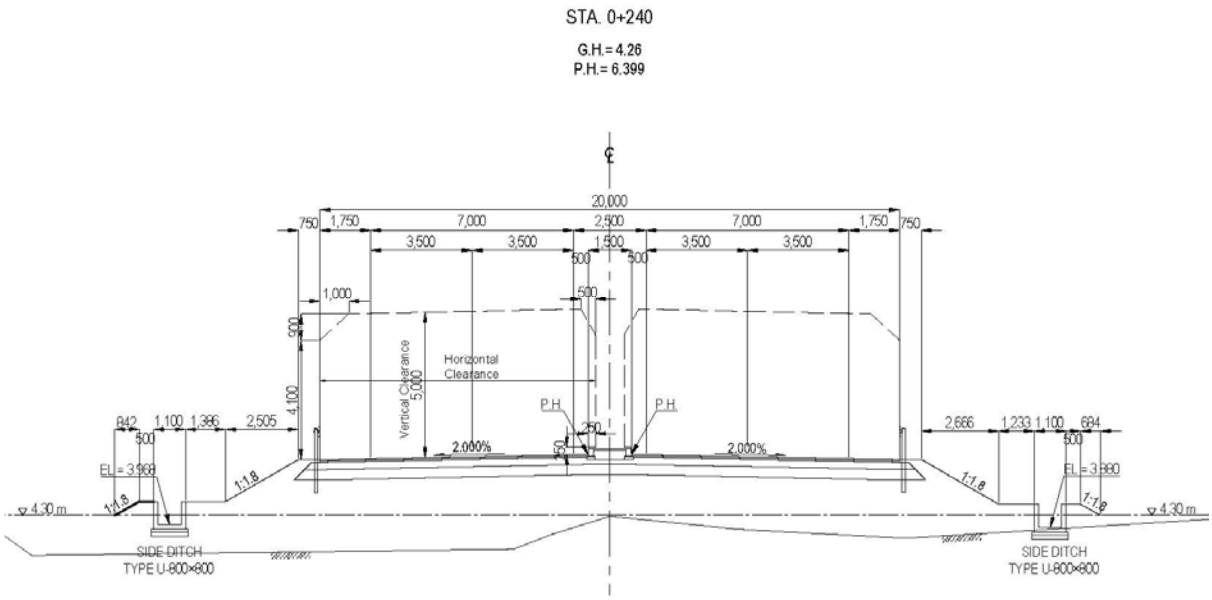
The cross section elements of the project road consist of median, inner shoulder, carriageways, and outer shoulder. Due to the design conditions of the bridge/flyover, the median width has some variations.

Table 3.1.4 Cross Section Elements of the Project Road

Cross Section Element	Width
Median Flyover Section	 0.750 m
Earthwork Section, and Steel Box Girder Bridge/PC Precast Box Girder Bridge Section	1.500 m
Steel Cable Stayed Bridge Section	3.700 m
Inner Shoulder	0.500 m
Two-lane Carriageway	2@3.5000 = 7.000 m
Outer Shoulder Earthwork Section in Package 1 Other Sections	 1.750 m 1.500 m

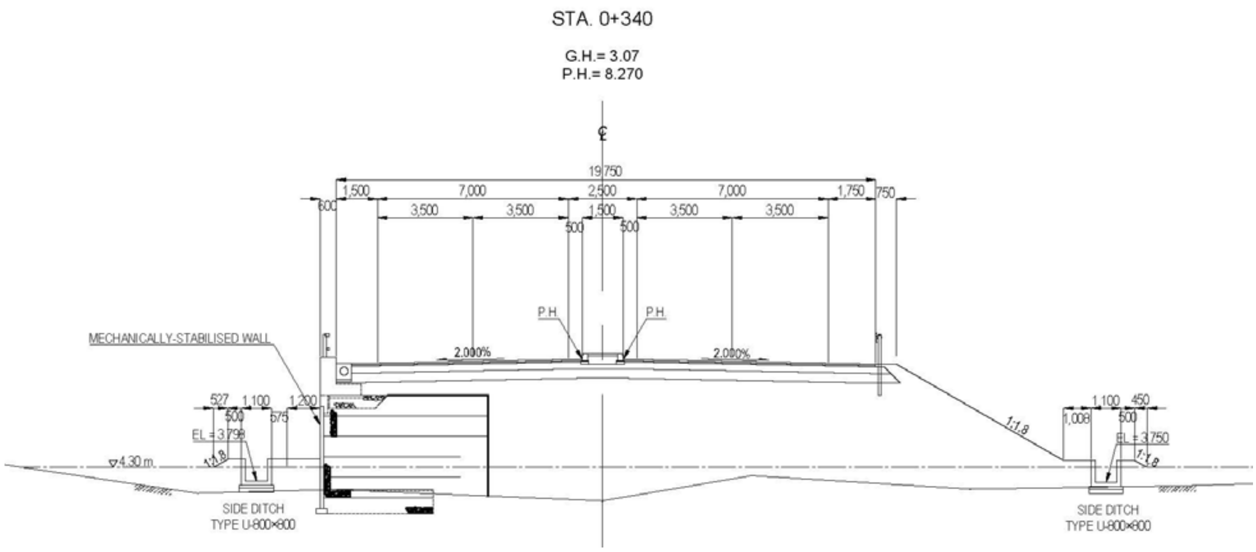
Source: JICA Study Team

Figure 3.1.1 to Figure 3.1.3 show the applied typical cross sections in the Project.



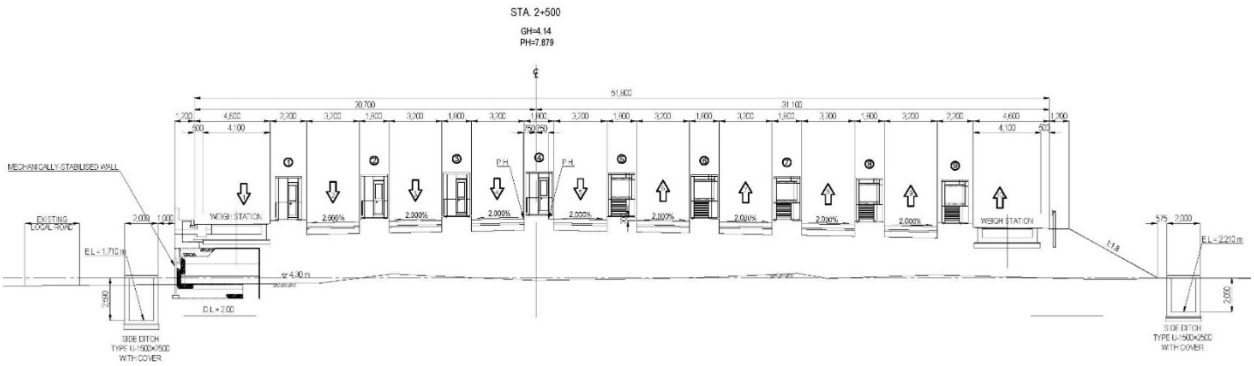
Source: JICA Study Team

Figure 3.1.1 Typical Cross Section of Earthwork Section in Package 1



Source: JICA Study Team

Figure 3.1.2 Typical Cross Section of Earthwork Section in Package 1 with Mechanically-stabilized Wall at the Left Side



Source: JICA Study Team

Figure 3.1.3 Typical Cross Section of Toll Plaza Area in Package 2 with Mechanically-stabilized Wall at the Left Side

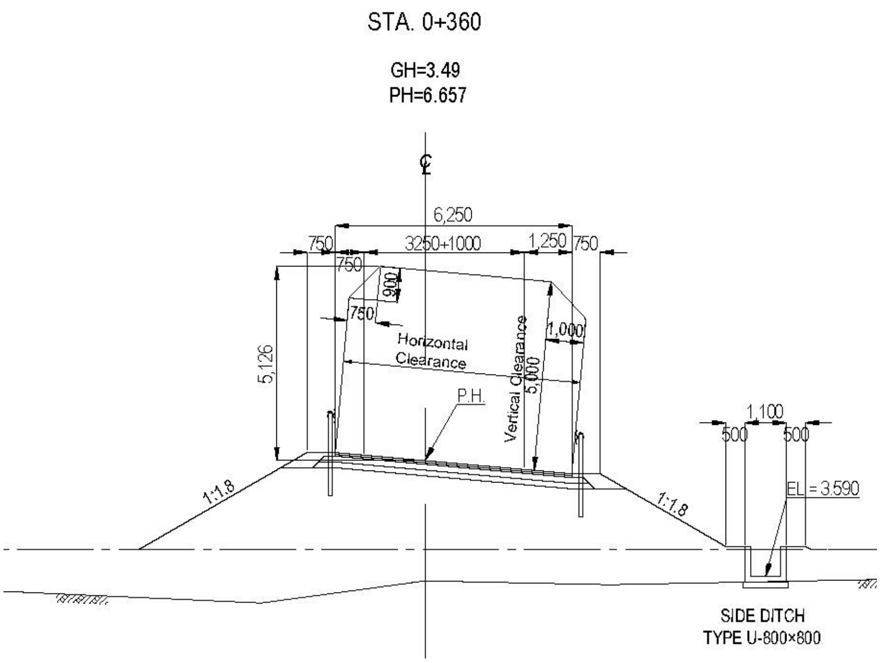
The approach road from the Star City Area to the project road was designed as one-lane ramp with cross section elements given in Table 3.1.3 .

Table 3.1.5 Cross Section Elements of the Ramp from the Star City Area

Cross Section Element	Width
Inner Shoulder	0.750 m
One-lane Carriageway	3.250 m
Outer Shoulder	1.250 m

Source: JICA Study Team

Figure 3.1.4 shows the typical cross section of the approach road in the circular curve (R = 58.0 m) section. In accordance with the design standards, the radius of R = 58.0 m requires widening of 1.0 m and superelevation of 9.0%. The height of 5.126 m from P.H. (proposed height) represents the required vertical clearance of 5.0 m in the 9.0% superelevation section.



Source: JICA Study Team

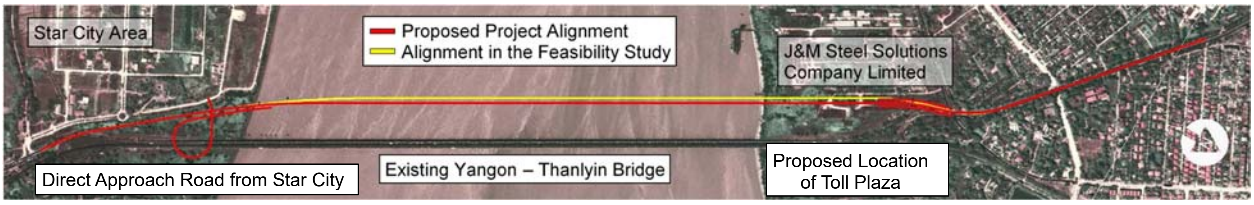
Figure 3.1.4 Typical Cross Section of Approach Road from the Star City Area

3.1.3 Road Alignment of Main Route

The Supplemental Survey for the Project for the Construction of Bago River Bridge (2016) amended the original project scope (2014) by adding the toll collection facilities (toll plaza) at the right bank side of Bago River and flyover section above Thanlyin Chin Kat Road.

Because of the introduction of the toll plaza which requires wider project land than the normal roadway section, the land acquisition of the J&M Steel Solutions Company Limited area will be required if the project alignment is not adjusted from the original plan proposed in the feasibility study.

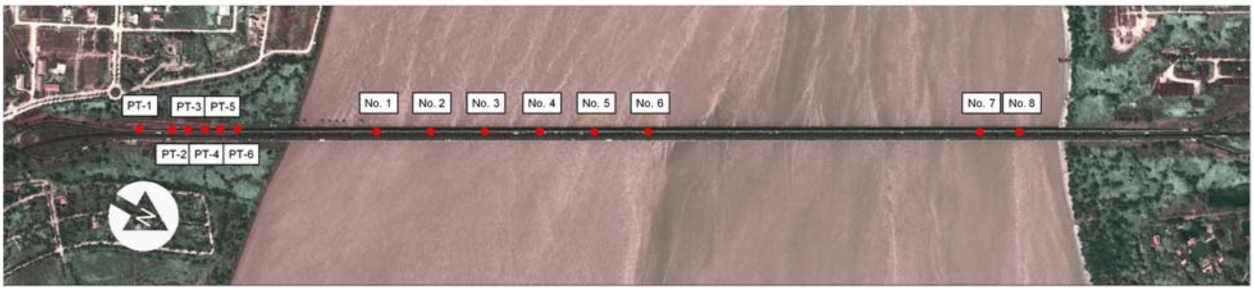
To minimize the required land acquisition in the J&M area, the centerline above the Bago River section was adjusted by shifting it 15 m upstream. With this adjustment, the J&M area and the local road along the J&M area will not be touched by the Project. Figure 3.1.5 shows the proposed alignment of the Project (red line) and the original alignment in the Feasibility Study (yellow line).



Source: JICA Study Team

Figure 3.1.5 Alignment of the Main Road

The navigation clearance on the Bago River is the important design control of the Project. The maintained navigation clearance under the existing Yangon – Thanlyin Bridge shall also be kept by the Bago Bridge. In order not to reduce the navigation clearance under the Bago Bridge, the soffit level of the existing Yangon – Thanlyin Bridge was surveyed. Figure 3.1.6 shows the survey locations and the surveyed existing soffit levels. It is noted that points No. 1 to No. 6 indicate the spans equipped with navigation signs on both sides. The proposed height of the Project (vertical alignment) was designed to maintain the surveyed height with around 50 cm allowance at the soffit level of Bago Bridge.



No.	1	2	3	4	5	6	7	8
Elevation	13.232	13.150	13.174	13.174	13.152	13.164	11.659	11.338
Easting	205372.930	205316.840	205260.784	205203.776	205147.730	205091.693	204749.172	204708.346
Northing	1857890.014	1857987.121	1858084.086	1858182.774	1858279.760	1858376.789	1858970.059	1859040.738

No.	PT-1	PT-2	PT-3	PT-4	PT-5	PT-6
Elevation	7.594	9.781	10.711	11.431	12.680	13.150
Easting	205612.913	205579.724	205564.310	205545.627	205529.867	205511.600
Northing	1857463.330	1857521.737	1857551.111	1857580.640	1857607.410	1857638.140

Source: JICA Study Team

Figure 3.1.6 Surveyed Location and Soffit Level of Existing Yangon - Thanlyin Bridge

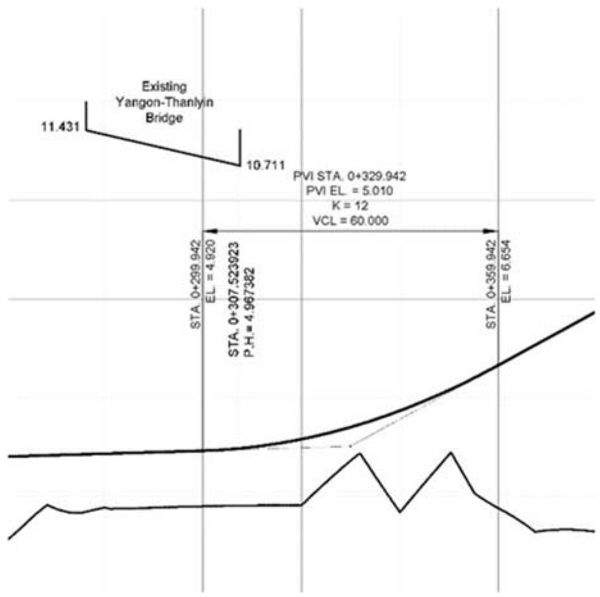
Points PT-1 to PT-6 were surveyed to check the vertical clearance required for the loop-type approach road from the Star City Area to Bago River Bridge when the road crosses under the existing Yangon – Thanlyin Bridge.

The design data of horizontal alignment and vertical alignment of the project roads are provided in the Design Report for Road Design.

3.1.4 Road Alignment of On-ramp

According to the required traffic movement from the Star City Area to Thaketa Area crossing the Bago River, the direct approach road (on-ramp) was proposed in the JICA Supplemental Survey to avoid the difficulty of traffic management in the at-grade intersection. The direct approach road from Star City has loop section with R = 58.0 m under the existing Yangon – Thanlyin Bridge, and it was provided with acceleration lane and taper section after merging with the Bago River Bridge. The outline of the road alignment at the left bank of Bago River is shown in Figure 3.1.5.

The horizontal alignment of this approach road shall cross under the existing Yangon – Thanlyin Bridge twice. It is crucial to confirm that the required vertical clearance of the approach road is secured at the crossing points under the existing bridge. As given in Figure 3.1.6, the soffit levels of the existing bridge were measured. It was confirmed that the lowest vertical clearance is 5.744 m at STA. 0+307.524 as shown in Figure 3.1.7, which satisfies the required vertical clearance of 5.126 m, as illustrated in Figure 3.1.4.

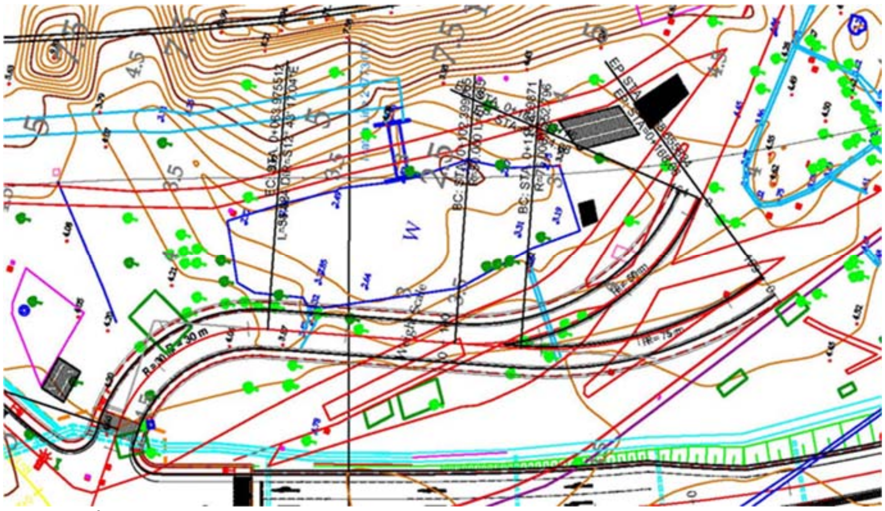


Source: JICA Study Team

Figure 3.1.7 Approach Road Profile under the Existing Bridge

3.1.5 Road Alignment of Local Approach

The existing approach roads between Shukhinthar Mayopat Road - Thanlyin Chin Kat Road intersection and the existing Yangon – Thanlyin Bridge shall be relocated due to the project construction. The proposed alignment, which follows the one proposed in the JICA Supplemental Survey, is shown in Figure 3.1.8.



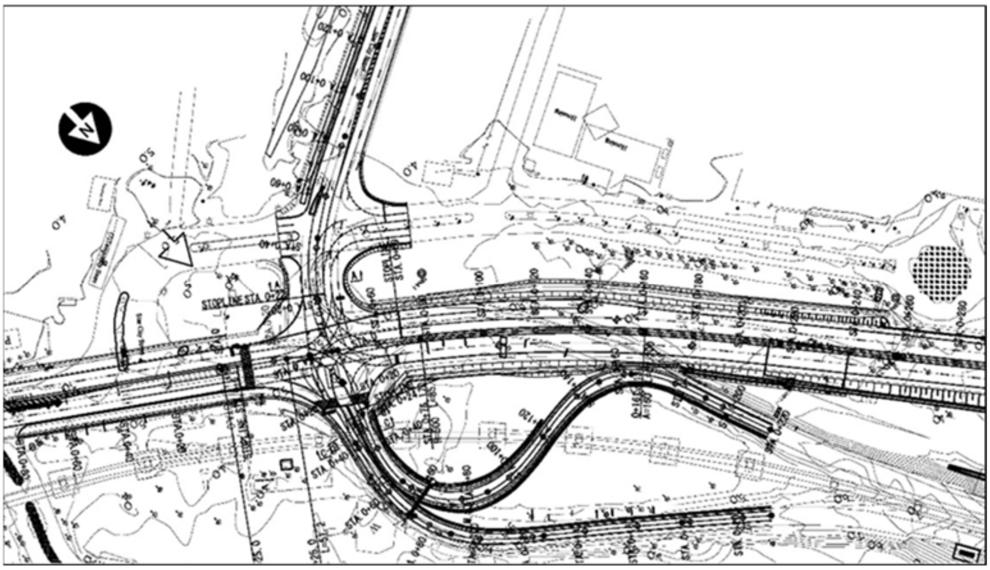
Source: JICA Study Team

Figure 3.1.8 Relocation Plan of Existing Road between Shukhinthar Mayopat Road - Thanlyin Chin Kat Road Intersection and the Existing Yangon – Thanlyin Bridge

The Thilawa Access Road Project included the improvement/upgrading of the existing intersection on Kyaik Khauk Pagoda Road at the front gate of the Star City Area. However, due to the recent construction of the new intersection in Star City, the proposed intersection improvement in the Thilawa Access Road Project was cancelled and new intersection design/construction was added in the scope of the Bago River Bridge Construction Project.

After the completion of Bago River Bridge, the Kyaik Khauk Pagoda Road - Bago River Bridge route will be the main highway route in the area. The existing approach roads, from Thanlyin side to Yangon - Thanlyin Bridge, and from Yangon - Thanlyin Bridge to Thanlyin side, shall be diverted from/merge into this new main highway route. To attain efficient traffic management, it is recommended to locate these diversion/merging point of the existing approach roads in the new intersection.

Following the above consideration, the intersection layout was proposed as given in Figure 3.1.9.



Source: JICA Study Team

Figure 3.1.9 New Intersection Layout in front of Star City Gateway

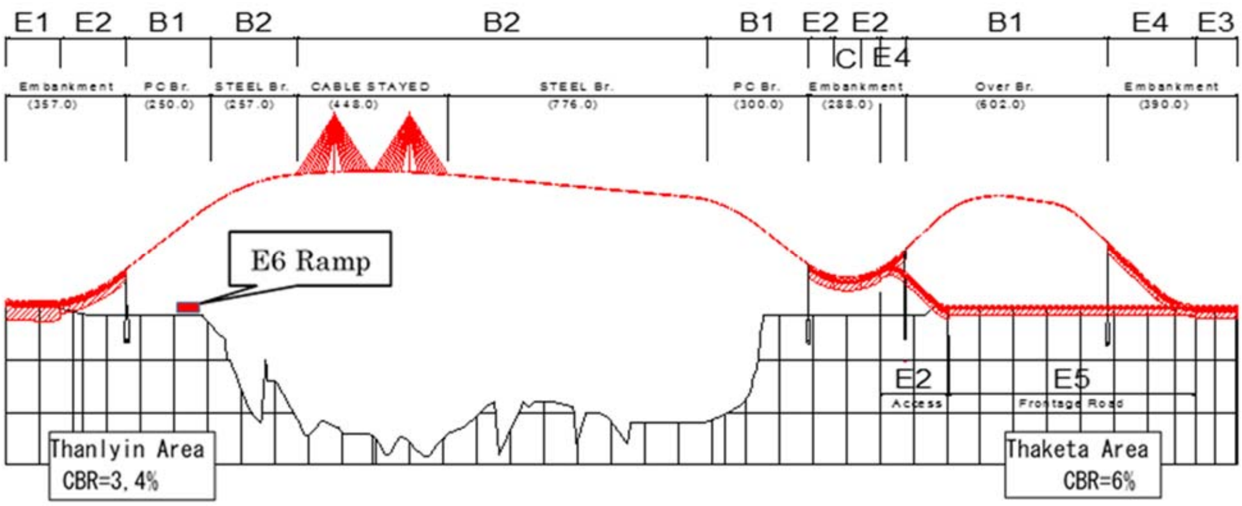
3.2 PAVEMENT DESIGN

3.2.1 Design Condition

The pavement should not only consider the traffic demand forecast but also the design that takes into consideration the bridge and embankment section. The generally required performance of pavement is as follows:

- Suitable pavement design for road structures including existing ground, embankment material, and bridge.
- Keep comfort and safety for driving.
- Keep durability to withstand vehicle load based on the traffic demand forecast.
- Select the pavement suitable for embankment and bridge structure.

Project road will be divided for suitable design of embankment and bridge section, as shown below. The embankment section is divided into seven types including concrete pavement while the bridge section is divided into two types.



Source: JICA Study Team

Figure 3.2.1 Pavement Sections

Table 3.2.1 Pavement Type

Pavement Section	Road Structure	Under the Pavement	Pavement Structures
E1, E3, E5, E6	Embankment	Cutting	Asphalt Pavement
E2, E4	Embankment	Filling	Asphalt Pavement
C	Toll Gate	Filling	Concrete Pavement
B1	PC-Box, Viaduct	RC Deck	Bridge Pavement
B2	Cable-stayed Bridge, Steel Box Girder	Steel Deck	Bridge Pavement

Source: JICA Study Team

The pavement layer designs shall be based on the "AASHOTO Guide for Design of Pavement Structures 1993" for each pavement section. This reason is used it many countries in Asia, the dimensions and weight of the vehicles can reflect actual situation in Myanmar. The pavement of the bridge section will be designed considering waterproofness, durability, and economy based on past records of Japanese bridges.

3.2.2 Design of Embankment Section

3.2.2.1 Design Method

In AASHTO, the Structural Number (SN), which is indicative of the thickness of each pavement layer, will be decided first. SN is obtained through the following equation:

$$\log_{10}W_{18} = Z_R \times S_0 + 9.36 \times \log_{10}(SN+1) - 0.20 + \frac{\log_{10}\left[\frac{\triangle PSI}{4.2 - 1.5}\right]}{0.4 + \frac{1094}{(SN+1)^{5.19}}} + 2.32 \times \log_{10}M_R - 8.07 \quad \text{(Equation 1)}$$

Where,

W_{18} = predicted number of 18-kip equivalent single axle load applications,

Z_R = standard normal deviate,

S_0 = combined standard error of the traffic prediction and performance prediction,

ΔPSI = difference between the initial design serviceability index, p0, and the design terminal

M_R = resilient modulus (psi), and

SN = structural number indicative of the total pavement thickness required.

The pavement layer is calculated by the following equation:

$$2.54 \times S_N = a_1 \times D_1 + m_2 \times a_2 \times D_2 + m_3 \times a_3 \times D_3$$

a_i = inch layer coefficient,

D_i = inch layer thickness (inches), and

m_i = inch layer drainage coefficient.

3.2.2.2 Asphalt Pavement Design

The following are the assumptions for asphalt pavement design:

(1) Performance Period

Opening year will be 2020. In the feasibility study (F/S), the traffic demand forecast was planned until 2035. In the pavement design, the target period is set to 15 years.

(2) Predicted Number of 18-kip Equivalent Single Axle Load (ESAL) Application: W_{18}

The vehicle type used for the calculation of W18 and the weight distribution of vehicle are set in consideration of the F/S report and the Thilawa Access Road.

Table 3.2.2 ESAL Equivalent Single Axle Load Pavement Type

Type od Vehicle	Total	Front			Rear 1			Rear 2			Rear 3			ESAL
	ton	ton	kip	Factor	ton	kip	Factor	ton	kip	Factor	ton	kip	Factor	
Car&Taxi、Van、	2	1	2.204	0.0002	1	2.204	0.0002							0.0004
SmallTrack,SmallBus	9.5	2	4.408	0.0060	7.5	16.53	0.8115							0.8175
LargeBus	10	2	4.408	0.0060	8	17.63	1.0000							1.0060
Track 2-axles	16	6	13.22	0.3173	10	22.04	2.1800							2.4973
Track 3-axles	23	5	11.02	0.1385	9	19.84	1.5100	9	19.84	1.5100				3.1585
Track 4-axles	34	6	13.22	0.3173	10	22.04	2.1800	9	19.84	1.5100	9	19.84	1.5100	5.5173

ESAL: Equivalency Single Axles Loads
kip : 1 ton =2.204 kip

Source: JICA Study Team

The predicted number of 18-kip Equivalent Single Axle Load application for each section is shown below.

Table 3.2.3 Predicted Number of ESAL of Each Section

Vehicle Types	Factor	E1,E2 Thanlyin Main Road Traffic	E1,E2 Thanlyin Main Road W18	E1,E2 Thanlyin Main Road Ratio	E4 Ov Approach Traffic	E4 Ov Approach W18	E4 Ov Approach Ratio	E3 Thaketa Widening Traffic	E3 Thaketa Widening W18	E3 Thaketa Widening Ratio
Car&Taxi	0.0004	73,767,595	29,507	0.1%	32,775,540	13,110	0.1%	50,225,825	20,090	0.1%
Van	0.0004	14,390,125	5,756	0.0%	7,540,535	3,016	0.0%	8,408,140	3,363	0.0%
Pass Truck & Small	0.8175	6,881,710	5,625,798	19.2%	5,947,675	4,862,224	35.0%	1,164,350	951,856	5.0%
Small Track	0.8175	6,067,395	4,960,095	16.9%	2,906,495	2,376,060	17.1%	3,888,345	3,178,722	16.7%
LargeBus	1.0060	2,130,870	2,143,655	7.3%	467,565	470,370	3.4%	2,054,585	2,066,913	10.9%
Track 2-axles	2.4973	1,825,365	4,558,393	15.5%	278,130	694,560	5.0%	1,914,060	4,779,886	25.1%
Track 3-axles	3.1585	2,559,015	8,082,649	27.5%	1,238,445	3,911,629	28.1%	1,632,645	5,156,709	27.1%
Track 4-axles	5.5173	713,940	3,938,985	13.4%	284,335	1,568,747	11.3%	521,220	2,875,701	15.1%
Total			29,344,839	100.0%		13,899,717	100.0%		19,033,241	100.0%

	Factor	E4 Access ramp Taketa Traffic	E4 Access ramp Taketa W18	E4 Access ramp Taketa Ratio	E5 Frontage Taketa Traffic	E5 Frontage Taketa W18	E5 Frontage Taketa Ratio	E6 On ramp Tanlyin Traffic	E6 On ramp Tanlyin W18	E6 On ramp Tanlyin Ratio
Car&Taxi	0.0004	40,992,055	16,397	0.1%	9,233,770	3,694	0.1%	507,350	203	0.0%
Van	0.0004	6,849,590	2,740	0.0%	1,558,550	623	0.0%	235,060	94	0.0%
Pass Truck & Small	0.8175	934,035	763,574	4.9%	230,315	188,283	5.2%	175,565	143,524	0.9%
Small Track	0.8175	3,160,900	2,584,036	16.7%	727,445	594,686	16.6%	60,955	49,831	0.3%
LargeBus	1.0060	1,663,305	1,673,285	10.8%	391,280	393,628	11.0%	0	0	0.0%
Track 2-axles	2.4973	1,547,235	3,863,833	25.0%	366,825	916,054	25.5%	43,435	108,468	0.7%
Track 3-axles	3.1585	1,320,570	4,171,020	27.0%	312,075	985,689	27.5%	25,915	81,853	0.5%
Track 4-axles	5.5173	429,605	2,370,238	15.3%	91,615	505,463	14.1%	25,915	142,980	0.9%
Total			15,445,122	100.0%		3,588,119	100.0%		526,952	3.4%

Source: JICA Study Team

(3) Standard Normal Deviate: Z_R

In AASHTO, interstates are recommended to have R=85%-99.9%. The project road will connect Yangon and Thilawa Industrial Park; therefore, R=95% is decided in order to match with the Thilawa Access Road.

The standard normal deviate is set at 1.645.

(4) Overall Standard Deviation: S_0

The overall standard deviation is set at 0.45 for flexible pavement.

(5) Initial Serviceability Index: P_0, Terminal Serviceability Index: P_t

The serviceability index is set considering harmony with the Thilawa Access Road.

Table 3.2.4 Serviceability Loss

Design Serviceability Index	
P_0	4.2
P_t	2.5
ΔPSI	1.7

Source: JICA Study Team

(6) Effective Resilient Modulus of Subgrade: M_R

The effective resilient modulus is calculated by the following equation:

M_R（Resilient Modulus）＝1500×CBR

MR is calculated based on the test result of the roadbed (CBR). The design CBR is shown below.

1) Current Ground

The results of the survey and calculation on Thaketa and Thanlyin side are shown below.

Table 3.2.5 Design CBR on Existing Ground

Item	Thanlyin Main Road (E1)		Thaketa On-Ramp (E6)		Thanlyin and Thaketa (E1,E5)		Remark
Current CBR	No.1	9	No.10	3	No.5	8	
	No.2	4	No.11	4	No.6	7	
	No.3	13 (reject)	No.12	20 (reject)	No.7	3 (reject)	
	-	-	-	-	No.8	13 (reject)	
	-	-	-	-	No.9	9	
Average	6.5		3.5		8.0		
Standard Deviation	2.5		0.5		0.9		
Section CBR	4.0		3.0		7.1		
Design CBR	4		3		6		
MR=1500 × CBR	6,000		4,500		9,000		

Source: JICA Study Team

2) Banking

The CBR of banking is calculated based on the test results of the banking survey. The banking survey investigated five locations, three of which are not suitable for banking material; therefore, CBR is set considering the test results of the two suitable locations.

Table 3.2.6 Design CBR on Banking

	CBR			Ave.	Selected CBR	Remark
Location 1 (MARGA)	11	14	15	13.0	13	
Location 2 (KO TOE)	20	17	12	18.5	18	12 are rejected
Location 3 (GREAT MOTION)	2	1	1	1.3	-	All are rejected
Location 4 (GREAT MOTION)	4	5	4	4.3	-	All are rejected
Location 5 (AUNG WIN)	34	5	4	4.0	-	All are rejected
Average					15.5	
Standard Deviation					8.0	
Section CBR					7.5	
Design CBR					6	

Source: JICA Study Team

The design CBR and effective resilient modulus of subgrade are shown in Table 3.2.7.

Table 3.2.7 Design CBR and M_R

Section	CBR	MR	Remark
Thanlyin Main Road (E1)	4	6,000	
Thanlyin On-ramp (E6)	3	4,500	
Thaketa Main Road, Frontage Road (E1, E5)	6	9,000	
Banking (E2)	6	9,000	

Source: JICA Study Team

(7) Structural Number: S_N

The structural number is shown in Table 3.2.8.

Table 3.2.8 Structural Number: S_N

Section	Thanlyin			Thaketa					
	Main		On-ramp	Main Toll Gate)	Main (Ov)	Access	Main (Ov)	Main Widenin g	Frontage
SN	5.028	4.503	3.601	4.503	4.014	4.082	4.014	4.216	3.222

Source: JICA Study Team

Pavement layers are decided through the following equation:

$$2.54 \text{ x } S_N = a_1 \text{ x } D_1 + m_2 \text{ x } a_2 \text{ x } D_2 + m_3 \text{ x } a_3 \text{ x } D_3$$

a_i = inch layer coefficient,

D_i = inch layer thickness (inches), and

m_i = inch layer drainage coefficient.

(8) Each Layer Characterization Coefficients

To plan the pavement layer, each layer characterization coefficient will be calculated. Characterization coefficients are based on AASHTO, and should consider harmony with the Thilawa Access Road.

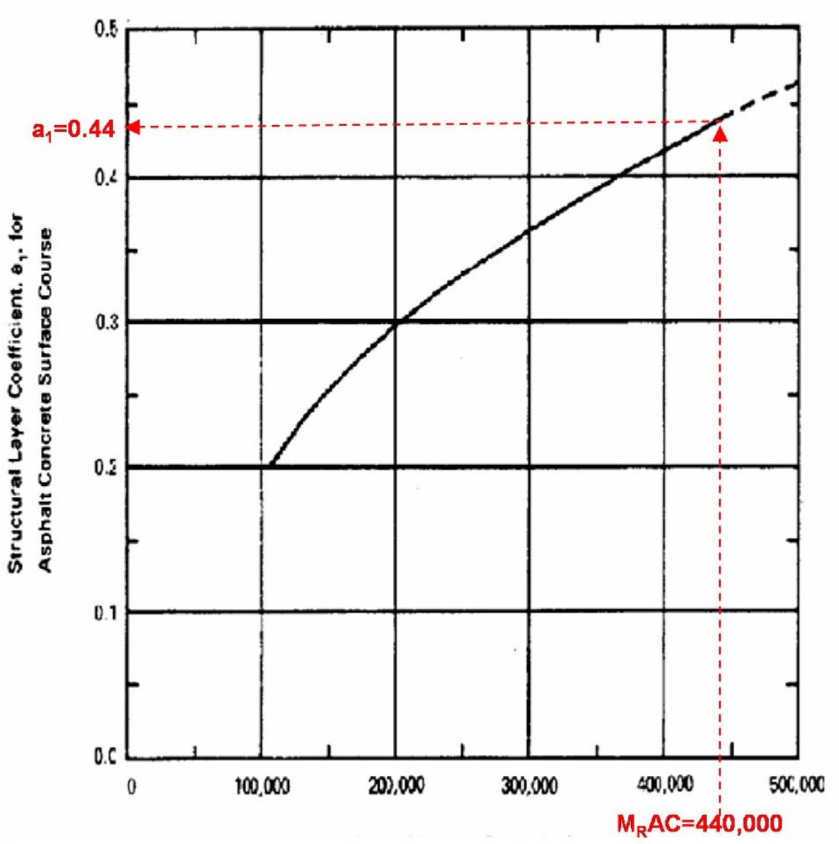
1) Asphalt (Elastic Modulus of Asphalt Concrete): E_{AC}

E_{AC} is calculated from the figure below.

In Thilawa Access Road,

E_{AC} is 440,000 psi.

a_1 is 0.44.



Source: JICA Study Team

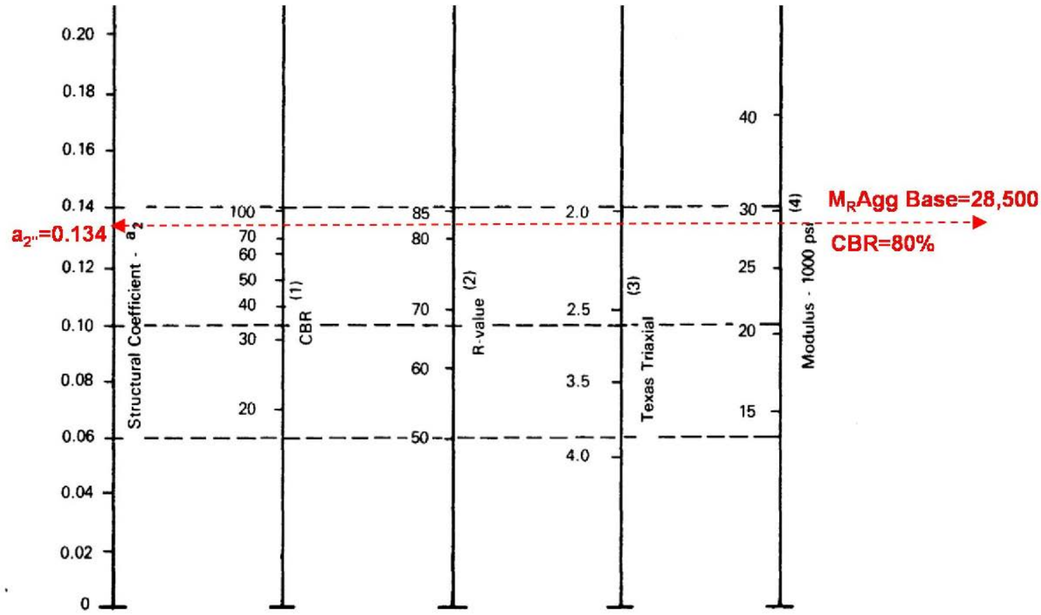
Figure 3.2.2 Elastic Modulus of Asphalt Concrete

2) Upper Subbase (Elastic Modulus of Base Course Aggregate) E_{BS}

The CBR of the upper subbase shall be equivalent to 80.

E_{BS} is 28,500 psi.

a_2 is 0.134.



Source: JICA Study Team

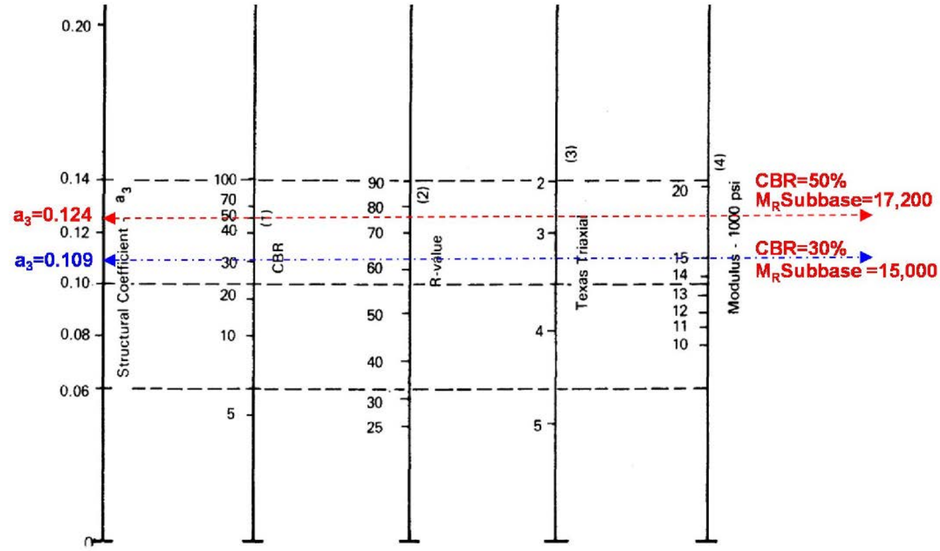
Figure 3.2.3 Elastic Modulus of Upper Subbase

3) Lower Subbase (Elastic Modulus of Subbase Course Aggregate): E_{SB}

The CBR of the lower subbase shall be equivalent to 30.

E_{SB} is 15,000 psi.

a. is 0.109.



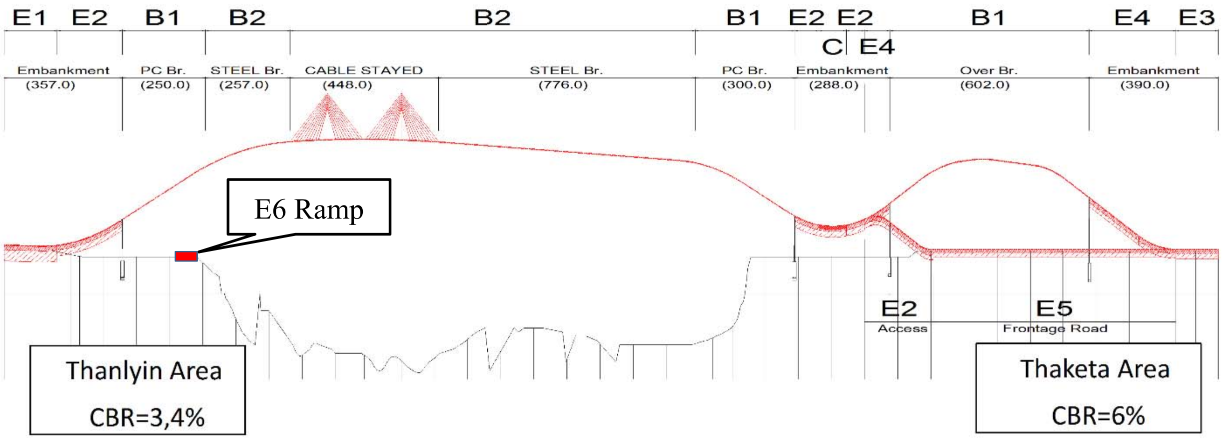
Source: JICA Study Team

Figure 3.2.4 Elastic Modulus of Lower Subbase

3.2.2.3 Pavement Layer of Embankment

The pavement layer of embankment is shown in Figure 3.2.5.

	Thanlyin			Thaketa					
Location	Approach E1	Approach E2	Ramp E6	Toll E2	Ov Approach E4	Access Ramp E4	Ov Approach E4	Widenning E3	Frontage Road E5
DesignCBR	4.0 -	6.0 -	3.0 -	6.0 –				6.0 -	6.0 -
M_R (psi)	6000.0	9000.0	4500.0	9000.0					
a_1	0.440								
a_2	0.134								
a_3	0.109								
m	1.000								
Traffic	29,344,839	29,344,839	526,952	29,344,839	13,899,717	15,445,122	13,899,717	19,033,241	3,588,119
SN	5.028	4.503	3.061	4.503	4.014	4.081	4.014	4.216	3.222
Thickness of Pavemnet Structures									
As	10	10	10	10	10	10	10	10	10
Base Course	25	25	15	25	25	25	25	25	15
SubBase	50	35	15	35	25	25	25	30	25
SubGrade	-	-	-	-	-	-	-		-
Totall	85	70	40	70	60	60	60	65	50



Source: JICA Study Team

Figure 3.2.5 Pavement Layer List

The pavement of embankment is adopted straight asphalt pavement. This is because it is difficult to secure quantitative quality of improved asphalt as a result of hearings from local contractors. And it is one of the reason that it is easy to procure local materials.

3.2.2.4 Concrete Pavement

Since stopping and starting are repeated at the toll gate, the pavement needs high resistance against flow. Concrete pavement has high resistance. The design method is based on AASHTO similar to that for asphalt pavement. In AASHTO, the thickness of concrete pavement is calculated through the following equation:

NOMOGRAPH SOLVES:

$$\log_{10}W_{18} = Z_R*S_o + 7.35*\log_{10}(D+1) - 0.06 + \frac{\log_{10}\left[\frac{\Delta PSI}{4.5-1.5}\right]}{1+\frac{1.624*10^7}{(D+1)^{8.46}}} + (4.22-0.32p_t)*\log_{10}\left[\frac{S'_c * C_d\left[D^{0.75}-1.132\right]}{215.63*J\left[D^{0.75}-\frac{18.42}{(E_c/k)^{0.25}}\right]}\right]$$

Where

W_{18} = predicted number of 18-kip equivalent single axle load application,

Z_R = standard normal deviate,

S_0 = combined standard error of the traffic prediction and performance prediction,

= 0.35 for concrete pavement

ΔPSI = difference between the initial design serviceability index, p0, and the design terminal,

D = thickness of concrete pavement (inch),

S_C' = estimated mean value for PCC modulus of rupture (psi),

Cd = drainage coefficient,

J = joint coefficient (middle value is 2.8),

E_C = elastic modulus of concrete,

$E_C = 5.0 \times 10^6$

kj = M_R / 19.4

M_R = 1,500 x CBR = 1.500 x 6 = 9,000

W18, and ZR are the same as in asphalt pavement.

As a result of the calculation, the thickness of concrete pavement is 9.1 inches (23 cm), rounded to 25 cm. The thickness of the upper subbase is set so that the total thickness which includes the lower subbase is the same as the asphalt pavement thickness.

Table 3.2.9 Pavement Thickness

<table>
<tr><th>Layer</th><th>Asphalt Pavement</th><th>Concrete Pavement</th></tr>
<tr><td rowspan="2">Surface Course ~ Upper Subbase</td><td rowspan="2">5 cm
5 cm
25 cm</td><td>25 cm</td></tr>
<tr><td>10 cm</td></tr>
<tr><td>Lower Subbase</td><td>35 cm</td><td>35 cm</td></tr>
<tr><td>Total</td><td>70 cm</td><td>70 cm</td></tr>
</table>

Source: JICA Study Team

3.2.3 Bridge Section

Generally, the pavement of the bridge section is composed of surface course and surface base. The thickness of each layer is set to the minimum value of 40 m. It is necessary for the design of the bridge pavement to consider the following items:

Waterproofing:

The durability of bridges greatly depends on the effectiveness of waterproofing of the deck. Therefore, the following are required: adhesion between deck and bituminous layer, compatibility with bituminous mixture, and resistance against high temperatures during the application of the hot asphalt mixture. Materials for waterproofing are divided into two main categories, namely, sheet type and liquid (sprayed) type. In this project, it is recommended to apply liquid type in consideration of workability.

Sealing and Bonding Waterproofing:

The pavement does not have bonding effect to the bridge deck. Therefore, an intermediate sealing layer is necessary for bonding the waterproofing layer. The function of the sealing layer on the bridge deck is strong adhesion to the RC/steel deck and the waterproofing layer.

3.2.3.1 Steel Deck Section

The steel deck section has different features from the embankment section, as follows:

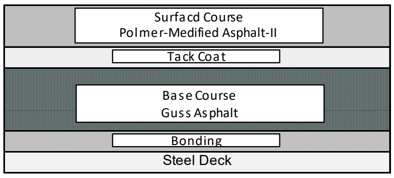
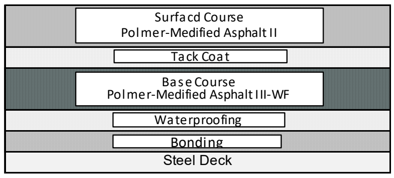
- Road surface deflects easily; pavement has to follow this deflection.
- It is necessary to protect the steel deck from rainwater.
- It is necessary to have the bonding effect between pavement and steel deck.

In Japan, Guss asphalt is adopted for the pavement of steel plate deck. It was introduced in the Honshu-Shikoku Bridge and it has lasted more than 20 years after construction. However, it needs special construction technique such as exclusive asphalt finisher, cooker heater, and shot blasting. For construction in Myanmar, both materials and heavy machinery will be imported and expensive. Also, there are restrictions on importing vehicles that have been in existence for more than ten years.

Improved asphalt is an alternative to Guss asphalt. The improved asphalt has elasticity like that of rubber and the material has increased viscosity. It can also be applied using general heavy machine.

Table 3.2.10 compares the guss asphalt and improved asphalt.

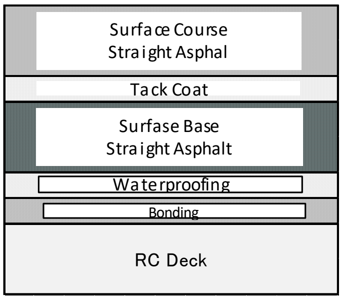
Table 3.2.10 Pavement for Steel Deck

	Case 1 GUSS ASPHALT		Case 2 IMPROVED ASPHALT	
Asphalt Layer				
Surface Course	Polmer-Modified Asphalt II t=40mm		Polmer-Modified Asphalt II t=40mm	
Tack Coat	0.4ℓ/㎡		0.4ℓ/㎡	
Base Course	Guss Asphalt t=40mm		Polmer-Modified Asphalt III-WF t=40mm	
Waterproofing	-		Hot-applied Asfalt Menmbrance Waterproofing	
Bonding	Solvent-type Rubber Asphalt Primer		Solvent-type Rubber Asphalt Primer	
Thickness	Total 80mm		Total 80mm	
Featuers	- Guss Asphalt has excellent flexibility. - Guss has Waterproofing featuer, unneccessary Waterlayer. - Special Construction Machines are required.	◎	- Improved Asphalt has excellent flexibility. - Waterlayer is necessary. - Can be constructed with normal Machines.	◎
Construction Period	3 Days/1000m2	○	1 Day/1000m2	◎
Maintain	- Special Construction Machines are required	○	- Improved Asphalt has easy maintain. - Can be constructed with normal Machines.	◎
COST	1.3	○	1.0	◎
Evaluation			Recommended	

Source: JICA Study Team

3.2.3.2 RC Deck Section

RC deck has less deck deflection than steel deck. For economical consideration, straight asphalt pavement is recommended.



Source: JICA Study Team

Figure 3.2.6 Pavement Layer on RC Deck

3.2.4 Effect of Overloaded Vehicles

The overloaded vehicles are not only violating traffic rules but also adversely affecting the road, especially bridge. A weighting apparatus is installed on the current road (Thaketa side) before the bridge to measure the weight of vehicles and control the overloaded vehicles.

3.2.4.1 Results of Previous Survey

Table 3.2.11 below shows the results of the previous overloading survey.

Table 3.2.11 Results of Overloading Survey

	Number	Ave. Weight	Ave. Overweight	Remark
2016.4～2017.3	661	40.2 t	10.9 t	

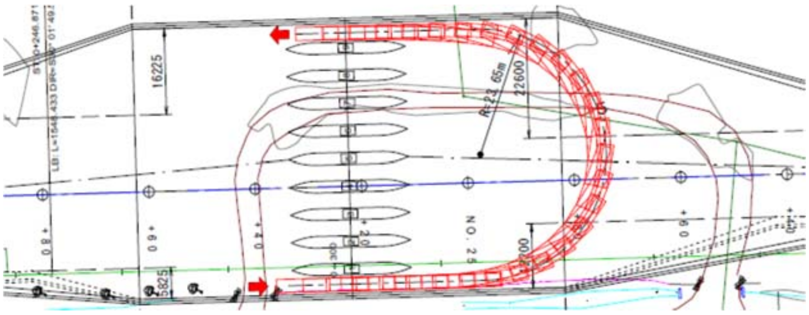
Source: JICA Study Team

Based on the statistics in the past year, the number of overloaded vehicle is 661/yr (2016.4-2017.3); the average weight is 40.2 t; and the average overweight is about 10 t. Assuming that the traffic volume is 27,400/day (this is the traffic demand forecast for 2018 based on the FS report), the overloaded vehicle ratio is 2.4%.

3.2.4.2 Measures Against Overloaded Vehicle

The survey result of 2.4% indicates about two cars per day, which is not very much, and it is thought that this result shows the effect of control. However, it is expected that the increased traffic will cause the increase of overloaded vehicles and careful control will be needed in the future. The following are measures against overloaded vehicles:

- Overloaded vehicles shall not be allowed on any roads; continuous control is necessary in the future.
- In the project road, the weigher will be set under the carriageway; the control will be easier than the present.
- The project road will have a structure that will require overloaded vehicles to make a U-turn, and that will not allow them to go through the bridge.
- The weigher will be set only on the Thaketa side. Setting of the weigher on the Thanlyin side will be desirable in the future.



Source: JICA Study Team

Figure 3.2.7 U-Turn Track on Toll Gate

3.3 SOFT SOIL TREATMENT

In this Project, embankments are planned on the main line (STA.0+0.000~STA.0+352.000, STA.2+392.500~STA.2+676.000), approach road (STA.0+0.000~STA.0+184.986), and on-ramp (STA.0+0.000~STA.0+406.000). The maximum embankment height is about 5 m from the base surface of the construction. Based on the results of the geological survey, soft soil is distributed in the foundation ground planned for the embankment; the depth is 22 m in the Thanlyin side; and the depth is 14 m in the Thaketa side. In the case of embankment construction on these soft soils, there are some concerns, such as the subsidence occurring after the service due to consolidation settlement, securing stability during and after the construction, collapse of embankment by liquefaction, retracted subsidence of surrounding ground due to consolidation subsidence, insufficient bearing capacity of structure, and the influence on the abutment or existing structure due to the lateral movement of the soil.

Engineering analysis is conducted for these problems, and when measures are required, the optimal countermeasures will be investigated.

3.3.1 Design Standard

In performing the engineering analysis, the design conditions are set as follows. In Myanmar, the design criteria for soft soil countermeasures are not in place, so Japan's "Guideline for Road Earthwork of Soft Ground Treatment (August, 2012)" was applied in the design.

3.3.1.1 Ground Condition

The geological stratigraphy revealed by the geological survey is shown in Table 3.3.1 to Table 3.3.5.

Table 3.3.1 Geological Stratigraphy Table

Age	Symbols	Formation	Description
Quarter-nary	Q2	Alluvium	This top soil was deposited in recent time as river deposits and it is blanketing over the project area. This formation has brown to gray color and the main constituent is clay and silty sand with clay patches. These deposits are built by the effect of flood action. This formation yields medium to high water content
Miocene-Pliocene	Tm-Tp	Irrawaddy Formation and its equivalent	This formation is composed of yellowish fine sand of the Irrawaddian Group. The outcropping areas can be seen in Danyingone, Arzarnigone, Southern Twin Te, and the left bank of Yangon-Thanlyin across the Pegu (Bago) River.
Miocene	Tm	Upper Pegu Group and its equivalent	This formation is mainly composed of sand and shale interbeds. The outcropping areas are found along the anticlinal ridges of the Danyingone and Thanlyin areas. Most of them are composed of reddish brown oxidized lateritic soil.

Source: JICA Study Team

Table 3.3.2 Geologic Stratigraphy of Thanlyin Side

Formation	Soil Type	N-value (Representative -Value)	Description
Alluvium	1. Filled Soil	0~3 (1)	The thickness is about 1.0 m to 2.0 m. The color is brown. The plasticity is low to medium, and the water content is moist. The consistency is very soft to soft.
	2. CLAY-I	0~4 (1)	The thickness is about 3.0 m to 11.0 m. The color is gray. The water content is moist to wet, and the plasticity is low to high. The consistency is very soft to soft.
	3. Sandy CLAY-I	2~10 (3)	The thickness is about 2.0 m to 7.0 m. The color is gray, and the water content is moist. The plasticity is low to medium. The consistency is soft to stiff.
	4. Clayey SAND-A	2~15 (3)	The thickness is about 4.0 m to 6.0 m. The color is brownish gray, and the water content is moist to wet. The plasticity of clay is low. The grain size of sand is fine to medium.
	5. Silty SAND-I	5~29 (15)	The thickness is about 3.0 m to 9.0 m. The color is gray, and the water content is moist. The grain size is fine to medium. The relative density is loose to medium dense.
	6. CLAY-AII	2~19 (5)	The thickness is about 7.0 m to 13.0 m. The color is gray, and the water content is moist. The plasticity is low to medium. The consistency is soft to very stiff.
	7. Clayey SAND-B	15~27 (17)	The thickness is about 2.0 m. The color is gray, and the water content is moist. The plasticity of clay is low and the grain size of sand is fine. The relative density is medium dense.
	8. CLAY-AIII	3~33 (7)	The thickness is about 14.0 m to 26.0 m. The color is gray, and the water content is moist to wet. The plasticity is low to medium. The consistency is soft to hard.
	9. Clayey SAND-C	20~32 (20)	The thickness is about 2.0 m. The color is gray, and the water content is moist. The plasticity of clay is low to medium. The grain size of sand is fine to medium.
Irrawaddy Formation	10. Clayey SAND-I	10~≧50 (23)	The thickness is about 3.0 m to 12.0 m. The color is greenish gray to yellowish brown, and the water content is moist. The grain size of sand is fine to medium.
	11. Clayey SAND-II	≧50 (50)	The thickness of this layer is more than 9.0 m. The color is yellowish brown, and the water content is moist. The grain size of sand is fine to medium.

Source: JICA Study Team

Table 3.3.3 Geologic Stratigraphy of River Section

Formation	Soil Type	N-value (Representative -Value)	Description
Alluvium	1. Silty SAND-River Sediments	2~10 (3)	The thickness is about 2.0 m to 11.0 m. The color is brownish gray, and the water content is moist to wet. The grain size of sand is fine to medium.
	2. CLAY-I	0~4 (1)	The thickness is about 1.0 m to 5.5 m. The color is gray, and the water content is wet to moist. The plasticity is low to medium. The consistency is very soft to soft.
	3. Clayey SAND-A	2~5 (3)	The thickness is about 4.0 m to 6.0 m. The color is brownish gray, and the water content is moist to wet. The grain size is fine to medium. The relative density is very loose to loose.
	4. Silty SAND-I	3~38 (13)	The thickness is about 4.0 m to 12.0 m. The color is gray, and the water content is moist. The grain size of sand is fine to medium. The relative density is very loose to medium.
	5. Sandy CLAY-II	7~14 (9)	The thickness is about 2.0 m to 4.0 m. The color is gray, and the water content is moist. The plasticity of clay is low to medium. The consistency is firm to stiff.
	6. CLAY-AII	4~19 (7)	The thickness is about 2.0 m to 19.0 m. The color is gray, and the water content is moist. The plasticity of clay is low to medium. The consistency is soft to very stiff.
	7. Clayey SAND-B	7~19 (13)	The thickness is about 1.0 m to 6.0 m. The color is gray, and the water content is moist. The grain size is fine to medium. The relative density is loose to medium.
	8. Silty SAND-A	17~36 (25)	The thickness is about 3.0 m to 7.0 m. The color is gray, and the water content is moist. The grain size is fine to medium. The relative density is medium dense to dense.
	9. CLAY-AIII	11~35 (18)	The thickness is about 5.0 m to 23.0 m. The color is gray, and the water content is moist. The plasticity is medium to high. The consistency is stiff to hard.
	10. Clayey SAND-C	10~40 (20)	The thickness is 2.0 m to 19.0 m. The color is gray, and the water content is moist. The grain size is fine to medium. The relative density is medium dense to dense.
	11. Silty SAND-II	17~43 (30)	The thickness is about 3.0 m to 16.0 m. The color is gray, and the water content is moist. The grain size is fine to medium. The relative density is medium dense to dense.
Irrawaddy Formation	12. Clayey SAND-I	24~50 (35)	The thickness is about 1.0 m to 8.0 m. The color is gray and reddish brown to yellowish brown at some depths. The water content is moist. The grain size is fine to medium.
	13. CLAY-AIV	26~≧50 (30)	The thickness is about 6.0 m to 10.5 m. The color is gray, and the water content is moist. The plasticity is low to medium. The consistency is very stiff to hard. Fine grained sand is included.
	14. Clayey SAND-II	44~≧50 (50)	The thickness is more than 12.0 m. The color is yellowish brown. The grain size is fine to medium. The relative density is dense to very dense.

Source: JICA Study Team

Table 3.3.4 Geologic Stratigraphy of Thaketa Side

Formation	Soil Type	N-value (Representative -Value)	Description
Alluvium	1. Filled Soil	3 (3)	The thickness is about 3.0 m. The color is brown, and the water content is low to moist. The plasticity is low to medium. The consistency is soft.
	2. CLAY-I	0~5 (1)	The thickness is about 6.0 m to 10.0 m. The color is gray. The water content is moist to wet, and the plasticity is low to high. The consistency is very soft to firm.
	3. Silty SAND-I	4~30 (13)	The thickness is about 3.0 m to 9.0 m. The color is gray, and the water content is moist to wet. The grain size is fine. The relative density is loose to medium dense.
	4. Sandy SILT	5~7 (7)	The thickness is about 3.0 m. The water content is moist to wet, and the plasticity is low to medium. The consistency is firm.
	5. Silty SAND-II	13~47 (25)	The thickness is about 14.0 m to 19.0 m. The color is gray, and the water content is moist. The grain size is fine to medium. The relative density is medium dense to dense.
Irrawaddy Formation	6. Clayey SAND-I	14~50 (35)	The thickness is about 7.0 m to 15.0 m. The color is gray, and the water content is moist. The grain size is fine to medium. The relative density is medium dense to dense.
	7. Clayey SAND-II	≧50 (50)	The thickness is more than 8.0 m. The color is yellowish brown to reddish brown, and the water content is moist. The relative density is dense to very dense.

Source: JICA Study Team

Table 3.3.5 Geologic Stratigraphy of Flyover Section

Formation	Soil Type	N-value (Representative -Value)	Description
Alluvium	1. Filled Soil	3~7 (4)	This filled soil layer is almost observed as CLAY, and Sandy CLAY and Silty SAND in some boreholes. The thickness is about 1.0 m to 2.0 m.
	2. CLAY-I	2~8 (4)	The thickness is about 1.0 m to 6.0 m. The color is gray. The plasticity is low to medium, and the water content is moist. The consistency is soft to firm.
	3. Silty SAND-I	2~33 (10)	The thickness is about 3.0 m to 8.0 m. The color is gray. The grain size is fine, and the water content is moist and wet at some depths The relative density is very loose to dense.
	4. Sandy SILT	2~19 (7)	The thickness is about 2.0 m to 5.0 m. The color is gray, and the water content is moist. The plasticity of silt is low. The consistency is soft to very stiff.
	5. Silty SAND-II	6~48 (22)	The thickness is about 9.0 m to 21.0 m. The color is gray, and the water content is moist. The grain size of sand is fine to medium. The relative density is loose to dense.
	6. CLAY-II	11~41 (20)	The thickness is about 1.0 m to 11.0 m. The color is gray, and the water content is moist. Moreover, fine grained sand is included in this layer. The consistency is stiff to hard.
Irrawaddy Formation	7. Clayey SAND-I	10~≧50 (35)	The thickness is about 2.0 m to 16.0 m. The color is yellowish brown and gray at some depths, and the water content is moist. The relative density is loose to very dense.
	8. CLAY-III	19~≧50 (31)	The thickness is about 7.0 m to 9.0 m. The color is gray, and the water content is moist. The plasticity of this layer is low to medium. The consistency is very stiff to hard.
	9. Clayey SAND -II	34~≧50 (50)	The thickness is more than 10.0 m. The color is gray, and the water content is moist. The grain size of sand is fine to medium. The relative density is dense to very dense.
	10. CLAY-IV	32~≧50 (50)	The thickness of this layer is more than 14.0 m. The plasticity of this layer is low to medium. Moreover, fine grained sand is included. The consistency is hard.

Source: JICA Study Team

In the range requiring soft soil research, the layers corresponding to soft soil among the layers shown in Table 3.3.1 to Table 3.3.5 are generally cohesive soil ground with an N-value of 4 or less, and sandy soil ground with an N-value of 10 to 15 or less. The soft soil layers in each side are shown in Table 3.3.6.

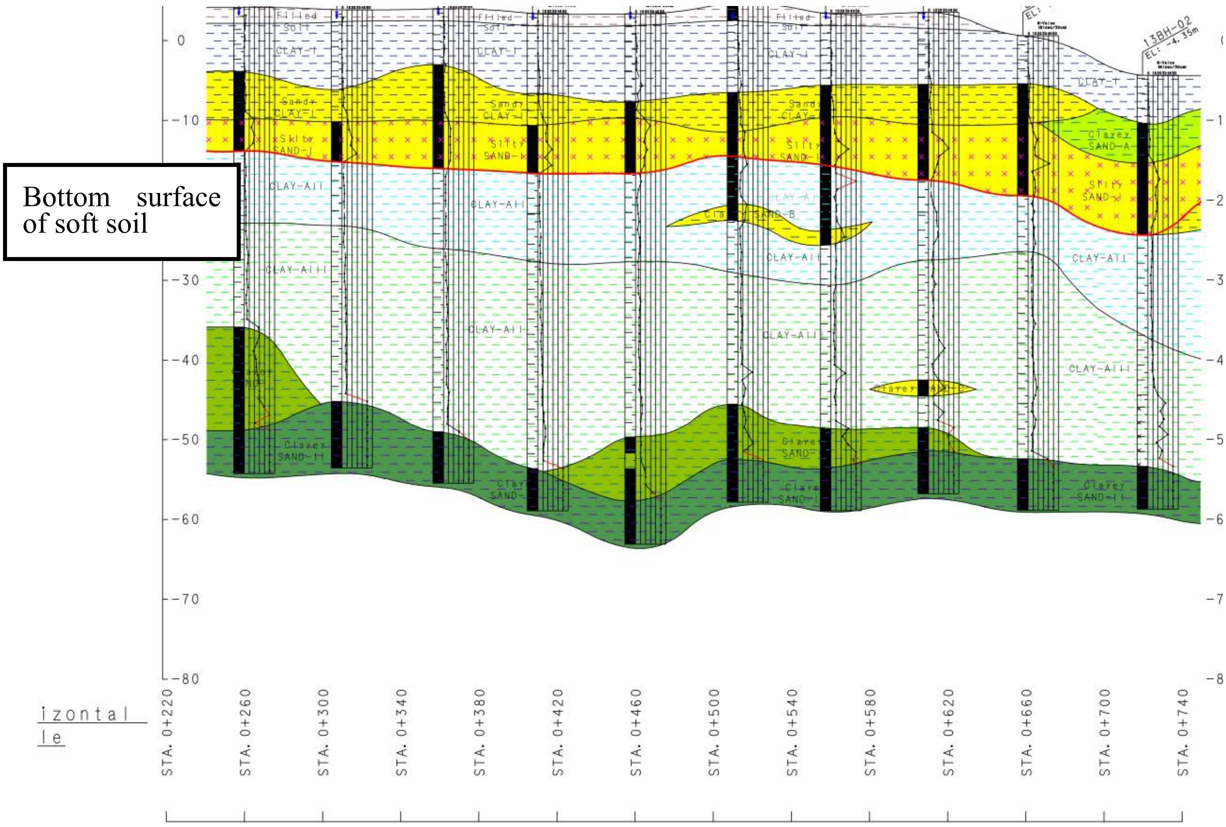
Table 3.3.6 List of Soft Soil

Thanlyin Side		River Section	
Soil Name	N-value (Average)	Soil Name	N-value (Average)
Filled Soil	1	SiltySAND-River Sediments	3
CLAY-I	1	CLAY-I	1
Sandy CLAY-I	3	Clayey SAND-A	3
Silty SAND-I	15	Silty SAND-I	13
Clayey SAND-A	3	Clayey SAND-B	13

Thaketa Side		Flyover Section	
Soil Name	N-value (Average)	Soil Name	N-value (Average)
Filled Soil	3	Filled Soil	4
CLAY-I	1	CLAY-I	4
Silty SAND-I	13	Silty SAND-I	10

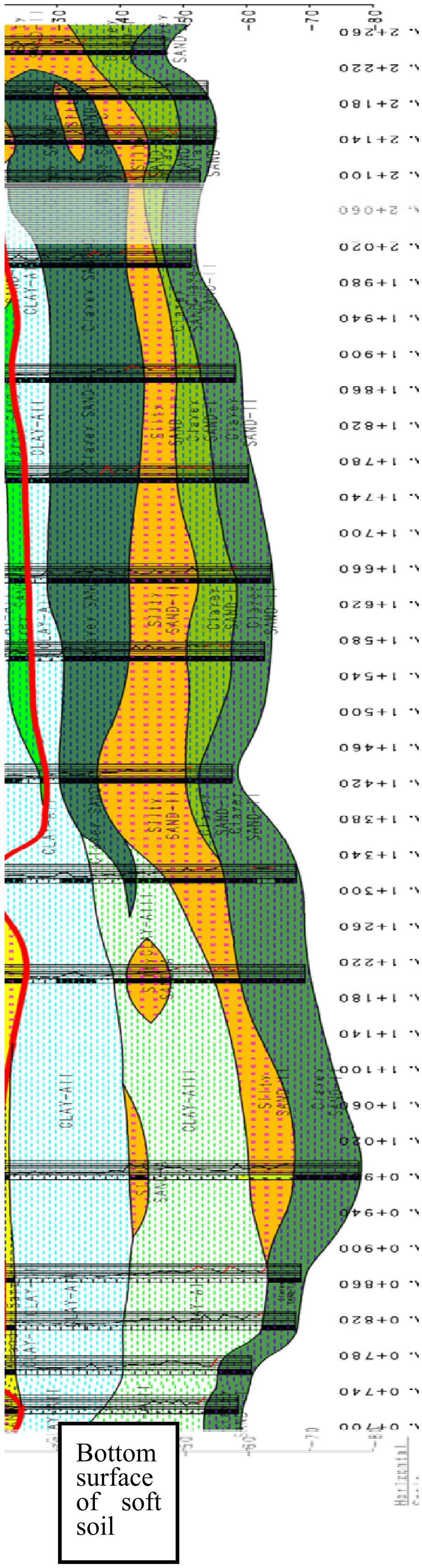
Source: JICA Study Team

From the above, it can be considered that the depths of the distribution of soft soil layer from the construction base are 18 m to 22 m in the Thanlyin side, 10 m to 22 m in the river section, 12 m to 20 m in the Thaketa side, and 9 m to 13 m in the flyover section. In the geological profile of each side, the bottom of the soft soil layer is indicated by a red line.



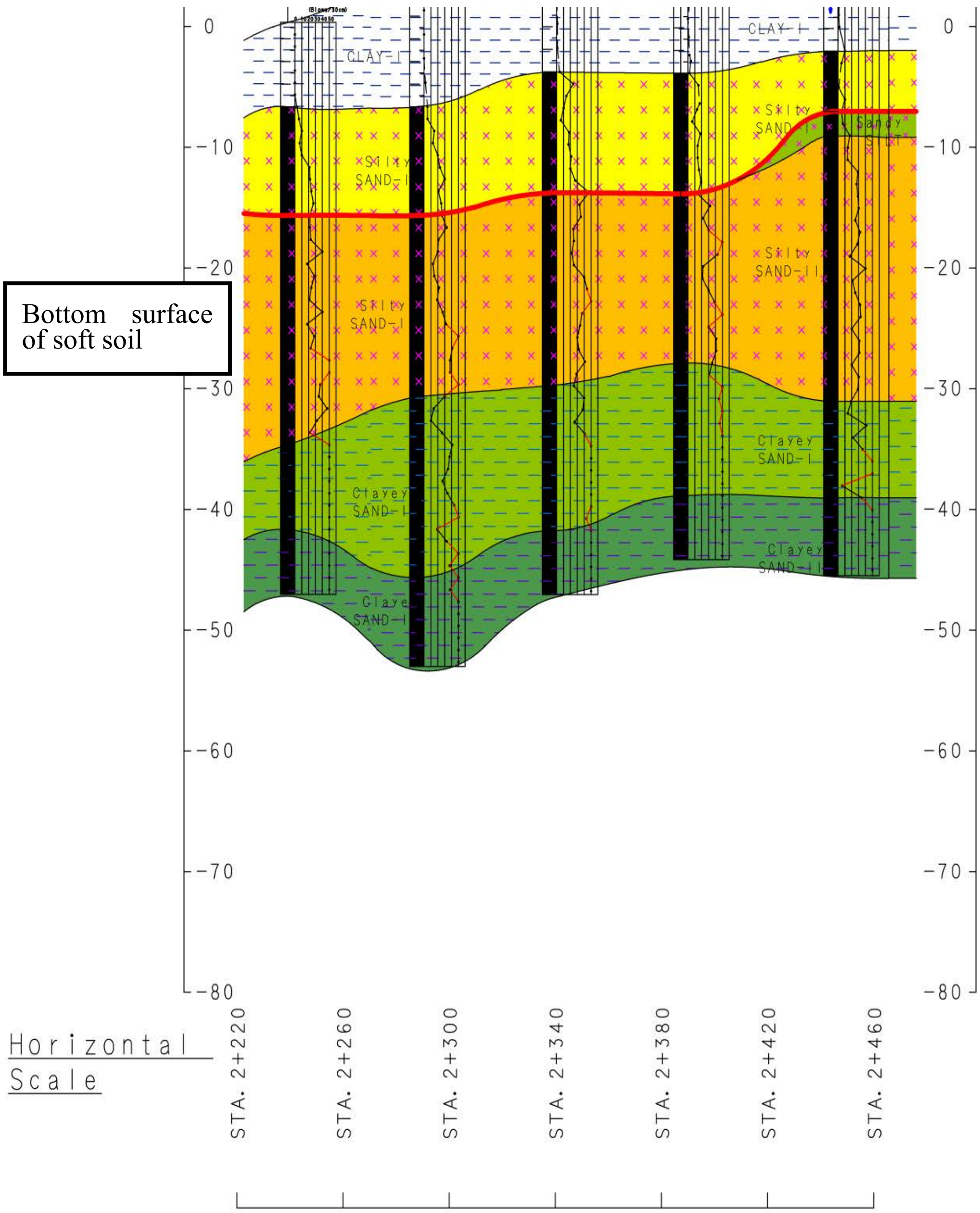
Source: JICA Study Team

Figure 3.3.1 Soil Profile of Thanlyin Side



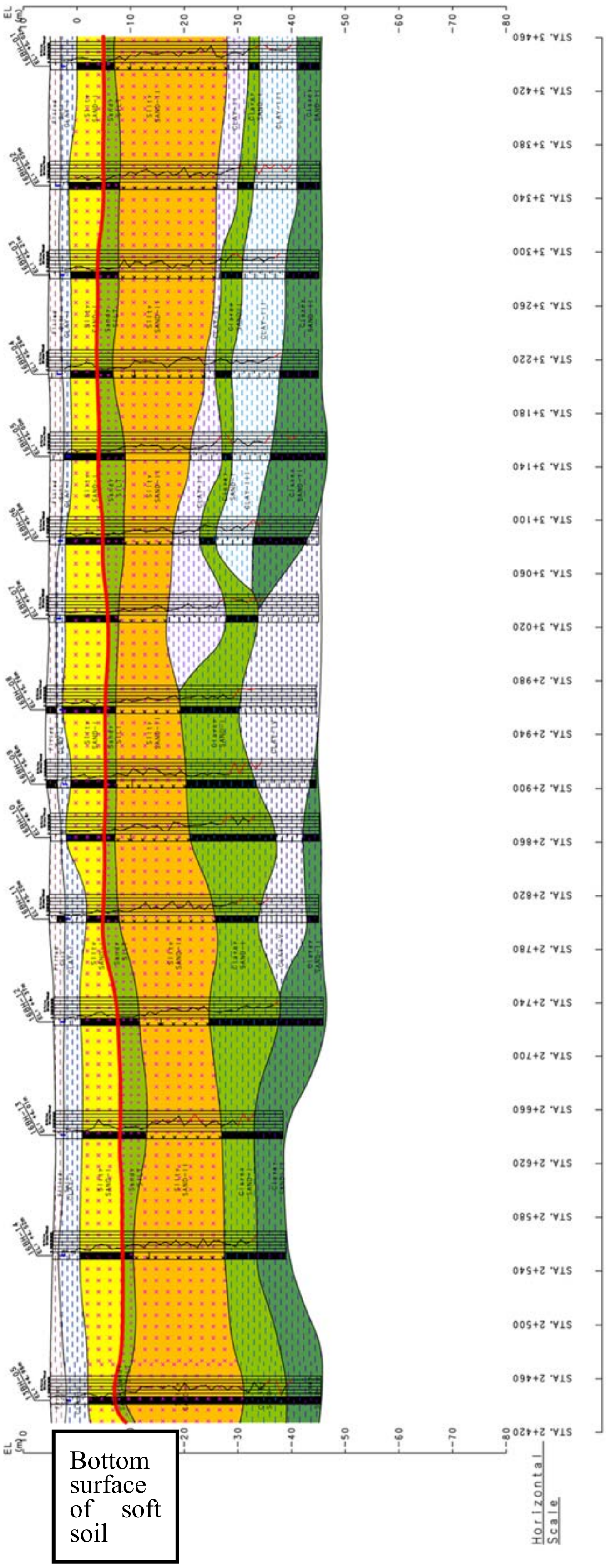
Source: JICA Study Team

Figure 3.3.2 Geological Profile of River Section



Source: JICA Study Team

Figure 3.3.3 Geological Profile of Thaketa Side



Source: JICA Study Team

Figure 3.3.4 Geological Profile of Flyover Section

3.3.1.2 Setting of the Analysis Block Classification

In the soft soil analysis, the areas are divided into blocks as shown in Table 3.3.7 in consideration of the ground conditions, among others.

The analytical model columnar section uses the geological composition of the researched areas based on the geological profile.

Table 3.3.7 Analysis Block Classification

Area Name	Block Numbe	Stationary Point	Extension (m)	Embankment Height(m)	Soft Soil Layer Thickness(m)	Reason for Setting
Thanlyin Area	-	STA.0+000.000 ~ STA.0+130.000	130.0	0.41 ～ 0.6	-	It is the range planned on the current road, as it can be thought that it is being compacted at present condition, so it is out of the scope of consideration for soft ground treatment.
	Block1	STA.0+130.000 ~ STA.0+250.000	120.0	1.31 ～ 2.1	18	Low embankment structure.
	Block2	STA.0+250.000 ~ STA.0+322.000	110.0	2.1 ～ 3.51	18～20	An embankment structure on the upstream side and a retaining wall structure on the downstream side.
	Block3	STA.0+322.000 ~ STA.0+352.000	30.0	3.51 ～ 4.36	19～20	At the rear of the A1 bridge, the upstream side is the embankment structure and the downstream side is the retaining wall structure.
Thaketa Area	Block4	STA.2392.500 ~ STA.2+593.800	201.3	3.6 ～ 4.23	12～14	From the geological longitudinal map, the soft soil layers are composed of Filled soil, Clay-1 and Silty sand-1, with the embankment structure on the upstream side and the retaining wall structure on the downstream side.
	Block5	STA.2+593.800 ~ STA.2+676.000	82.2	3.56 ～ 4.37	12～13	The main line is a retaining wall structure, the approach road is an embankment structure on the upstream side, and the retaining wall structure on the downstream side.
	Block6	STA.2+676.000 ~ STA.2+800.000	124.0	0.5 ～ 4.37	9～12	Designed with a low embankment structure with one lane on one side of the approach road.
On-ramp	Block7	STA.0+000.000 ~ STA.0+367.483	367.5	0.22 ～ 2.57	17～20	Low embankment structure.
	Block8	STA.0+367.483 ~ STA.0+406.000	38.5	2.57 ～ 4.86	17～20	Retaining wall structure at the rear of the A1 bridge.

Source: JICA Study Team

The analysis of the ground after the countermeasure is carried out in accordance with the "purpose of countermeasure" only when it is judged that "countermeasure is necessary" from the current ground analysis result.

The number of analyzed cross sections is summarized in Table 3.3.8.

Table 3.3.8 Number of Examined Cross Sections

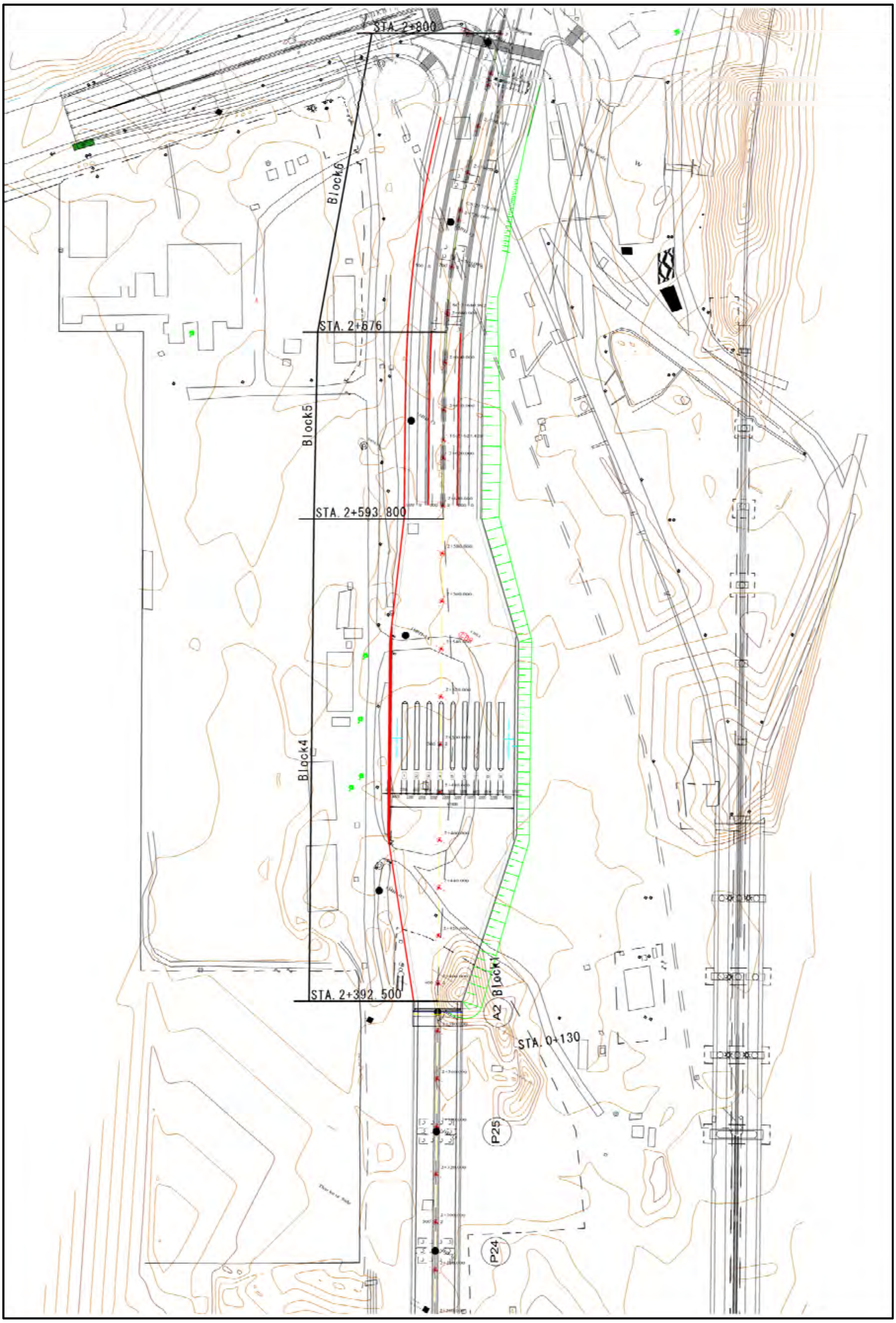
<table>
<tr><th rowspan="2">Area Name</th><th rowspan="2">Block Number</th><th colspan="6">Number of analyzed cross sections</th><th rowspan="2">Analyzed cross section position</th></tr>
<tr><th>Settlement analysis</th><th>Deformation analysis</th><th>Liquefaction research</th><th>Stability analysis</th><th>Lateral movement research</th><th>Bearing capacity research</th></tr>
<tr><td rowspan="3">Thanlyin side</td><td>Block1</td><td>1</td><td>1</td><td>1</td><td>1</td><td>-</td><td>-</td><td>STA.0+240.000</td></tr>
<tr><td>Block2</td><td>1</td><td>1</td><td>1</td><td>1</td><td>-</td><td>-</td><td>STA.0+320.000</td></tr>
<tr><td>Block3</td><td>1</td><td>1</td><td>1</td><td>1</td><td>1</td><td>1</td><td>STA.0+340.000</td></tr>
<tr><td rowspan="3">Thaketa side</td><td>Block4</td><td>1</td><td>1</td><td>1</td><td>1</td><td>1</td><td>1</td><td>STA.2+400.000</td></tr>
<tr><td>Block5</td><td>1</td><td>1</td><td rowspan="2">1</td><td>1</td><td>1</td><td>1</td><td>STA.2+620.000</td></tr>
<tr><td>Block6</td><td>1</td><td>1</td><td>1</td><td>-</td><td>-</td><td>STA.2+680.000</td></tr>
<tr><td rowspan="2">On-ramp</td><td>Block7</td><td>1</td><td>1</td><td rowspan="2">1</td><td>1</td><td>-</td><td>-</td><td>STA.0+360.000</td></tr>
<tr><td>Block8</td><td>1</td><td>1</td><td>1</td><td>1</td><td>1</td><td>STA.0+400.000</td></tr>
</table>

Source: JICA Study Team

The analysis blocks and analysis cross section positions are shown in Figure 3.3.5 to Figure 3.3.6.

Source: JICA Study Team

Figure 3.3.5 Block Classification (Thanlyin Side and On-ramp)



Source: JICA Study Team

Figure 3.3.6 Block Classification (Thaketa Side)Setting of Geotechnical Parameters

(1) Unit Weight (γt), Cohesive Strength (c), Internal Friction Angle (φ)

The values set in Section 2.1 (Soil Investigation) are used here for the geotechnical parameters.

The soil constants are shown from Table 3.3.9 to Table 3.3.10

Table 3.3.9 Geotechnical Parameters in Thanlyin Side

No.	Soil Name	Representative N Value	Unit Weight			Internal Friction Angle	Cohesive Strength
			γt (kN/m^3)	Γsat (kN/m^3)	γ' (kN/m^3)	Φ (°)	c (kN/m^2)
1	Filled Soil	1 1)	18.0 2)	18.0	8.0	-	15 2)
2	CLAY-I	1 1)	17.5 1)	17.5	7.5	-	22 1)
3	Sandy CLAY-I	3 1)	17.5 1)	17.5	7.5	-	24 1)
4	Silty SAND-I	15 1)	16.5 1)	17.5	7.5	33 3)	-
5	Clayey SAND-A	3 1)	17.0 2)	18.0	8.0	28 3)	-
6	CLAY-AII	5 1)	17.5 1)	17.5	7.5	-	30 1)
7	Clayey SAND-B	17 1)	17.0 2)	18.0	8.0	33 3)	-
8	CLAY-AIII	7 1)	17.6 1)	17.6	7.6	-	42 3)
9	Clayey SAND-C	20 1)	17.0 2)	18.0	8.0	32 3)	-
10	Clayey SAND-I	23 1)	17.0 2)	18.0	8.0	31 3)	-
11	Clayey SAND-II	50 1)	19.0 2)	20.0	10.0	35 3)	-

1) These values were obtained from field test or soil laboratory test results.
2) These values were obtained from the reference value shown in NEXCO.
3) These values were obtained from the SPT N-value formula.
4) These values were obtained from the formula.

Source: JICA Study Team

Table 3.3.10 Geotechnical Parameters in Flyover Section

No.	Soil Name	Representative N Value	Unit Weight			Internal Friction Angle	Cohesive Strength
			γt (kN/m^3)	Γsat (kN/m^3)	γ' (kN/m^3)	Φ (°)	c (kN/m^2)
1	Filled Soil	4[5)]	16.0[2)]	16.0	6.0	-	24[3)]
2	CLAY-I	4[1)]	18.0[1)]	18.0	8.0	-	24[1)]
3	Silty SAND-I	10[1)]	17.0[1)]	18.0	8.0	32[3)]	-
4	Sandy SILT	8[1)]	18.0[2)]	18.0	8.0	-	48[3)]
5	Silty SAND-II	23[1)]	19.0[2)]	20.0	10.0	33[3)]	-
6	CLAY-II	22[1)]	18.0[2)]	18.0	8.0	-	132[3)]
7	Clayey SAND-I	41[1)]	19.0[2)]	20.0	10.0	33[3)]	-
8	CLAY-III	35[1)]	18.0[2)]	18.0	8.0	-	210[3)]
9	Clayey SAND-II	50[1)]	19.0[2)]	20.0	10.0	37[3)]	-
10	CLAY-IV	50[1)]	18.0[2)]	18.0	8.0	-	300[3)]

1) These values were obtained from field test or soil laboratory test results.
2) These values were obtained from the reference value shown in NEXCO.
3) These values were obtained from the SPT N-value formula.
4) These values were obtained from the formula.

Source: JICA Study Team

(2) e-log p Curve

1) Cohesive Soil

The e-log p curve is the average curve based on the consolidation test data obtained from the geological survey performed in each block. However, when the consolidation test is not conducted, the surrounding test values considered to be similar in terms of soil quality are used.

Table 3.3.11 shows the values used for each layer of each block.

Table 3.3.12 shows the e-log p curve used for each boring and block.

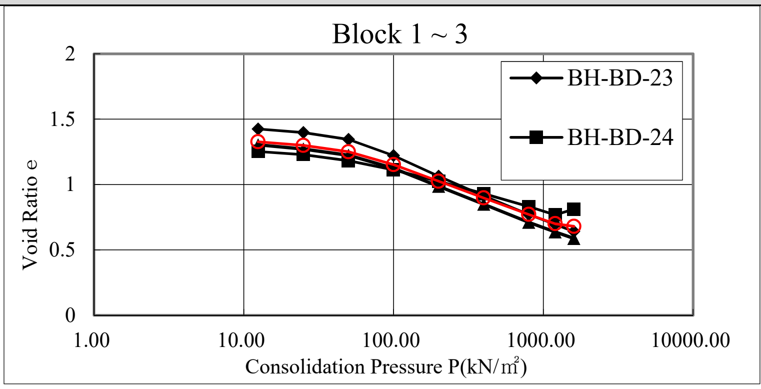
Table 3.3.11 e-log p Curve List

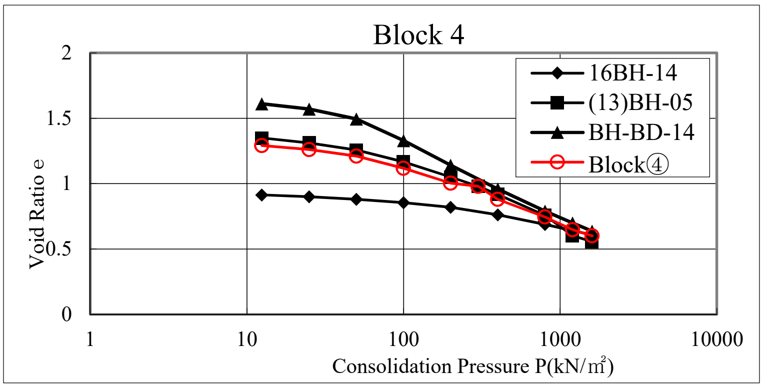
Block No.	Block 1, 2, 3				Block 4	
Soil Name	CLAY- I		Sandy CLAY- I		CLAY- I	
Values	P	e	P	e	P	e
	12.46	1.33	12.46	1.26	12.46	1.29
	25.02	1.30	25.02	1.24	25.02	1.26
	50.03	1.25	50.03	1.21	50.03	1.21
	100.06	1.15	100.06	1.15	100.06	1.12
	200.03	1.03	200.03	1.04	200.03	1.00
	400.15	0.90	400.15	0.91	300.09	0.98
	800.30	0.77	800.30	0.78	400.15	0.88
	1200.55	0.70	1200.55	0.71	800.30	0.74
	1600.50	0.68	1600.50	0.66	1200.55	0.65

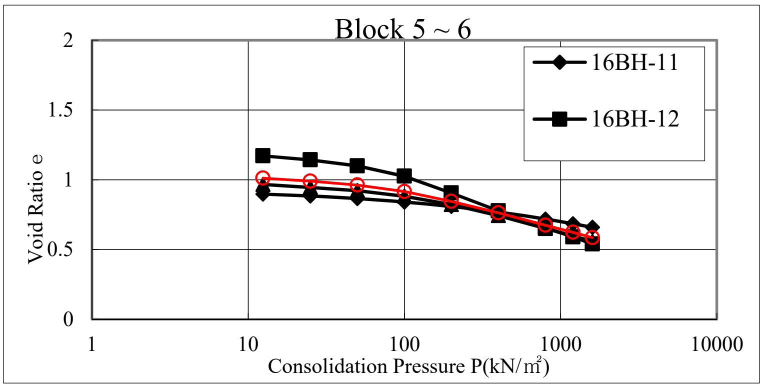
Block No.	Block 5, 6		Block 7, 8			
Soil Name	CLAY- I		CLAY- I		Sandy CLAY- I	
Values	P	e	P	e	P	e
	12.46	1.01	12.46	1.38	12.46	1.22
	25.02	0.99	25.02	1.34	25.02	1.20
	50.03	0.96	50.03	1.26	50.03	1.16
	100.06	0.92	100.06	1.15	100.06	1.11
	200.03	0.85	200.03	1.03	200.03	1.02
	400.15	0.76	300.09	0.91	400.15	0.90
	800.30	0.67	400.15	0.91	800.30	0.76
	1200.55	0.62	800.30	0.78	1200.55	0.69
	1600.50	0.59	1200.55	0.71	1600.50	0.63
	-	-	1600.50	0.67	-	-

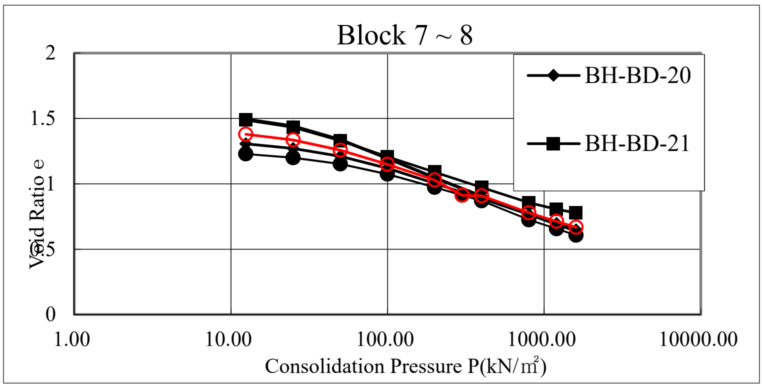
Source: JICA Study Team

Table 3.3.12 e-log p Curve

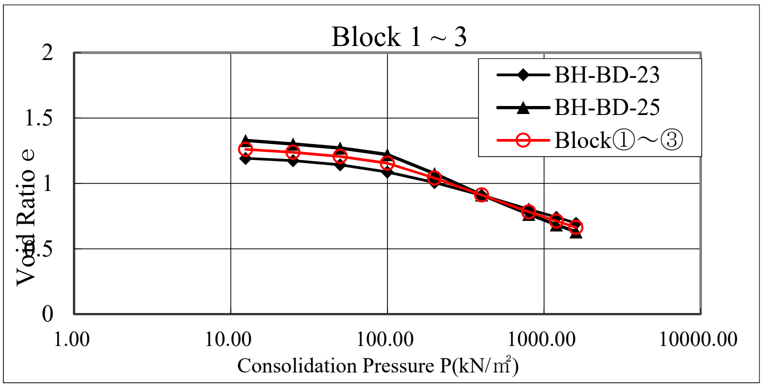
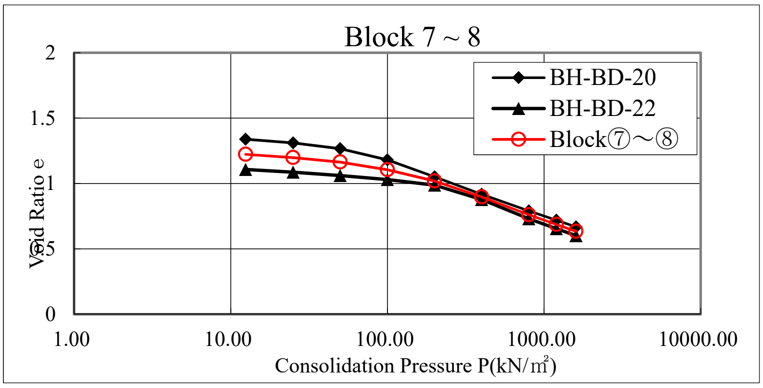
<table>
<tr><th>Soil Name</th><th>e-log p Curve</th></tr>
<tr><td>CLAY- I</td><td>








</td></tr>
</table>

Sandy CLAY- I	 

Source: JICA Study Team

2) Sandy Soil

This is set based on the N-value from Figure 3.3.7.

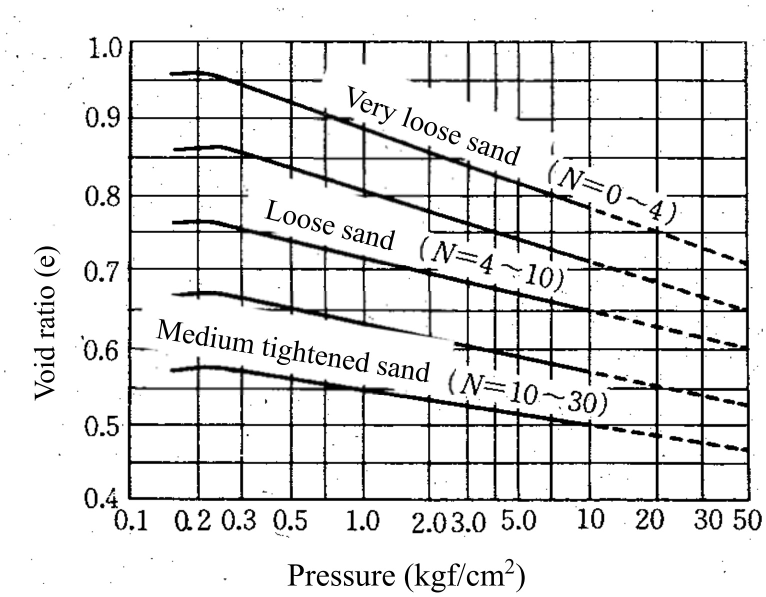


Figure 3.3.7 Pressure-Void Ratio Curve of Sand

Source: Road Earthwork - Guideline for Road Earthwork of Soft Ground Treatment (Japan Road Association) p. 125

(3) cv-log p Curve

1) Cohesive Soil

The cv-log p curve is the average curve based on the consolidation test data obtained from the geological survey performed in each block. However, when the consolidation test is not conducted, the surrounding test values considered to be similar in terms of soil quality are used.

Table 3.3.13 shows the values used for each layer of each block. Table 3.3.14 shows the cv-log p curve used for each boring and block.

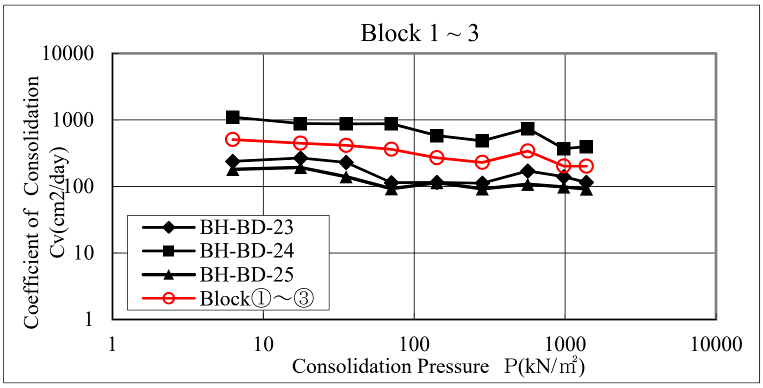
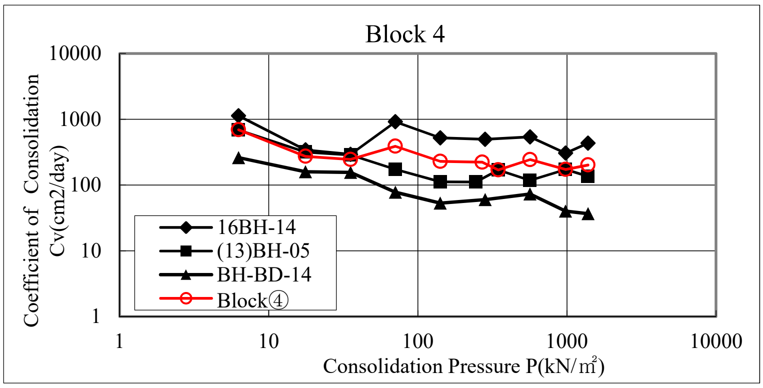
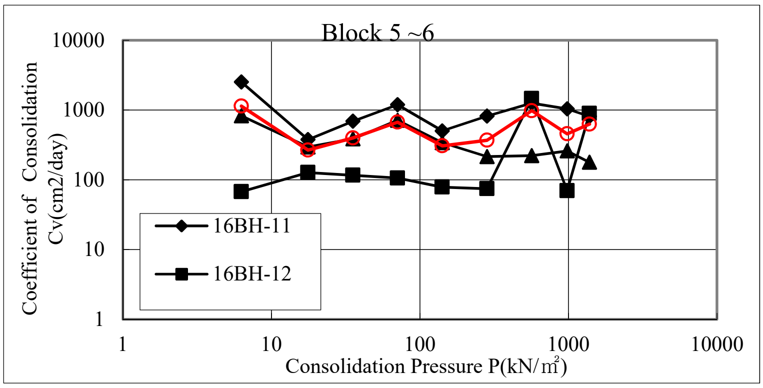
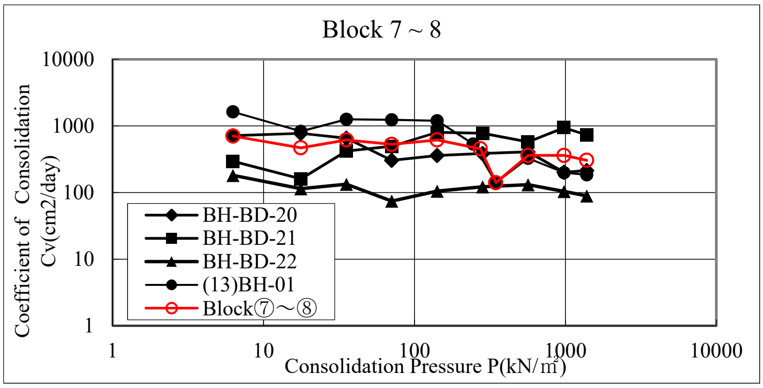
Table 3.3.13 cv-log p curve List

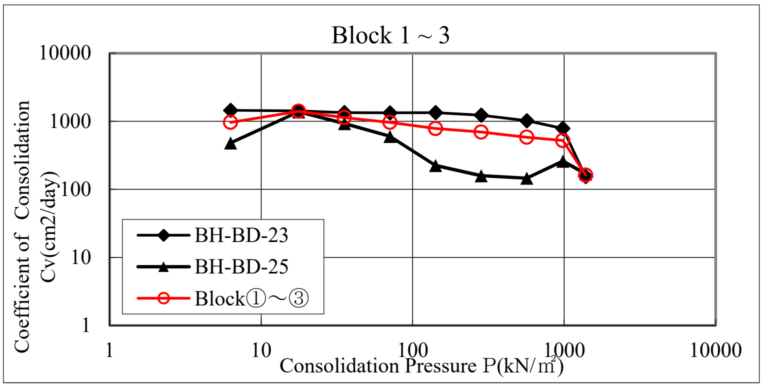
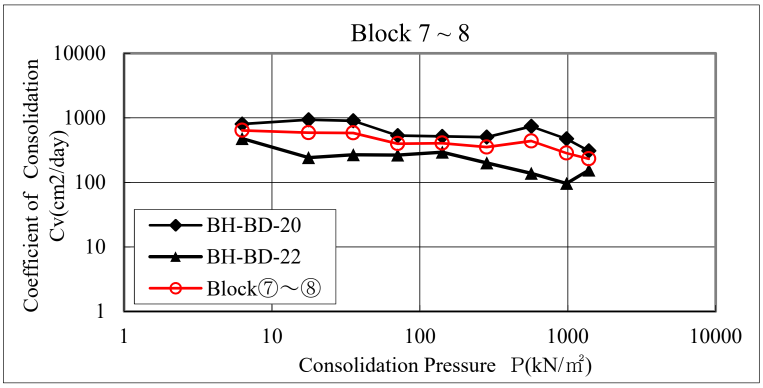
Block No.	Block 1, 2, 3				Block 4	
Soil Name	CLAY- I		Sandy CLAY- I		CLAY- I	
Values	P	e	P	e	P	e
	6.28	507.17	6.28	965.50	6.28	692.17
	17.66	447.00	17.66	1400.00	17.66	273.00
	35.41	414.50	35.41	1133.50	35.41	245.13
	70.73	361.60	70.73	965.50	70.73	388.63
	141.46	270.17	141.46	782.50	141.46	228.98
	282.92	229.93	282.92	694.50	270.27	222.52
	565.94	341.00	565.94	583.00	346.49	169.70
	980.22	202.63	980.22	522.00	565.94	243.28
	1386.15	201.43	1386.15	161.50	980.22	172.40

Block No.	Block 5, 6		Block 7, 8			
Soil Name	CLAY- I		CLAY- I		Sandy CLAY- I	
Values	P	e	P	e	P	e
	6.28	1138.77	6.28	703.50	6.28	639.00
	17.66	266.33	17.66	469.51	17.66	588.00
	35.41	398.33	35.41	614.79	35.41	583.50
	70.73	670.17	70.73	528.12	70.73	398.00
	141.46	307.00	141.46	615.53	141.46	405.00
	282.92	369.53	273.43	454.37	282.92	351.50
	565.94	979.33	346.49	140.80	565.94	437.50
	980.22	456.60	565.94	360.67	980.22	284.15
	1386.15	628.00	980.22	360.83	1386.15	231.50
	-	-	1386.15	304.55	-	-

Source: JICA Study Team

Table 3.3.14 cv-log p Curve

Soil Name	cv-log p Curve
CLAY- I	   

Sandy CLAY- I	 

Source: JICA Study Team

(4) Strength Increase Rate (m)

1) Cohesive Soil

The increase rate of strength according to soil quality is estimated from Table 3.3.15. Since the soils confirmed from the geological survey in this Project are "cohesive soil" and "silt", the minimum value of the strength increase rate applicable for both soils, i.e., m=0.30, is adopted.

Table 3.3.15 Range of Strength Increase Rate Based on Soil Quality

Soil	m
Cohesive soil	0. 30～0. 45
Silt	0. 25～0. 40
Organic Soil and Black	0. 20～0. 35
Peat	0. 35～0. 50

Source: Road Earthwork - Guideline for Road Earthwork of Soft Ground Treatment (Japan Road Association) p.83

(5) Consolidation Yield Stress (Pc) and Overburden Pressure (P0)

Consolidation yield stress (Pc) and overburden pressure (P0) are calculated based on the consolidation test data obtained from the geological survey carried out in each block, layer structure, layer thickness, and geological parameter.

Table 3.3.16 shows the values used for each layer of each block.

Table 3.3.16 Consolidation Yield Stress and Overburden Pressure

Block	Soil name	Boring No.	Sample No.	Po (kN/m^2)	Pc (kN/m^2)	Pc-P0	Adopted value	Pc/P0	qu (kN/m^2)
1 ~ 3	CLAY- I	BH-BD-23	T-1	37.2	54.4	17.2	55.9	1.46	54.35
		BH-BD-24	T-1	33.1	45.8	12.7		1.38	32.55
			T-2	70.8	149.0	78.2		2.11	46.70
		BH-BD-25	T-1	32.4	88.2	55.9		2.73	40.95
	Sandy CLAY- I	BH-BD-23	T-2	68.0	112.3	44.4	44.4	1.65	47.90
		BH-BD-25	T-3	92.4	98.0	5.7		1.06	49.95
4	CLAY- I	(13)BH-05	T-1	55.2	122.0	66.8	66.8	2.21	42.58
			T-2	79.8	102.3	22.5		1.28	30.92
		16BH-14	T-1	44.8	261.3	216.5		5.83	92.55
		BH-BD-14	T-1	44.2	47.4	3.2		1.07	26.85
5 ~ 6	CLAY- I	16BH-11	T-1	64.8	220.3	155.5	197.4	3.40	29.65
			T-2	81.2	278.6	197.4		3.43	30.10
		16BH-12	T-1	47.2	75.9	28.7		1.61	43.90
		16BH-13	D-1	39.2	171.3	132.1		4.37	56.40
7 ~ 8	CLAY- I	BH-BD-20	T-1	37.3	62.3	25.1	29.7	1.67	31.30
			T-2	67.3	88.2	21.0		1.31	43.95
		(13)BH-01	T-1	48.7	78.4	29.7		1.61	34.45
			T-2	95.2	78.4	-16.8		0.82	48.95
			T-3	141.7	159.1	17.4		1.12	33.32
		BH-BD-21	T-1	20.6	24.3	3.7		1.18	15.85
			T-3	65.6	294.0	228.4		4.48	40.80
		BH-BD-22	T-1	37.6	35.3	-2.3		0.94	25.50
	Sandy CLAY- I	BH-BD-22	T-2	86.0	239.9	154.0	154.0	2.79	130.20

: Abnormal Value

Source: JICA Study Team

3.3.1.3 Construction Condition

(1) Geological Parameter of Embankment Material

The geological parameters of embankment material are shown in Table 3.3.17.

Table 3.3.17 Geological Parameter of Embankment Material

Classification	Geological Parameter
Unit weight	19 kN/m^3
Cohesive strength	0 kN/m^3
Internal friction angle	30°

Source: JICA Study Team

(2) Embankment Speed

The embankment speed is shown in Table 3.3.18 based on the "Road Earthwork - Guideline for Road Earthwork of Soft Ground Treatment" (Japan Road Association), p.54.

Table 3.3.18 Embankment Speed

Ground Condition	Embankment Speed (cm/day)
Thick cohesive soil ground and black mud, as well as peat soil where thick organic deposit is deposited	3
Ordinary cohesive soil ground	5
Thin cohesive soil ground and black mud, or thin peat soil almost without organic soil	10

Source: Road Earthwork - Guideline for Road Earthwork of Soft Ground Treatment (Japan Road Association) p.54

Cohesive soil and sandy soil distributed up to about 20 m from the ground surface are considered to be a problem in this part, so the embankment speed on the main line is set to 5 cm/day. However, for the on-ramp, which is close to the existing piers, it is set to 3 cm/day in order to suppress the occurrence of transformation.

(3) Post-embankment Period

According to the construction plan of this Project, the Post-embankment period is shown in Table 3.3.19.

Table 3.3.19 Post-embankment Period

Area	Block	Post-embankment Period (Day)
Thanlyin	Block 1 Block 2 Block 3	480 (16 months)
Thaketa	Block 4 Block 5 Block 6	390 (13 months)
On-ramp	Block 7 Block 8	480 (16 months)

Source: JICA Study Team

3.3.1.4 Road Condition

(1) Traffic Load

As shown in Figure 3.3.8, since the dispersion effect of the embankment load gets bigger when the embankment thickness increases, the traffic equivalent load of the embankment on the soft soil tends to decrease.

In this research, the settlement analysis and stability analysis are performed with a traffic equivalent load of 11.6 kN/m^2 for embankments with a height of 3.0 m or more. However, if the embankment height is less than 3.0 m, the value obtained from Figure 3.3.8 is analyzed as the traffic load and is shown in Table 3.3.20.

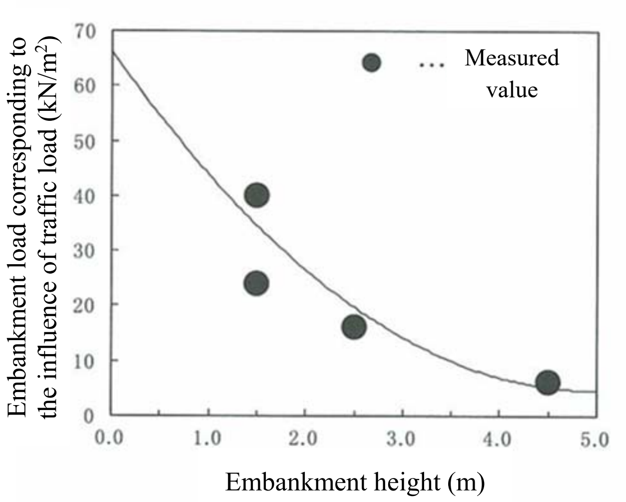


Figure 3.3.8 Embankment Load Corresponding to the Influence of Traffic Load

Source: Road Earthwork - Guideline for Road Earthwork of Soft Ground Treatment (Japan Road Association) p.143

Table 3.3.20 Traffic Load at Each Embankment Height

Embankment Height (m)	Traffic Load (kN/m^2)
3.0~	11.6
2.5~3.0	15
2.0~2.5	20
1.5~2.0	25
1.0~1.5	35
0.5~1.0	45
0.0~0.5	55

Source: JICA Study Team

(2) Target Value for Design

1) Allowable Residual Settlement

The allowable residual settlement is related to the problem of maintenance management and it is set to about 10 to 30 cm in three years after paving under the "Road Earthwork – Guideline for Road Earthwork of Embankment (April, 2010)" (Japan Road Association).

In this Project, considering smooth maintenance and management, as well as the reduction of cost of future overlay after paving and after the road becomes serviceable, and likewise satisfying the residual settlement from the start of the embankment construction to the start of the paving work (PKG1 is 480 days, PKG2 is 390 days), the residual settlement within the influence area of the abutment is set to 10 cm, and the residual settlement outside the influence area of the abutment is set to 30 cm.

2) Allowable Safety Factor

The allowable safety factors for ground fracture during normal period, at the time of earthquake, and at the time of liquefaction are set as shown in Table 3.3.21.

Table 3.3.21 Permissible Safety Factor for Normal, Earthquake, and Liquefaction Periods

Item	Allowable Safety Factor
Normal	Fs=1.25
At the time of earthquake and liquefaction	Fs=1.10

Source: JICA Study Team

Also, referring to "Road Earthworks - Guideline for Road Earthwork of Soft Ground Treatment (August 2012)" p.146, the safety factors for embankment construction and service are shown in Table 3.3.22.

Table 3.3.22 Allowable Safety Factor for Embankment Construction and Service Periods

Item	Allowable Safety Factor
Embankment construction	Fs=1.10
Service period	Fs=1.10

Source: JICA Study Team

3) Allowable Displacement for Proximate Structure

Considering the supporting ground as consolidation layer and the structure type as reinforced concrete structure, the limit inclination angle is 1/700 and the allowable maximum settlement is 10 cm based on Table 3.3.23 and Table 3.3.24, respectively.

Table 3.3.23 Examples of Limit Inclination Angle by Structure

Supporting ground	Structure type	Basic form	Lower limit transformation angle×	Upper limit transformation angle×
Consolidation layer	*RC* *RCW* *CB* *W*	Individual, Continuous, Mat Continuous Continuous Continuous	0.7 0.8 0.3 1.0	1.5 1.8 1.0 2.0 ～ 3.0
Weathered granite (Sand and soil)	*RC* *RCW*	Individual Continuous	0.6 0.7	1.4 1.7
Sand layer	*RC* • *RCW* *CB*	Individual, Continuous, Mat Continuous	0.5 0.3	1.0 1.0
Volcanic clay soil	*RC*	Individual	0.5	1.0
All ground	*S*	Individual, Continuous (Nonflexible finish)	2.0	3.5

[Note] Lower limit transformation angle: Transformation angle at which the number of spaces where cracks occur are exceeding the number of spaces where no crack occurs, and where crack occurrence probability exceeds 50% or transformation angle where crack initiation cumulative number exceeds 30%.

Upper limit transformation angle: Transformation angle at which cracks almost occur, and the crack initiation cumulative number exceeds 70%.

- Abbreviations indicate the following structural types (Same in Table 3.3.23 and 3.3.24):

  RC: Reinforced concrete structure RCW: Wall type reinforced concrete structure

Source: Road Earthwork - Guideline for Architectural Foundation Design (Architectural Institute of Japan) p.153

Table 3.3.24 Examples of Limit Value of Total Settlement by Structure

Supporting ground	Structure type				
	Basic form	Mat	Individual	Continuous	Mat
Consolidation layer	Standard value Maximum	2 4	5 10	10 20	10～(15) 20～(30)
Weathered granite (Sand and soil)	Standard value Maximum	–	1.5 2.5	2.5 4.0	–
Sand layer	Standard value Maximum	1.0 2.0	2.0 3.5	–	–
Volcanic clay soil	Standard value Maximum	–	1.5～2.5 2.0～4.0	–	–
	Structure type	Basic form		Standard value	Maximum
Consolidation layer	W	Continuous, Mat		2.5 2.5～(5.0)	5.0 5.0～(10.0)
Instant settlement	W	Continuous		1.5	2.5

[Note] For consolidation layer, it is the settlement at the end of consolidation (calculated value ignoring the rigidity of the construction); for the others, they are the instantaneous settlements, and the values in ( ) are for the case of sufficient rigidity such as double slab.

Source: Road Earthwork - Guideline for Architectural Foundation Design (Architectural Institute of Japan) p.154

3.3.2 Engineering Analysis

3.3.2.1 Settlement Analysis

For each cross section, the settlement analysis at the planned embankment height is carried out by the following analysis method, and the settlement amount as well as the settling time are calculated.

(1) Analysis Method

1) Calculation Method of the Amount of Settlement

The consolidated settlement (Sc) of the cohesive soil is obtained by focusing on the change of the gap ratio due to the embankment load and calculated by the Δe method in the following equation. In addition, the instantaneous settlement (Si) of the sandy soil layer is calculated by the same formula, and the pressure-void ratio curve of sand by N-value in that case is shown in Figure 3.3.9

$$Sc = \frac{e_0 - e_1}{1 + e_0} \cdot H$$

Where, Sc: Consolidation subsidence (m)

e0: Initial void ratio (void ratio for effective overburden pressure P0 + q0)

e1: Void ratio after consolidation (void ratio for P0 + ΔP from the e-log p curve)

ΔP: Vertical increase stress due to consolidation load

H: Layer thickness of consolidation layer (m)

However, e0＞e1.

2) Calculation Method of Settling Time

The consolidation settling time is calculated by the following equation considering one-dimensional consolidation where drainage is performed only in the vertical direction:

$$T = \frac{D^2}{C_V} \cdot Tv$$

Where, T: Consolidated settling time (day)

D: Maximum drainage distance (cm)

Cv: Consolidation coefficient (cm^2/day)

Tv: Time coefficient

It should be noted that the time coefficient (Tv) is a coefficient that changes according to consolidation degree (U) (Figure 3.3.9).

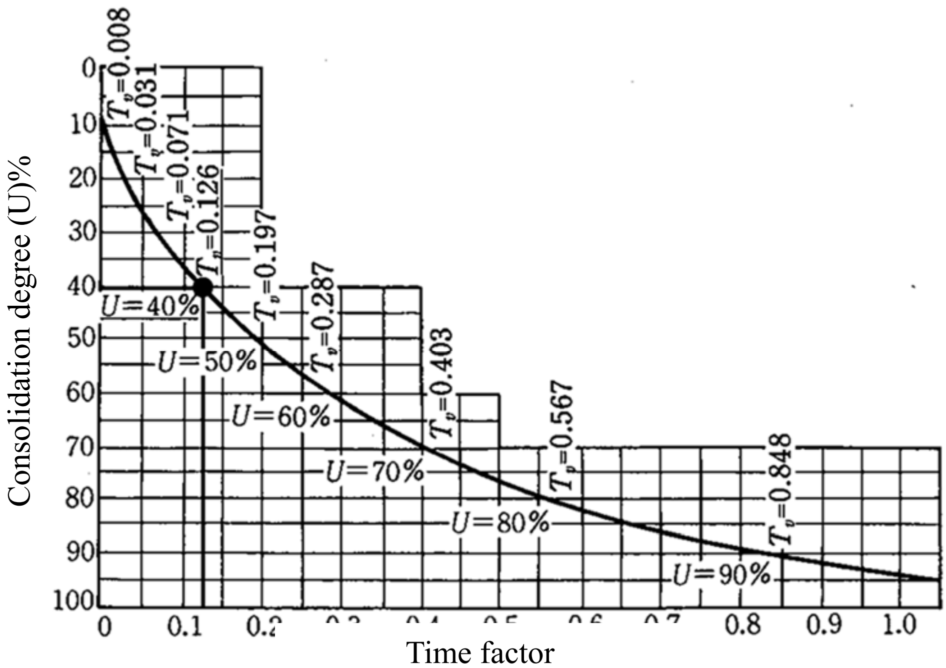


Figure 3.3.9 Relationship between the Average Consolidation Degree (U) and the Time Coefficient (Tv) in the Entire Consolidation Layer

(Pore water pressure right after loading Δu0 = constant)

Source: Road Earthwork - Guideline for Road Earthwork of Soft Ground Treatment (Japan Road Association) p.128

(2) Analysis Result

The results of the settlement analysis are shown in Table 3.3.25.

Table 3.3.25 Results of Settlement Analysis

Area	Block No.	Planned Embankment Height (m)	Embankment Speed (cm/day)	Embankment Period (day)	Total Settlement (cm)	Residual Settlement (cm)	Settlement Period (day)
Thanlyin side	1	2.10	5	43	59.566	18.562	480
	2	3.51	5	71	73.548	10.786	480
	3	3.97	5	80	75.908	9.752	480
Thaketa side	4	4.57	5	92	69.851	27.632	390
	5	4.81	5	96	34.530	2.915	390
	6	2.85	5	57	22.677	3.436	390
On-ramp	7	2.57	3	86	47.125	8.146	480
	8	4.55	3	152	54.301	5.620	480

Source: JICA Study Team

3.3.2.2 Transformation Analysis

(1) Analysis Method

The relationship between the settlement shape of the embankment and the distance of the lateral ground displacement is shown in Figure 3.3.10.

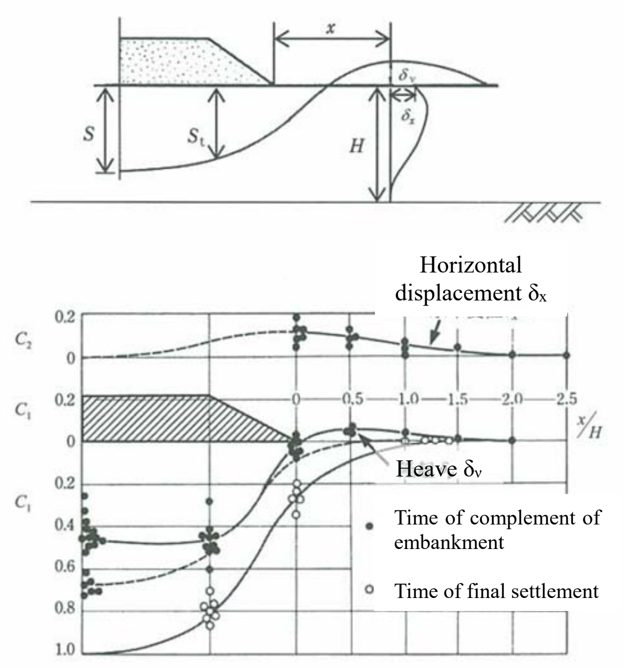


Figure 3.3.10 Settlement Shape of Embankment and Lateral Influence

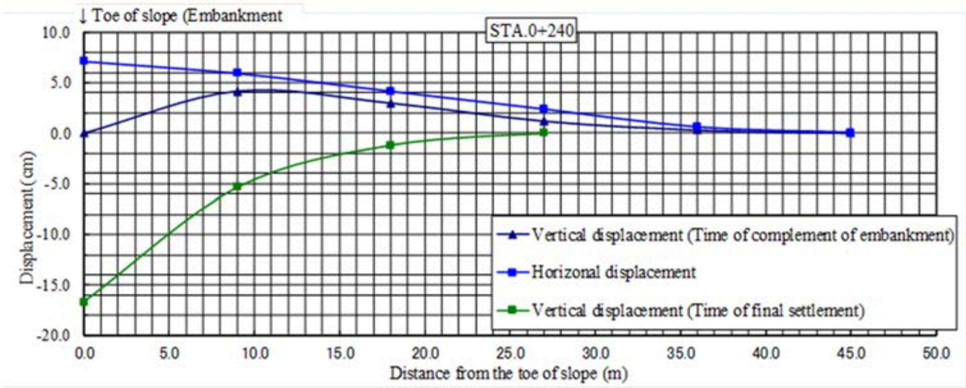
Source: Road Earthwork - Guideline for Road Earthwork of Soft Ground Treatment (Japan Road Association), p.128

As shown in Figure 3.3.10 , when there is a structure within about two times the thickness of the soft layer from the toe of the embankment slope, influence from the embankment is possible.

(2) Analysis Result

The distance from the embankment, settlement, and ground inclination angle are analyzed in each examined cross section.

The results of the analysis are shown from Figure 3.3.11 to Figure 3.3.18.



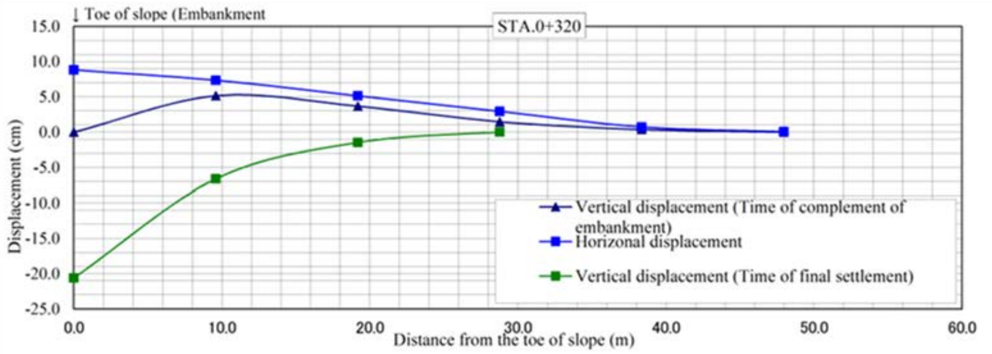
	Distance from the toe of slope (m)					
	0.0	9.0	18.0	27.0	36.0	45.0
Heave of lateral ground $\delta v = C_1 \cdot S$ (cm)	0.0	4.2	3.0	1.2	0.3	0.0
Horizontal displacement of lateral ground $\delta x = C_2 \cdot S$ (cm)	7.1	6.0	4.2	2.4	0.6	0.1
Settlement $St = C_1 \cdot S$ (cm)	16.7	5.4	1.2	0.0	-	-
Distance over the displacement of lateral ground L (m) = C3·H =					36.0	
Ground inclination angle (Heave) $\alpha v/1000$	4.63	-1.32	-1.99	-0.99	-0.33	0.00
Ground inclination angle (Settlement) $\alpha x/1000$	-12.58	-4.63	-1.32	-	-	-

Maximum of total settlement (S(m)) = 0.60

Soft layer thickness (H(m)) = 18.00

Source: JICA Study Team

Figure 3.3.11 Ground Influence Summary (STA.0+240)



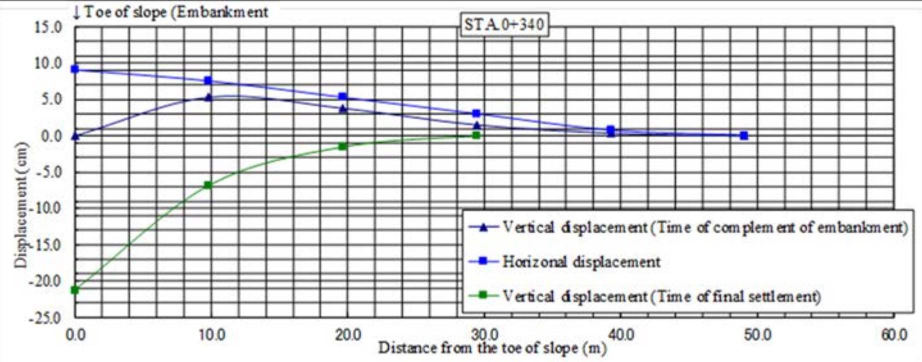
	Distance from the toe of slope (m)					
	0.0	9.6	19.2	28.8	38.4	48.0
Heave of lateral groun(d) δv = C1・S (cm)	0.0	5.1	3.7	1.5	0.4	0.0
Horizontal displacement of δx = C2・S (cm)	8.8	7.4	5.1	2.9	0.7	0.1
Settlement St = C1・S (cm)	20.6	6.6	1.5	0.0	-	-
(D)istance over the displacement of lateral ground L (m) = C3・H					38.4	
Ground inclination angle (Heave) αv/1000	5.37	-1.53	-2.30	-1.15	-0.38	0.00
Ground inclination angle (Settlement) αx/1000	-14.57	-5.37	-1.53	-	-	-

(M)aximum of total settlement S(m) = 0.74

Soft layer thickness H(m) = 19.18

Source: JICA Study Team

Figure 3.3.12 Ground Influence Summary (STA.0+320)



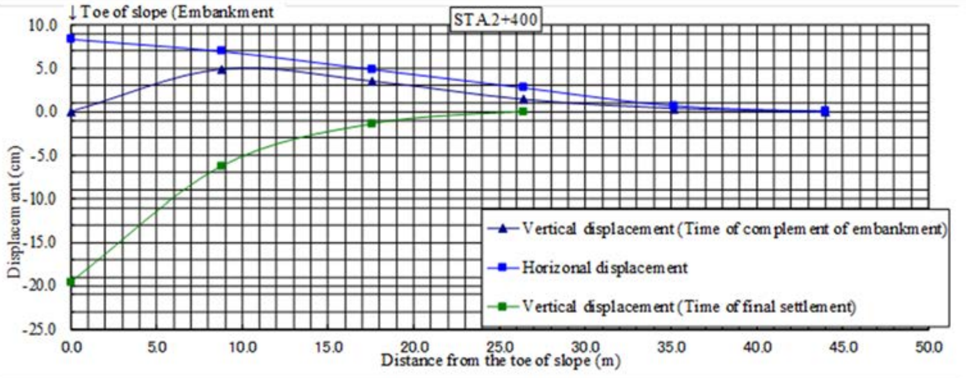
	Distance from the toe of slope (m)					
	0.0	9.8	19.6	29.4	39.3	49.1
Heave of lateral ground δv = C1・S (cm)	0.0	5.3	3.8	1.5	0.4	0.0
Horizontal displacement of lateral ground δx = C2・S (cm)	9.1	7.6	5.3	3.0	0.8	0.1
Settlement St = C1・S (cm)	21.3	6.8	1.5	0.0	-	-
Distance over the displacement of lateral ground L (m) = C3・H					39.3	
Ground inclination angle (Heave) αv/1000	5.41	-1.55	-2.32	-1.16	-0.39	0.00
Ground inclination angle (Settlement) αx/1000	-14.69	-5.41	-1.55	-	-	-

Maximum of total settlement S(m) = 0.76

Soft layer thickness H(m) = 19.63

Source: JICA Study Team

Figure 3.3.13 Ground Influence Summary (STA.0+340)



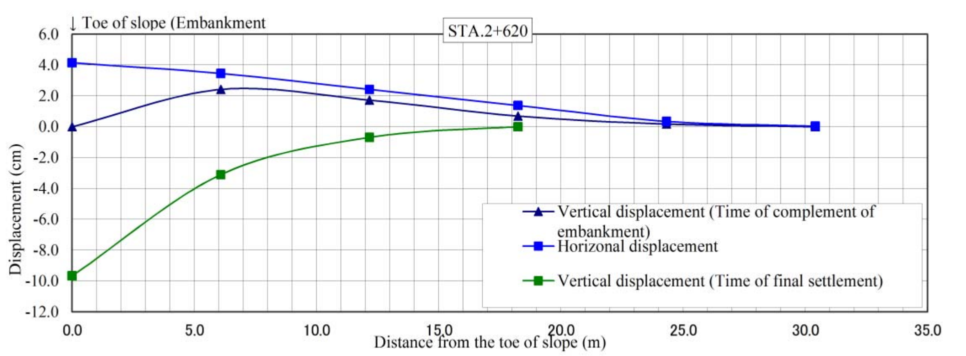
	Distance from the toe of slope (m)					
	0.0	8.8	17.6	26.4	35.2	44.0
Heave of lateral ground $\delta v = C_1 \cdot S$ (cm)	0.0	4.9	3.5	1.4	0.3	0.0
Horizontal displacement of lateral ground $\delta x = C_2 \cdot S$ (cm)	8.4	7.0	4.9	2.8	0.7	0.1
Settlement $St = C_1 \cdot S$ (cm)	19.6	6.3	1.4	0.0	-	-
Distance over the displacement of lateral ground L (m) = C3·H =					35.2	
Ground inclination angle (Heave) αv/1000	5.56	-1.59	-2.38	-1.19	-0.40	0.00
Ground inclination angle (Settlement) αx/1000	-15.08	-5.56	-1.59	-	-	-

Maximum of total settlement S(m) = 0.70

Soft layer thickness H(m) = 17.60

Source: JICA Study Team

Figure 3.3.14 Ground Influence Summary (STA.2+400)



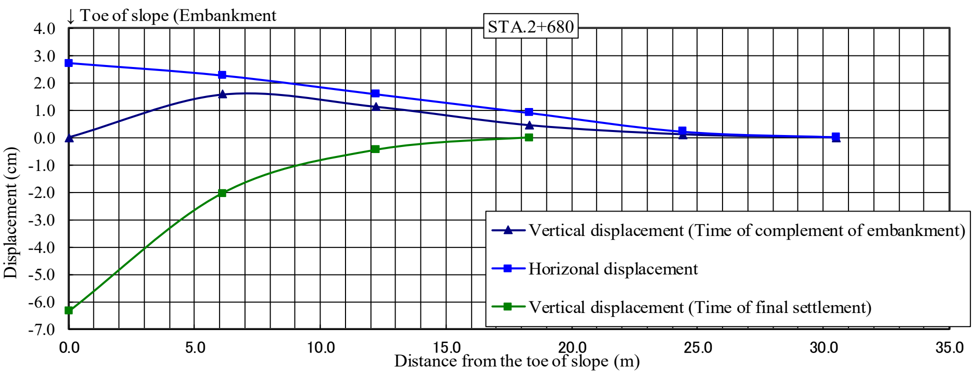
	Distance from the toe of slope (m)					
	0.0	6.1	12.2	18.2	24.3	30.4
Heave of lateral ground $\delta v = C_1 \cdot S$ (cm)	0.0	2.4	1.7	0.7	0.2	0.0
Horizontal displacement of $\delta x = C_2 \cdot S$ (cm)	4.1	3.5	2.4	1.4	0.3	0.0
Settlement $St = C_1 \cdot S$ (cm)	9.7	3.1	0.7	0.0	-	-
Distance over the displacement of lateral ground L (m) =					24.3	
Ground inclination angle (Heave) αv/1000	3.98	-1.14	-1.70	-0.85	-0.28	0.00
Ground inclination angle (Settlement) αx/1000	-10.79	-3.98	-1.14	-	-	-

Maximum of total settlement S(m) = 0.35

Soft layer thickness H(m) = 12.16

Source: JICA Study Team

Figure 3.3.15 Ground Influence Summary (STA.2+620)



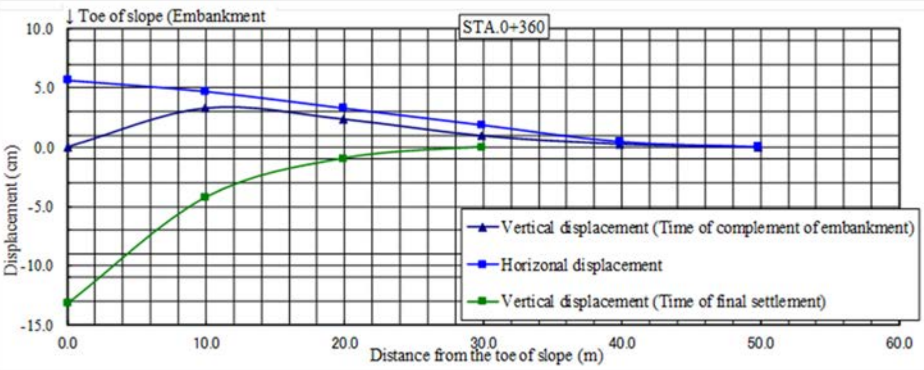
	Distance from the toe of slope (m)					
	0.0	6.1	12.2	18.3	24.4	30.5
Heave of lateral ground δv = C1・S (cm)	0.0	1.6	1.1	0.5	0.1	0.0
Horizontal displacement of lateral ground δx = C2・S (cm)	2.7	2.3	1.6	0.9	0.2	0.0
Settlement St = C1・S (cm)	6.3	2.0	0.5	0.0	-	-
Distance over the displacement of lateral ground L (m) = C3・H					24.4	
Ground inclination angle (Heave) αv/1000	2.60	-0.74	-1.12	-0.56	-0.19	0.00
Ground inclination angle (Settlement) αx/1000	-7.06	-2.60	-0.74	-	-	-

Maximum of total settlement S(m) = 0.23

Soft layer thickness H(m) = 12.20

Source: JICA Study Team

Figure 3.3.16 Ground Influence Summary (STA.2+680)



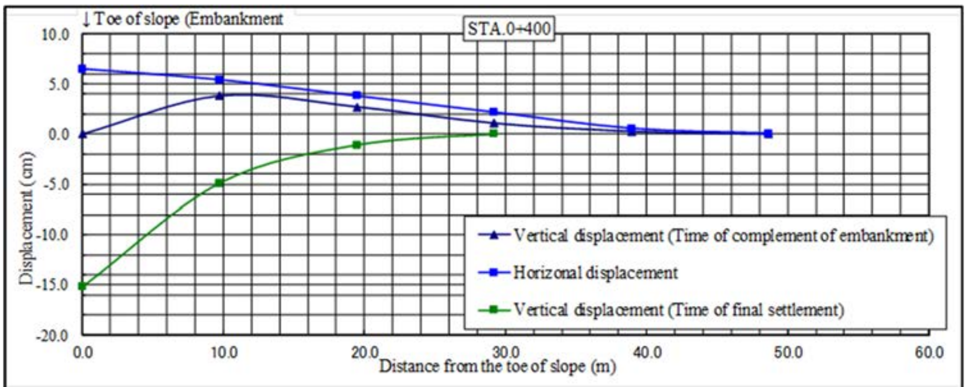
	Distance from the toe of slope (m)					
	0.0	10.0	19.9	29.9	39.8	49.8
Heave of lateral ground δv = C1・S (cm)	0.0	3.3	2.4	0.9	0.2	0.0
Horizontal displacement of lateral ground δx = C2・S (cm)	5.7	4.7	3.3	1.9	0.5	0.0
Settlement St = C1・S (cm)	13.2	4.2	0.9	0.0	-	-
Distance over the displacement of lateral ground L (m) =					39.8	
Ground inclination angle (Heave) αv/1000	3.31	-0.95	-1.42	-0.71	-0.24	0.00
Ground inclination angle (Settlement) αx/1000	-8.99	-3.31	-0.95	-	-	-

Maximum of total settlement S(m) = 0.47

Soft layer thickness H(m) = 19.92

Source: JICA Study Team

Figure 3.3.17 Ground Influence Summary (STA.0+360)



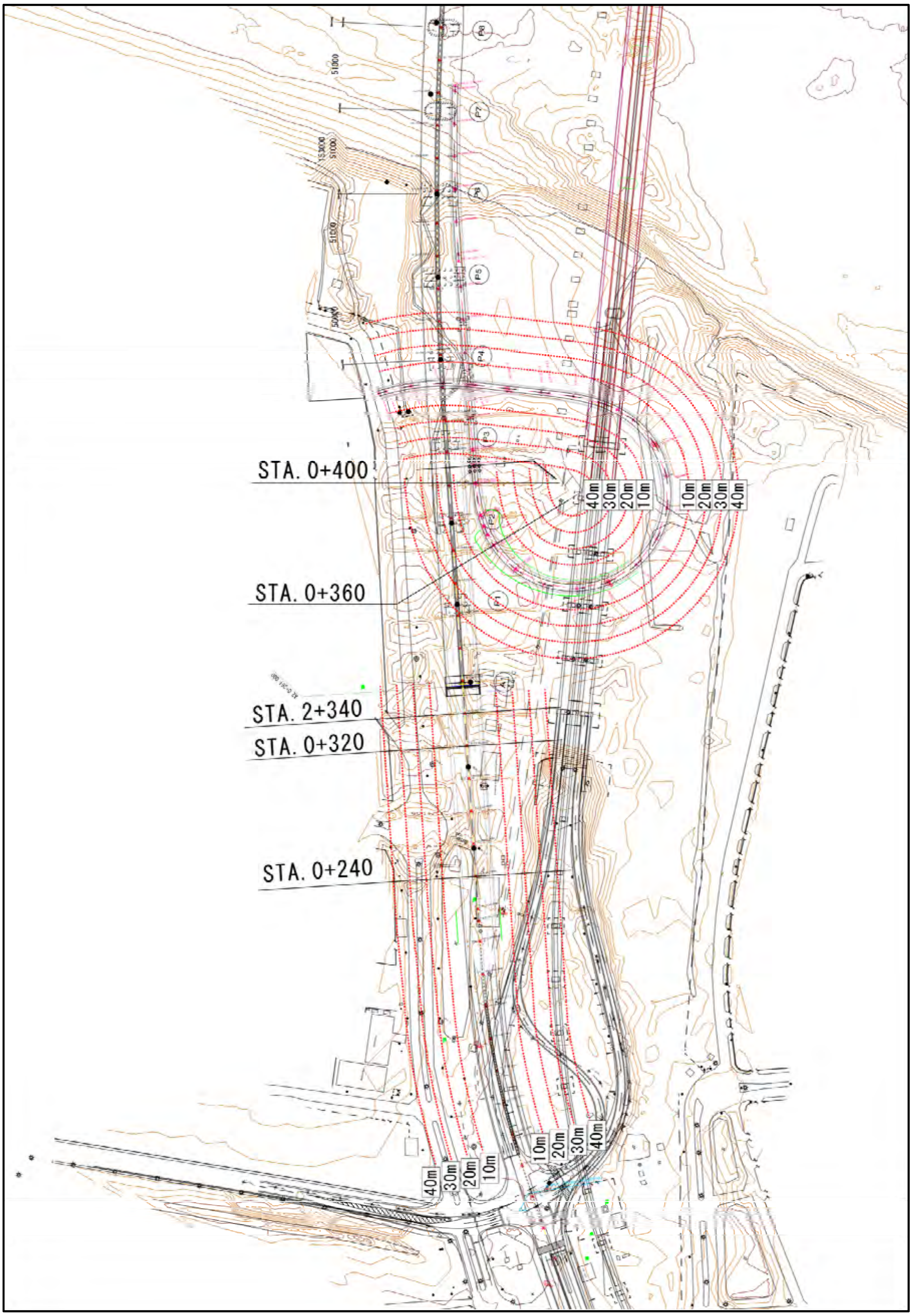
	Distance from the toe of slope (m)					
	0.0	9.7	19.5	29.2	38.9	48.7
Heave of lateral ground $\delta v = C_1 \cdot S$ (cm)	0.0	3.8	2.7	1.1	0.3	0.0
Horizontal displacement of lateral ground $\delta x = C_2 \cdot S$ (cm)	6.5	5.4	3.8	2.2	0.5	0.1
Settlement $St = C_1 \cdot S$ (cm)	15.2	4.9	1.1	0.0	-	-
Distance over the displacement of lateral ground L (m) = $C_3 \cdot H$					38.9	
Ground inclination angle (Heave) $\alpha v/1000$	3.91	-1.12	-1.67	-0.84	-0.28	0.00
Ground inclination angle (Settlement) $\alpha x/1000$	-10.60	-3.91	-1.12	-	-	-

Maximum of total settlement S(m) = 0.54

Soft layer thickness H(m) = 19.46

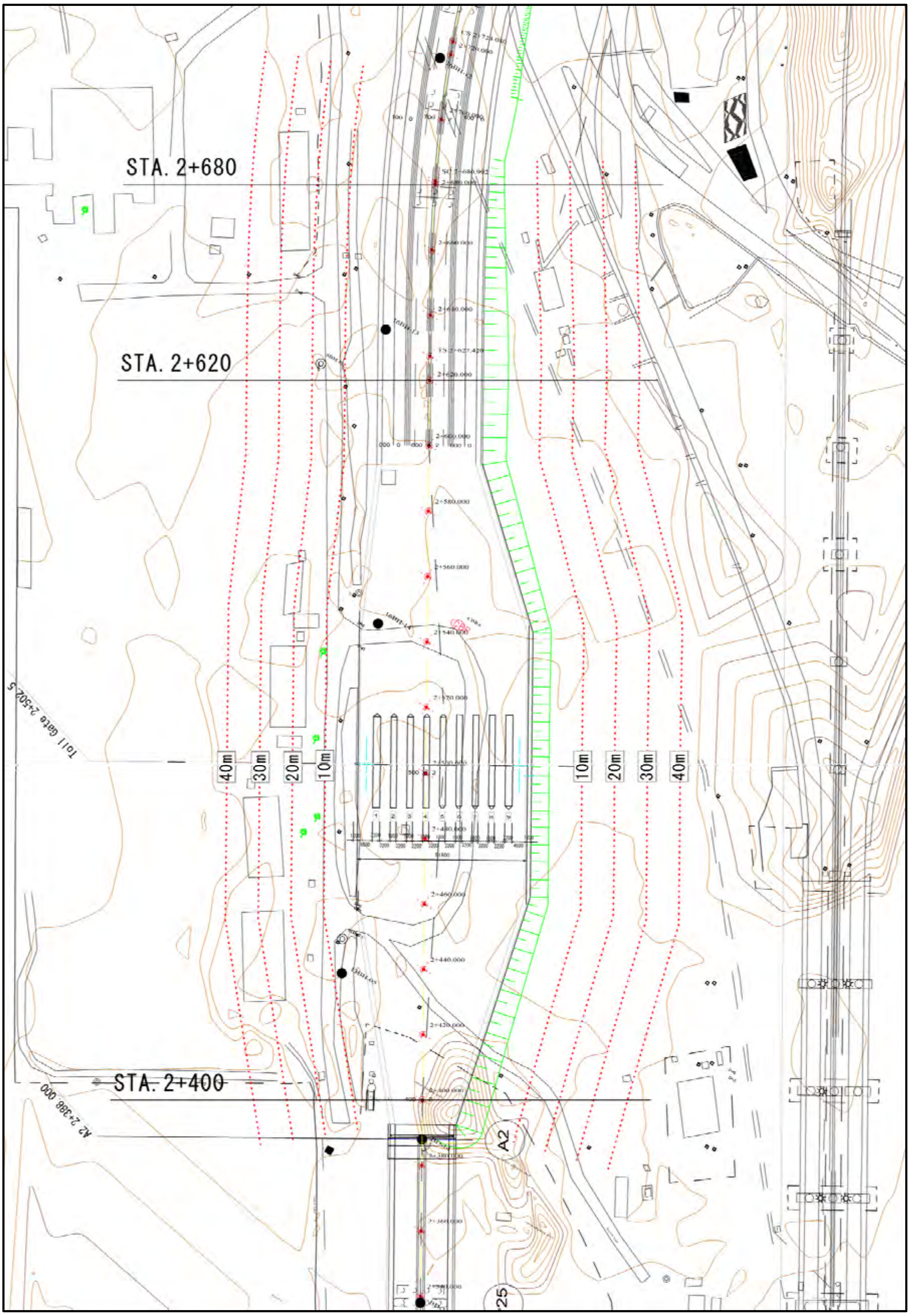
Source: JICA Study Team

Figure 3.3.18 Ground Influence Summary (STA.0+400)



Source: JICA Study Team

Figure 3.3.19 Distance from the Examined Section to the Toe of Slope (Thanlyin Side and On-ramp)



Source: JICA Study Team

Figure 3.3.20 Distance from the Examined Section to the Toe of Slope (Thaketa Side)

Based on Figure 3.3.11 to Figure 3.3.20, the total settlement and the ground inclination angle for Block 1 to 3, Block 4 to 6, and Block 8 are within the allowable values at a distance where the existing structures are confirmed.

However, for Block 7, existing bridges are present at a distance indicating a value that is greater than the allowable value; therefore, in case of embankment, construction displacement may occur.

3.3.2.3 Liquefaction Research

(1) Research Method

1) Soil Layer that Needs to be Examined for Liquefaction

Layers that could liquefy are determined based on the following conditions:

- Saturated soil layer having a groundwater level within 10 m from the ground surface and existing at a depth within 20 m from the ground surface
- Soil layer having a fine grain content (FC) of 35% or less, or soil layer having a plasticity index (IP) of 15 or less when FC exceeds 35%
- Soil layer having an average particle diameter (D50) of 10 mm or less and 10% particle size (D10) of 1 mm or less

2) Determination of Liquefaction

The resistivity to liquefaction (FL) of the soil layer that needs to be examined for liquefaction, based on item 1) above, will be calculated. The soil layer will be considered as liquefying if FL is 1.0 or less.

$$\mathrm{F_L} = \mathrm{R/L}$$
$$\mathrm{R} = C_w \cdot R_L$$
$$\mathrm{L} = r_d \cdot k_h \cdot (\sigma_v \cdot \sigma_v')$$
$$r_\mathrm{d} = 1.0 - 0.015\mathrm{x}$$

Where, F_L: Resistivity to liquefaction

R: Dynamic shear strength ratio

L: Shear stress ratio at the time of earthquake

C_w: Correction factor due to earthquake ground motion characteristics

R_L: Repeated triaxial strength ratio

r_d: Reduction factor in the depth direction of shear stress ratio at the time of earthquake

k_h: Design horizontal seismic intensity

σ_V: Total overburden pressure at depth x(m) from the ground surface (kN/m^2)

σ_V': Effective overburden pressure at depth x(m) from the ground surface (kN/m^2)

x: Depth from the ground surface (m)

3) Repeated Triaxial Strength Ratio

R_L used in the above item 2) is obtained by the following formula:

$$RL = \begin{cases} 0.0882 \cdot \sqrt{Na/1.7} & (Na < 14) \\ 0.0882 \cdot \sqrt{Na/1.7} + 1.6 \times 10^{-6} \cdot (Na - 14)^{4.5} & (14 \leqq Na) \end{cases}$$

$Na = c_1 \cdot N_1 + c_2$

$N_1 = 170 \cdot N / (\sigma_{vb} + 70)$

$$c_1 = \begin{cases} 1 & (0\% \leqq Fc < 10\%) \\ (Fc + 40)/50 & (10\% \leqq Fc < 60\%) \\ Fc/20 - 1 & (60\% \leqq Fc) \end{cases}$$

$$c_2 = \begin{cases} 2 & (0\% \leqq Fc < 10\%) \\ (Fc - 10)/18 & (10\% \leqq Fc) \end{cases}$$

Where, R_L: Repeated triaxial strength ratio

N: N-value obtained from standard penetration test

N_1: N-value converted to be equivalent to the effective overburden pressure of 100 kN/m^2

Na: Corrected N-value in consideration of influence of grain size

σ'_{vb}: Effective overburden pressure at the depth from the ground surface when performing standard penetration test (kN/m^2)

c_1,c_2: Correction factor of N-value depending on fine grain content

Fc: Fine grain content (%)

D_{50}: 50% particle size (mm)

4) Design Horizontal Seismic Intensity

The design horizontal seismic intensity (kh) used for determining the liquefied ground is obtained from the standard value of design horizontal seismic intensity shown in

Table 3.3.26.

Table 3.3.26 Standard Value of Design Horizontal Seismic Intensity Used for Determining Liquefaction

Seismic motion		Ground type		
		Type 1	Type 2	Type 3
Seismic motion level 1		0. 12	0. 15	0. 18
Seismic motion level 2	Type 1	0. 30	0. 35	0. 40
	Type 2	0. 80	0. 70	0. 60

Source: Road Earthwork - Guideline for Road Earthwork of Soft Ground Treatment (Japan Road Association) p.168

(2) Research Result

The boring data used in each examined section under this liquefaction research is shown in Table 3.3.27. This is the boring result investigated at the closest position from the examined section.

Table 3.3.27 Boring Data Used for Liquefaction Research

Area	Block	Examined Section	Boring Used
Thanlyin side	Block1	STA.0+240	BH-BD-25
	Block2	STA.0+320	BH-BD-24
	Block3	STA.0+340	BH-BD-23
Thaketa side	Block4	STA.2+400	BH-BD-14
	Block5 Block6	STA.2+620 STA.2+680	(16)BH-13
On-ramp	Block7 Block8	STA.0+360 STA.0+400	BH-BD-21

Source: JICA Study Team

The research results are shown in Table 3.3.28 to Table 3.3.30.

Table 3.3.28 Liquefaction Determination Result (1)

Area	Thanlyin side	
Block	Block 1	
Depth (m)	BH-BD-25	
	Soil Layer Name	Liquefaction Potential
0.60	Filled Soil	No Liquefaction
1.30	Filled Soil	
2.30	CLAY-I	
4.30	CLAY-I	
5.30	CLAY-I	
7.30	CLAY-I	Liquefaction
8.30	Sandy CLAY-I	No Liquefaction
10.30	Sandy CLAY-I	Liquefaction
11.30	Sandy CLAY-I	
12.30	Sandy CLAY-I	No Liquefaction
13.30	Sandy CLAY-I	
14.30	Silty SAND-I	
15.30	Silty SAND-I	Liquefaction
16.30	Silty SAND-I	
17.30	Silty SAND-I	
18.30	CLAY-AII	No Liquefaction
19.30	CLAY-AII	

Area	Thanlyin side	
Block	Block 2	
Depth (m)	BH-BD-24	
	Soil Layer Name	Liquefaction Potential
0.70	Filled Soil	No Liquefaction
1.30	Filled Soil	
2.30	CLAY-I	
4.30	CLAY-I	
5.30	CLAY-I	Liquefaction
6.30	CLAY-I	
7.30	CLAY-I	No Liquefaction
9.30	CLAY-I	
10.80	Sandy CLAY-I	Liquefaction
11.30	Sandy CLAY-I	
12.30	Sandy CLAY-I	No Liquefaction
13.30	Sandy CLAY-I	
14.30	Silty SAND-I	Liquefaction
15.30	Silty SAND-I	No Liquefaction
16.30	Silty SAND-I	
17.30	Silty SAND-I	
18.30	Silty SAND-I	
19.30	CLAY-AII	

Source: JICA Study Team

Table 3.3.29 Liquefaction Determination Result (2)

<table>
<tr><th>Area</th><th colspan="2">Thanlyin side</th></tr>
<tr><th>Block</th><th colspan="2">Block 3</th></tr>
<tr><th rowspan="2">Depth (m)</th><th colspan="2">BH-BD-23</th></tr>
<tr><th>Soil Layer Name</th><th>Liquefaction Potential</th></tr>
<tr><td>0.42</td><td>Filled Soil</td><td rowspan="6">No Liquefaction</td></tr>
<tr><td>1.30</td><td>Filled Soil</td></tr>
<tr><td>2.30</td><td>CLAY-I</td></tr>
<tr><td>3.80</td><td>CLAY-I</td></tr>
<tr><td>5.30</td><td>CLAY-I</td></tr>
<tr><td>6.80</td><td>CLAY-I</td></tr>
<tr><td>7.30</td><td>Sandy CLAY-I</td><td rowspan="4">Liquefaction</td></tr>
<tr><td>9.30</td><td>Sandy CLAY-I</td></tr>
<tr><td>10.80</td><td>Sandy CLAY-I</td></tr>
<tr><td>11.30</td><td>Sandy CLAY-I</td></tr>
<tr><td>12.30</td><td>Sandy CLAY-I</td><td rowspan="2">No Liquefaction</td></tr>
<tr><td>13.30</td><td>Sandy CLAY-I</td></tr>
<tr><td>14.30</td><td>Silty SAND-I</td><td rowspan="6">Liquefaction</td></tr>
<tr><td>15.30</td><td>Silty SAND-I</td></tr>
<tr><td>16.30</td><td>Silty SAND-I</td></tr>
<tr><td>17.30</td><td>Silty SAND-I</td></tr>
<tr><td>18.30</td><td>Silty SAND-I</td></tr>
<tr><td>19.30</td><td>Silty SAND-I</td></tr>
</table>

<table>
<tr><th>Area</th><th colspan="2">Thaketa side</th></tr>
<tr><th>Block</th><th colspan="2">Block 4</th></tr>
<tr><th rowspan="2">Depth (m)</th><th colspan="2">BH-BD-14</th></tr>
<tr><th>Soil Layer Name</th><th>Liquefaction Potential</th></tr>
<tr><td>1.30</td><td>CLAY-I</td><td rowspan="6">No Liquefaction</td></tr>
<tr><td>2.30</td><td>CLAY-I</td></tr>
<tr><td>3.80</td><td>CLAY-I</td></tr>
<tr><td>4.40</td><td>CLAY-I</td></tr>
<tr><td>5.30</td><td>CLAY-I</td></tr>
<tr><td>6.80</td><td>CLAY-I</td></tr>
<tr><td>7.30</td><td>CLAY-I</td><td rowspan="2">Liquefaction</td></tr>
<tr><td>8.30</td><td>Silty SAND-I</td></tr>
<tr><td>9.30</td><td>Silty SAND-I</td><td rowspan="3">No Liquefaction</td></tr>
<tr><td>10.80</td><td>Silty SAND-I</td></tr>
<tr><td>11.30</td><td>Silty SAND-I</td></tr>
<tr><td>12.30</td><td>Silty SAND-I</td><td>Liquefaction</td></tr>
<tr><td>13.30</td><td>Silty SAND-I</td><td>No Liquefaction</td></tr>
<tr><td>14.30</td><td>Silty SAND-I</td><td rowspan="2">Liquefaction</td></tr>
<tr><td>15.30</td><td>Silty SAND-I</td></tr>
<tr><td>16.30</td><td>Silty SAND-I</td><td rowspan="2">No Liquefaction</td></tr>
<tr><td>17.30</td><td>Silty SAND-I</td></tr>
<tr><td>18.30</td><td>Silty SAND-II</td><td>Liquefaction</td></tr>
<tr><td>19.30</td><td>Silty SAND-II</td><td>No Liquefaction</td></tr>
</table>

Source: JICA Study Team

Table 3.3.30 Liquefaction Determination Result (3)

Area	Thaketa side	
Block	Block 5, Block 6	
Depth (m)	(16)BH-13	
	Soil Layer Name	Liquefaction Potential
1.00	Filled Soil	No Liquefaction
2.00	Filled Soil	
3.00	CLAY-I	
4.00	CLAY-I	
5.00	CLAY-I	
6.00	Silty SAND-I	Liquefaction
7.00	Silty SAND-I	
8.00	Silty SAND-I	
9.00	Silty SAND-I	No Liquefaction
10.00	Silty SAND-I	
11.00	Silty SAND-I	
12.00	Silty SAND-I	Liquefaction
13.00	Sandy SILT	
14.00	Sandy SILT	
15.00	Sandy SILT	
16.00	Sandy SILT	
17.00	Sandy SILT	No Liquefaction
18.00	Silty SAND-II	Liquefaction
19.00	Silty SAND-II	No Liquefaction
20.00	Silty SAND-II	

Area	On-ramp	
Block	Block 7, Block 8	
Depth (m)	BH-BD-21	
	Soil Layer Name	Liquefaction Potential
1.30	Filled Soil	No Liquefaction
2.25	CLAY-I	
3.30	CLAY-I	
4.30	CLAY-I	
5.30	CLAY-I	
6.30	CLAY-I	
7.30	CLAY-I	
8.25	CLAY-I	Liquefaction
9.30	CLAY-I	
10.30	CLAY-I	
11.30	Sandy CLAY-I	
12.30	Sandy CLAY-I	
13.30	Silty SAND-I	
14.30	Silty SAND-I	No Liquefaction
15.30	Silty SAND-I	
16.30	Silty SAND-I	
17.30	Silty SAND-I	
18.30	Silty SAND-I	
19.30	Silty SAND-I	

Source: JICA Study Team

3.3.2.4 Stability Analysis

(1) Analysis Method

Stability analysis is carried out according to the following formula in order to determine the safety factor (Fs) against the sliding failure of the embankment:

$$Fs = \frac{\sum (c \cdot \ell + w' \cdot \cos\alpha \cdot \tan\phi)}{\sum w \cdot \sin\alpha}$$

Where, Fs: Safety factor

c: Cohesion of soil (kN/m^2)

φ: shear resistance angle of soil (°)

w': Effective weight of strips considering buoyancy below the groundwater level (kN/m)

w: Total weight of the split piece soil

α: Angle between the straight line connecting the center of the slip surface cut by each split piece and the center of the slip circle and the vertical line (°)

l: Arc length of the slip surface cut by split piece

(2) Analysis Result

Stability analysis is conducted for three cases, namely: normal, at the time of earthquake, and at the time of liquefaction, with respect to the planned embankment height calculated for each block.

The traffic load used is 11.6 kN/m^2 for normal.

Since the analysis is performed only under level 1 seismic motion as well as the ground type is Type III, the design horizontal seismic intensity used is 0.12. The standard value of the design horizontal seismic intensity is shown in Table 3.3.31.

Table 3.3.31 Standard Value of Design Horizontal Seismic Intensity

Seismic motion		Ground type		
		Type 1	Type 2	Type 3
Seismic motion level 1	For inertial force	0. 08	0. 10	0. 12
Seismic motion level 2	For inertial force	0. 16	0. 20	0. 24

Source: Road Earthwork - Guideline for Road Earthwork of Soft Ground Treatment (Japan Road Association) p.171

Table 3.3.32 shows the stability analysis results.

Table 3.3.32 Stability Analysis Results

Area	Block No.	Design Embankment Height (m)	Consolidation Degree U (%)		Minimum Safety Factor Fs_{min}			
			At the Time of Construction	At the Time of Service	Normal		At the Time of Earthquake	At the Time of Liquefaction
					At the Time of Construction	At the Time of Service		
Thanlyin side	1	2.10	25.0	90	1.450	1.450	1.226	1.564
	2	3.51	39.2	90	1.434	1.434	0.876	1.199
	3	3.97	43.4	90	1.202	1.202	0.840	1.047
Thaketa side	4	4.57	35.7	90	1.226	1.226	0.895	1.379
	5	4.81	91.4	90	1.282	1.282	1.027	1.347
	6	2.85	82.6	90	1.880	1.880	1.466	1.772
On-ramp	7	2.57	83.0	90	1.635	1.635	1.207	2.413
	8	4.55	90.7	90	1.161	1.161	1.072	0.819

Source: JICA Study Team

3.3.2.5 Lateral Movement Research

(1) Research Result

The lateral movement of each abutment has been studied in each abutment design. As soft ground treatment, countermeasures for abutments that may have lateral movements will be considered based on the examined results.

Table 3.3.33 shows the results of examination of lateral movement of each abutment.

Table 3.3.33 Lateral Movement Result

Area	Abutment	I Value
Thanlyin side	A1	2.000 (≧1.20)
Thaketa side	A2	0.762 (<1.20)
	AF1	0.391 (<1.20)
On-ramp	AO1	3.167 (≧1.20)

Source: JICA Study Team

3.3.2.6 Retaining Wall Bearing Capacity Research

(1) Research Method

In this Project, mechanically-stabilized earth wall and gravity wall will be installed as retaining walls. The ground reaction force of the retaining wall and the allowable bearing capacity of the ground in the installed retaining wall section will be compared and then the necessity of countermeasures under the retaining wall will be determined.

In this section, only the allowable bearing capacity of the ground just under the mechanically-stabilized earth wall will be studied, and the examination of the bearing capacity depends on the comparison of the ground reaction force and the allowable bearing capacity of the mechanically-stabilized earth wall. The examination of the ground reaction force of the mechanically-stabilized earth wall and the studies for the gravity wall will be performed in Section 3.4 (Road Structure Design).

In addition, as the ground reaction force of the mechanically-stabilized earth wall is examined using the cross section shown in Table 3.3.34, the same cross section will be used for the allowable bearing capacity of the ground.

Table 3.3.34 Bearing Capacity of the Examined Section of Retaining Wall

Area	Block No.	Examined Section Position	Number of Examined Section
Thanlyin	3	STA.0+340	1
Thaketa	4	STA.2+400	1
	6	STA.2+620	1
On-ramp	8	STA.0+400	1

Source: JICA Study Team

The allowable bearing capacity of the ground just under the retaining wall is determined using the following formula multiplied by the safety factor:

$$q_d = \alpha c N_c + \frac{1}{2}\beta \gamma_1 B N_\gamma + \gamma_2 D_f N_q$$

Where, q_d: Ultimate bearing capacity of the foundation base (kN/m^2)

c: Cohesion of the ground below the foundation base (kN/m^2)

γ_1: Unit weight of the ground below the foundation base (kN/m^3)

(However, submerged unit weight is used when under the groundwater level.)

γ_2: Unit weight of the ground upon the foundation base (kN/m^3)

(However, submerged unit weight is used when under the groundwater level.)

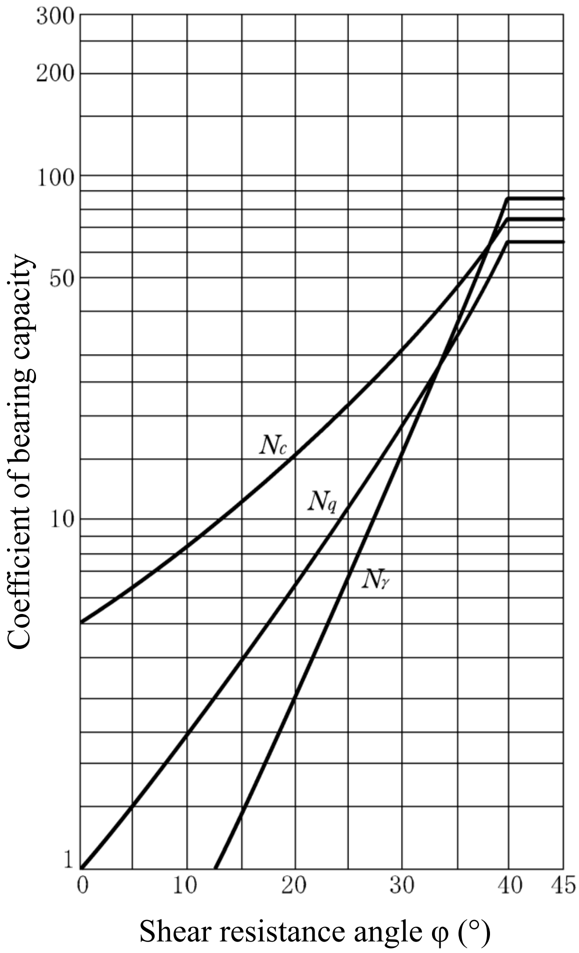
α, β: Shape factor of the foundation base

B: Foundation width (m)

D_f: Effective depth of embedment (m)

N_c, N_q, N_γ: coefficient of bearing capacity

The values of the coefficient of bearing capacity will be obtained from Figure 3.3.21.



Source: Specification for Highway Bridges 2012 (Japan Road Association) p.331

Figure 3.3.21 Graph for Finding Coefficient of Bearing Capacity

(2) Research Result

The research results are shown in Table 3.3.35.

Table 3.3.35 Research Results of Bearing Capacity of Retaining Wall

Area	Block	Location of Examination	Geological Condition		Retaining Wall Condition	Result
			Bearing Stratum	Allowable Bearing Capacity (kN/m^2)	Ground Reaction Force (kN/m^2)	
Thanlyin	Block3	STA.0+340	CLAY-I	35.8	152.96	NO
			Sandy CLAY-I	71.2	258.69	NO
			Silty SAND-I	1795.1	369.88	YES
Thaketa	Block4	STA.2+400	CLAY-I	54.9	206.37	NO
			Silty SAND-I	955.6	329.12	YES
			Silty SAND- II	2563.3	472.17	YES
	Block6	STA.2+620	Silty SAND-I	50.4	194.52	NO
			Silty SAND-I	676.8	286.94	YES
			Sandy SILT	171.4	401.55	NO
			Silty SAND- II	2691.2	503.86	YES
On-ramp	Block8	STA.0+400	CLAY-I	47.6	260.66	NO
			Sandy CLAY-I	99.2	413.85	NO
			Silty SAND-I	1893.5	454.14	YES

Source: JICA Study Team

3.3.2.7 Summary of the Analysis Result

(1) Thanlyin Side

The analysis results are summarized in Table 3.3.36.

Table 3.3.36 Summary of Analysis Results (Thanlyin Side)

<table>
<tr><th colspan="3">Block</th><th colspan="2">Block 1</th><th colspan="2">Block 2</th><th colspan="2">Block 3</th></tr>
<tr><th colspan="3">Analysis cross section</th><th colspan="2">STA.0+240</th><th colspan="2">STA.0+320</th><th colspan="2">STA.0+360</th></tr>
<tr><td rowspan="3">Settlement analysis</td><td colspan="2">Residual settlement (cm)</td><td colspan="2">18.562</td><td colspan="2">10.786</td><td colspan="2">9.752</td></tr>
<tr><td colspan="2">Allowable value (30 cm)</td><td colspan="2">OK</td><td colspan="2">OK</td><td colspan="2">-</td></tr>
<tr><td colspan="2">Allowable value (10 cm)</td><td colspan="2">-</td><td colspan="2">-</td><td colspan="2">OK</td></tr>
<tr><td colspan="3">Transformation analysis</td><td colspan="2">OK</td><td colspan="2">OK</td><td colspan="2">OK</td></tr>
<tr><td colspan="3">Liquefaction research</td><td colspan="2">NG</td><td colspan="2">NG</td><td colspan="2">NG</td></tr>
<tr><td rowspan="4">Safety analysis</td><td rowspan="2">Normal</td><td>At the time of construction</td><td>1.450</td><td>OK</td><td>1.434</td><td>OK</td><td>1.202</td><td>OK</td></tr>
<tr><td>At the time of service</td><td>1.450</td><td>OK</td><td>1.434</td><td>OK</td><td>1.202</td><td>NG</td></tr>
<tr><td colspan="2">At the time of earthquake</td><td>1.226</td><td>OK</td><td>0.876</td><td>NG</td><td>0.895</td><td>NG</td></tr>
<tr><td colspan="2">At the time of liquefaction</td><td>1.564</td><td>OK</td><td>1.199</td><td>OK</td><td>1.047</td><td>NG</td></tr>
<tr><td colspan="3">Lateral movement</td><td colspan="2">-</td><td colspan="2">-</td><td>2.00</td><td>NG</td></tr>
<tr><td colspan="3">Retaining wall bearing capacity</td><td colspan="2">-</td><td colspan="2">NG</td><td colspan="2">NG</td></tr>
</table>

Source: JICA Study Team

- Block 1

Residual settlement based on the settlement analysis is smaller than the allowable value of 30 cm. Based on the transformation analysis, the possibility of affecting the existing structures is considered to be low. The liquefaction research shows that it is a ground which can liquefy. However, since the value exceeds the allowable safety factor based on the stability analysis, the possibility of embankment destruction due to liquefaction is low.

- Block 2

Residual settlement based on the settlement analysis is smaller than the allowable value of 30 cm. Based on the transformation analysis, the possibility of affecting the existing structures is considered to be low. The liquefaction research result shows that it is a ground which can liquefy. However, since the value exceeds the allowable safety factor based on the stability analysis, the possibility of embankment destruction due to liquefaction is low. However, the result was lower than the allowable value at the time of earthquake. In addition, since the bearing capacity of the retaining wall cannot be secured, measures to ensure the bearing capacity are necessary.

- Block 3

Residual settlement based on the settlement analysis is smaller than the allowable value of 10 cm at the back of the abutment. Based on the transformation analysis, the possibility of affecting the existing structures is considered to be low. The liquefaction research result shows that it is a ground which can liquefy. Based on the stability analysis, the result was lower than the permissible value at all times (at the time of service), at the time of earthquake, and at the time of liquefaction. In addition, since the bearing capacity of the retaining wall cannot be secured, measures to ensure the bearing capacity are necessary. Based on the result of the research on the lateral movement of the abutment, countermeasures for lateral movement are necessary since the I value is larger than 1.20. In addition, since the bearing capacity of the retaining wall cannot be secured, measures to ensure the bearing capacity are necessary.

(2) Thaketa Side

The analysis results are summarized in Table 3.3.37.

Table 3.3.37 Summary of Analysis Results (Thaketa Side)

Block			Block 4		Block 5		Block 6	
Analysis cross section			STA.2+400		STA.2+620		STA.2+680	
Settlement analysis	Residual settlement (cm)		27.632		2.915		3.436	
	Allowable value (30 cm)		OK		OK		OK	
	Allowable value (10 cm)		NG		OK		-	
Transformation analysis			OK		OK		OK	
Liquefaction research			NG		NG		NG	
Safety analysis	Normal	At the time of construction	1.226	OK	1.282	OK	1.880	OK
		At the time of service	1.226	NG	1.282	OK	1.880	OK
	At the time of earthquake		0.895	NG	1.027	NG	1.466	OK
	At the time of liquefaction		1.379	OK	1.347	OK	1.772	OK
Lateral movement			0.762	OK	0.391	OK	-	
Retaining wall bearing capacity			NG		NG		-	

Source: JICA Study Team

- Block 4

The settlement analysis revealed a residual settlement of 27.632 cm. Since the allowable value of residual settlement is 10 cm within the influence area of the back of abutment, countermeasures against settlement are necessary. However, the residual settlement value is smaller than the allowable value of residual settlement of 30 cm when outside the range of the abutment area. Based on the transformation analysis, the possibility of affecting the existing structures is considered to be low. The liquefaction research result shows that it is a ground which can liquefy. However, since the value exceeds the allowable safety factor based on the stability analysis, the possibility of embankment destruction due to liquefaction is low. However, stability analysis at normal time and at the time of earthquake resulted a value lower than the safety factor. The countermeasures against lateral movement of the A2 abutment is considered to be unnecessary because the I value is smaller than 1.20. Since the bearing capacity of the retaining wall cannot be secured, measures to ensure the supporting force are necessary.

- Block 5

The settlement analysis revealed a residual settlement of 2.915 cm. It is less than both the residual settlement allowable values of 10 cm and 30 cm. Based on the transformation analysis, the possibility of affecting the existing structures is considered to be low. The liquefaction research result shows that it is a ground which can liquefy. However, since the value exceeds the allowable safety factor based on the stability analysis, the possibility of embankment destruction due to liquefaction is low. Also, as the safety factor is lower than the designed safety factor in the safety analysis at normal time and at the time of an earthquake, countermeasures are necessary. The lateral movement of the AF1 abutment is thought to be unnecessary because the I value is smaller than 1.20. Since the bearing capacity of the retaining wall cannot be secured, measures to ensure the supporting force are necessary.

- Block 6

The settlement analysis revealed a residual settlement of 3.436 cm. It is less than both the residual settlement allowable values of 10 cm and 30 cm. Based on the transformation analysis, the possibility of affecting the existing structures is considered to be low. The liquefaction research result shows that it is a ground which can liquefy. However, since the value exceeds the allowable safety factor based on the stability analysis, the possibility of embankment destruction due to liquefaction is low. In addition, the stability factor is always higher than the designed safety factor in the safety analysis at normal time and at the time of earthquake.

(3) On-ramp

The analysis results are summarized in Table 3.3.38.

Table 3.3.38 Summary of Analysis Results (On-ramp)

<table>
<tr><th colspan="3">Block</th><th colspan="2">Block 7</th><th colspan="2">Block 8</th></tr>
<tr><th colspan="3">Analysis cross section</th><th colspan="2">STA.0+360</th><th colspan="2">STA.0+400</th></tr>
<tr><td rowspan="3">Settlement analysis</td><td colspan="2">Residual settlement (cm)</td><td colspan="2">8.146</td><td colspan="2">5.620</td></tr>
<tr><td colspan="2">Allowable value (30 cm)</td><td colspan="2">OK</td><td colspan="2">OK</td></tr>
<tr><td colspan="2">Allowable value (10 cm)</td><td colspan="2">-</td><td colspan="2">OK</td></tr>
<tr><td colspan="3">Transformation analysis</td><td colspan="2">NG</td><td colspan="2">NG</td></tr>
<tr><td colspan="3">Liquefaction research</td><td colspan="2">NG</td><td colspan="2">NG</td></tr>
<tr><td rowspan="4">Safety analysis</td><td rowspan="2">Normal</td><td>At the time of construction</td><td>1.635</td><td>OK</td><td>1.161</td><td>OK</td></tr>
<tr><td>At the time of service</td><td>1.635</td><td>OK</td><td>1.161</td><td>NG</td></tr>
<tr><td colspan="2">At the time of earthquake</td><td>1.207</td><td>OK</td><td>1.072</td><td>NG</td></tr>
<tr><td colspan="2">At the time of liquefaction</td><td>2.413</td><td>OK</td><td>0.819</td><td>NG</td></tr>
<tr><td colspan="3">Lateral movement</td><td colspan="2"></td><td>3.167</td><td>NG</td></tr>
<tr><td colspan="3">Retaining wall bearing capacity</td><td colspan="2"></td><td colspan="2">NG</td></tr>
</table>

Source: JICA Study Team

- Block 7

The settlement analysis revealed a residual settlement of 8.146 cm. The residual settlement is less than the residual settlement allowable value of 30 cm. Based on the transformation analysis, countermeasures are necessary because the existing bridge piers are close to the area where both of the total settlement and the inclination angle are assumed to be greater than the allowable value. The liquefaction research result shows that it is a ground which can liquefy. However, since the value exceeds the allowable safety factor based on the stability analysis, the possibility of embankment destruction due to liquefaction is low. In addition, the stability factor is always higher than the designed safety factor in the safety analysis at normal time and at the time of earthquake.

- Block 8

The settlement analysis revealed a residual settlement of 5.620 cm. The residual settlement allowable value is 10 cm within the influence area of the back of abutment. Based on the transformation analysis, the possibility of affecting the existing structures is considered to be low. The liquefaction research result shows that it is a ground which can liquefy. However, since the value exceeds the allowable safety factor based on the stability analysis, the possibility of embankment destruction due to liquefaction is low. The stability analysis at normal time (at the time of service), at the time of earthquake, and at the time of liquefaction resulted in a lower safety rate than the planned safety rate. As a result of the research on lateral movement of the abutment, countermeasures for lateral movement are necessary since the I value is larger than 1.20. In addition, since the bearing capacity of the retaining wall cannot be secured, measures to ensure the bearing capacity are necessary.

3.3.3 Comparison of Countermeasures

The countermeasure principles and effect of each countermeasure, based on the Guideline for Road Earthwork of Soft Ground Treatment, are summarized as shown in Table 3.3.39.

Table 3.3.39 Countermeasure Principle and Effect of Each Countermeasure

Principle	Representative countermeasure method		Effect															
			Subsidence		Collapse			Transformation		Liquefaction								Secure traffability
			Subsidence of embankment after the service	Reduction of total subsidence	Increase in strength by consolidation	Increase of sliding resistance	Reduction of sliding power	Stress interruption	Reduction of stress	Measures to prevent occurrence of liquefaction							Measure to mitigate the damage of facilities	
										Improvement of properties of sand soil				Increase of effective stress	Dissolution of excess porewater pressure	Suppression of shear deformation		
										Density increase	Consolidation	Improvement of granularit	Decrease in saturation					
Consolidation･Draining	Surface drainage method																	○
	Sand mat method		○															○
	Slow loading method				○													
	Method of loading banking load		○		○													
	Vertical drain method	Sand drain method	○		○													
		Prefabricated vertical drain method	○		○													
	Vaccum consolidaion method		○		○													
	Groundwater-level decreasing method		○		○								○	○				
Compaction	Vibration compaction method	Sand compaction pile method	○	○	○	○			○	○								
		Vibration stick method		○						○								
		Vibroflotation method		○						○								
		Vibro tamper method		○						○								
		Weight dropping compaction method		○						○								
	Static compaction method	Non-vibratory sand compaction pile method	○	○	○	○			○	○								
		Static press-in compaction construction method								○								○
Induration	Surface mixing processing method			○*		○		○			○							
	Deep mixing method	Deep mixing method (Machanical strring method)		○*		○		○	○		○					○	○	
		High pressure injection stirring construction method		○*		○		○	○		○					○	○	
	Lime pile construction method			○*		○				○	○							
	Chemical grouting method			○*		○					○							
	Freezing method					○												
Replacement by excavation	Replacement method by excavation			○		○		○				○						
Pore water pressure dissipation	Pore water pressure dissipation method														○			
Load alleviation	Lightweight banking	Styrofoam block construction method		○			○		○									
		Bubble mixed lightweight banking method		○			○		○									
		Fifing beads mixed lightweight banking method		○			○		○									
	Calvert method			○			○		○									
Embankment reinforcement	Embankment reinforcement method					○											○	
Measures withi the structure	Loading berm method					○										○	○	
	Cast-in-side diaphragm wall method																	
	Sheet pile method					○		○							○**		○	
	Pile method			○		○			○								○	
Laying reinforcement	Laying reinforcement method					○												○

*)Valid for sand soil

**)With drainage function

Source: Road Earthwork - Guideline for Road Earthwork of Soft Ground Treatment (Japan Road Association) p.191

3.3.3.1 Countermeasures

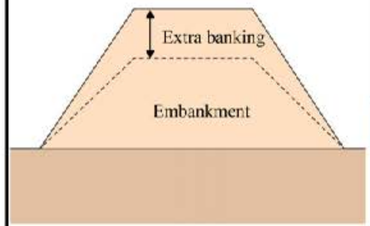
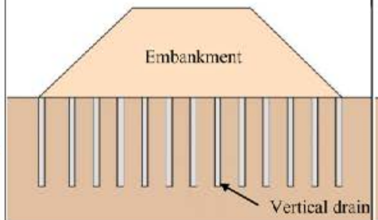
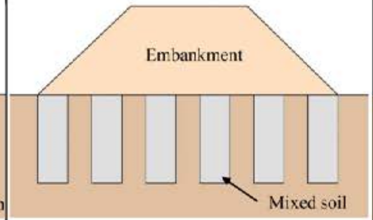
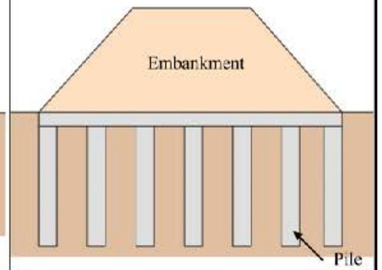
The countermeasures will be selected considering the purpose and necessity of countermeasures based on Table 3.3.39.

The Guideline for Road Earthwork of Soft Ground Treatment suggests to preferentially examine the method of loading banking load and the slow loading method, which are using the characteristics of the ground such as increase in strength due to consolidation. When the stability of the earthwork structure cannot be secured in the construction methods, application of consolidation, draining method, compaction method, and induration method will be considered. (Guideline for Road Earthwork of Soft Ground Treatment, Japan Road Association, p.180). Therefore, ①Method of loading banking load, ②PVD method which is consolidated drainage method, ③ Deep mixing method which is induration method, and ④Pile method will be compared.

Table 3.3.40 shows the comparison table for the abovementioned methods.

However, for the countermeasure against the bearing capacity of the mechanically-stabilized earth wall, the deep mixing method and the pile method are compared in Section 3.4 (Road Structure Design) and as a result, the deep mixing method is considered to be the optimum plan. Therefore, the deep mixing method is selected as the countermeasure against the bearing capacity of mechanically-stabilized earth wall.

Table 3.3.40 Soft Soil Countermeasures

		Option 1: Method of loading banking load	Option 2: PVD method	Option 3: Deep mixing method	Option 4: Pile method
Conceptual diagram					
Summary		Embankment is loaded higher than the design height, and consolidation is applied to facilitate settlement sufficiently to raise the strength of the ground. If a structure is built on the embankment, the extra banking method is also applied so that a load heavier than the structure is placed to cause consolidation settlement in advance.	Artificial products made of plastics or natural fiber are installed in cohesive soil to serve as drainage pillars. Where the method is applicable depends on the thickness of cohesive soil and characteristics of consolidation, and thus careful examination is required. It is also important to appropriately set the location of PVD on the ground to be improved, the depth of installation and other conditions.	Cement-based solidification materials are poured into the ground, and forcibly mixed with soft soil by mixing blades, to create stably treated soil to deep layers of the target ground. The method has a relatively small impact on the neighboring environment in terms of noise and vibration, and thus is applicable to a location near structures and residential buildings.	The load of embankment, etc. is transmitted to the foundation and deeper layers to improve the stability of structures and curb settlement. The method is used to curb unequal settlement and transformation of neighboring grounds that could occur as a result of embankment.
Advantages		Economically efficient	Economically efficient	Shorter construction period Assuredly effective	Shorter construction period Assuredly effective
Disadvantages		Longer construction period Observation during and after construction is needed to manage settlement/stability.	Longer construction period Observation during and after construction is needed to manage settlement/stability.	Economically inefficient	Economically inefficient
Effects	Settlement	○	○	○	○
	Stability	○	○	○	○
	Liquefaction	×	×	○	○
	Deformation	×	×	○	○
	Lateral movement	×	×	○	○
	Retaining wall bearing capacity	×	×	○	○

Legend : ○ Good × No Good

Source: JICA Study Team

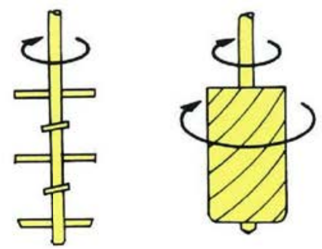
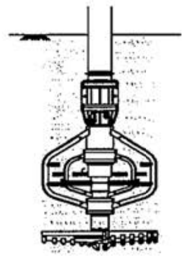
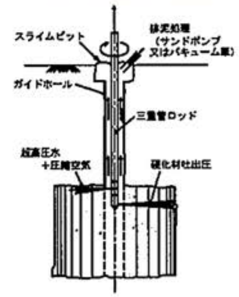
3.3.3.2 Construction Type for Countermeasure

Since there are various construction types for the deep mixing method, which is one of the countermeasures, the type of construction will also be selected.

Table 3.3.41 shows the result of the selection of the construction type.

From Table 3.3.41, the construction type for the deep mixing method in this Project is set as "1st plan: Teno-Column Construction Method".

Table 3.3.41 Selection of Construction Method

	Option 1: Teno-Column Construction Method (Deep mixing)	Option 2: Epo-column construction method (Deep mixing)	Option 3: CJG construction method (High pressure injection)
Conceptual diagram			
Summary	Cement and water are mixed in a mixing plant, then pressure-fed to the tip of the stirring blade with a slurry pump, and a soft soil in the improvement range is agitated and mixed at the original position in the ground to create a homogeneous improvement body.	Utilizing the characteristics of low speed rotation and high torque, improvement of the large diameter is created by the basket type stirring blade. By the compound relative stirring method, an improvement body excellent in improved quality is created.	After penetrating the triple tube rod to the depth of improvement, it cuts with high pressure water with compressed air, discharges the improvement material slurry and rotates it up, discharges excess slime, and creates a columnar improvement body.
Standard specification: Improved diameter	φ1.0 ~ 2.0m	φ1.8 ~ 2.5m	φ1.2 ~ 2.0m
Standard specification: Design intensity	Viscous soil / sandy soil: qu=200 ~ 1,000kN/m^2	Viscous soil / sandy soil: qu=100 ~ 1,000kN/m^2	Viscous soil / sandy soil: qu=1,000 ~ 3,000kN/m^2
Standard specification: Applicable ground	Viscous soil Sandy soil Organic soil	Viscous soil: N≦12 Sandy soil: N≦35 Gravel soil: N≦35	Viscous soil: N≦9 Sandy soil: N≦200
Standard specification: Construction machine	Three point supported type piling machine	Three point supported type piling machine	CJG machine
Standard specification: Depth of construction	Maximum 45m	Maximum 40m	About 30 ~ 40m
Advantages & Disadvantages	･Use modifying material in slurry. ･ It is possible to select an appropriate specification (imprvement shape, improvement strength, construction machine) according to improvement purpose. ･ Required strength can be obtained in a short time, so the construction term can be shortened. ･Because of mechanical stirring, it is impossible to work closely with the structure.	･Use modifying material in slurry.。 ･Improved diameter is sure. ･Excellent improved quality by composite relative stirring and mixing. ･Since it is a low-speed rotation, a high torque is exerted, and an improved body with a large diameter is created. ･Since an improved body with a large diameter is created, it is economical and can shorten the construction period.	･For construction by small aircraft, temporary temporary installation of the scaffolding is simple. ･Excellent applicability to hard ground compared to double pipe construction method. ･There is almost no influence on the surrounding ground to make sludge at the time of construction. ･Large quantities of waste mud treatment required for construction are necessary. ･Although the construction machine is compact, it requires a crane
Approximate direct construction cost: Approximate unit price	12.8 thousand yen/m^3	14.9 thousand yen/m^3	41.6 thousand yen/m^3
Approximate direct construction cost: Transportation cost	25,000 thousand yen/Unit (Including maritime transport)	50,000 thousand yen/Unit (Including maritime transport)	17,000 thousand yen/Unit (Including maritime transport)
Approximate direct construction cost: Assembly Dismantling expenses	2,200 thousand yen/Unit	2,000 thousand yen/Unit	1,500 thousand yen/Unit
Actual results in Myanmar	New Thaketa bridge	None	None
Evaluation	○	×	×

Legend: ○ Good × No Good

Source: JICA Study Team

3.3.4 Selection of Countermeasures

The necessary countermeasures in each cross section will be considered based on the analysis results.

3.3.4.1 Thanlyin Side

Table 3.3.42 shows the necessary measures and countermeasures in each cross section on the Thanlyin side.

Table 3.3.42 Countermeasure (Thanlyin Side)

Block	Block 1	Block 2	Block 3
Analysis section	STA.0+240.000	STA.0+320.000	STA.0+340.000
Settlement measure	Not needed	Not needed	Not needed
Transformation measure	Not needed	Not needed	Not needed
Stability measure	Not needed	Needed	Needed
Lateral movement measure	-	-	Needed
Retaining wall bearing capacity measure	-	Needed	Needed
Countermeasure	Method of loading banking load	Method of loading banking load + Deep mixing method	Deep mixing method

Source: JICA Study Team

3.3.4.2 Thaketa Side

Table 3.3.43 shows the necessary measures and countermeasures in each cross section on the Thaketa side.

Table 3.3.43 Countermeasures (Thaketa Side)

Block	Block 4	Block 5	Block 6
Analysis section	STA.2+440.000	STA.2+620.000	STA.2+680.000
Settlement measure	Needed	Not needed	Not needed
Transformation measure	Not needed	Not needed	Not needed
Stability measure	Needed	Needed	Not needed
Lateral movement measure	Not needed	Not needed	-
Retaining wall bearing capacity measure	Needed	Needed	-
Countermeasure	Method of loading banking load + Deep mixing method	Method of loading banking load + Deep mixing method	Slow loading method

Source: JICA Study Team

3.3.4.3 On-ramp

Table 3.3.44 shows the necessary measures and countermeasures in each cross section on the on-ramp.

Table 3.3.44 Countermeasures (On-ramp)

Block	Block 7	Block 8
Analysis section	STA.0+360.000	STA.0+400.000
Settlement measure	Not needed	Not needed
Transformation measure	Needed	Not needed
Stability measure	Not needed	Needed
Lateral movement measure	-	Needed
Retaining wall bearing capacity measure	-	Needed
Countermeasure	Slow loading method	Deep mixing method

Source: JICA Study Team

3.3.5 Ground Analysis after Countermeasure

3.3.5.1 Deep Mixing Method

A detailed research is conducted on the deep mixing method.

Deep mixing method is a ground improvement method by first supplying cement and other modifying materials underground as measures against settlement of embankment, circular slip of embankment, lateral movement of abutment, and bearing capacity of retaining wall, then moderately solidifying the ground by mixing and stirring forcibly with the original ground.

The design is based on the "Manual for Design and Construction of Deep Mixing Method on Land (2004)" (Public Works Research Institute).

(1) Design Principle

1) Improvement Strength and Improvement Rate

The improvement strength is set based on the results of the laboratory test of the New Thaketa Bridge designed and constructed in Yangon City of Myanmar.

Since the improvement strength at the New Thaketa Bridge is 300 to 700 kN/m^2, the maximum value of 700 kN/m^2 is considered as the improvement strength in this Project.

Improvement rate in each block is set based on the improvement strength. Improvement rate is 50% for countermeasure against arc slide on the slope of embankment and 87% for countermeasure against lateral movement at the back of abutment and retaining wall part. However, for countermeasure against retaining wall bearing capacity was examined in consideration of the ground reaction (q) of the retaining wall for each block. Also among the countermeasures for supporting the retaining walls, only 50% of the rubbing section immediately under the embankment was set.

The improvement rate results are shown in Table 3.3.45.

Table 3.3.45 Results of Improvement Rate

Area	Block No.	Improvement Rate (ap)		
		Slope Part	Retaining Wall Part	Back of Abutment
Thanlyin side	2	50%	87%,50%	-
	3	-	-	87%
Thaketa side	4	50%	87%,50%	-
	5	50%	87%,50%	-
On-ramp	8	-	-	87%

Source: JICA Study Team

2) Improvement Depth and Improvement Width

Improvement depth is set as below.

Slope part: At the depth through which the arc, which does not satisfy the designed safety factor in the calculation of circular slip with no measure, passes.

Retaining wall part: At the depth at which bearing capacity can be secured. However, when the support layer is shallower than the soft layer, the depth is set to the depth of the soft layer as countermeasure against settlement.

Back of abutment: In the lateral movement research, it shall extend to the bottom of the target layer.

Based on the "Manual for Design and Construction of Deep Mixing Method on Land (2004)" (Public Works Research Institute) p.77, the improvement width B is set to B/D = 0.5~1.0 or more with reference to the improvement length D.

Improvement width is set as follows:

Slope part: Ratio of the improvement length D and the improvement width B shall be D:B = 1:1.

Retaining wall part: As in "Manual for Design and Construction of Deep Mixing Method on Land (2004)" (Public Works Research Institute) p.177, a width of more than 1 m from the base width of the structure shall be secured. However, when B/D = 0.5 or less, the improvement width is adjusted until B/D = 0.5 or more.

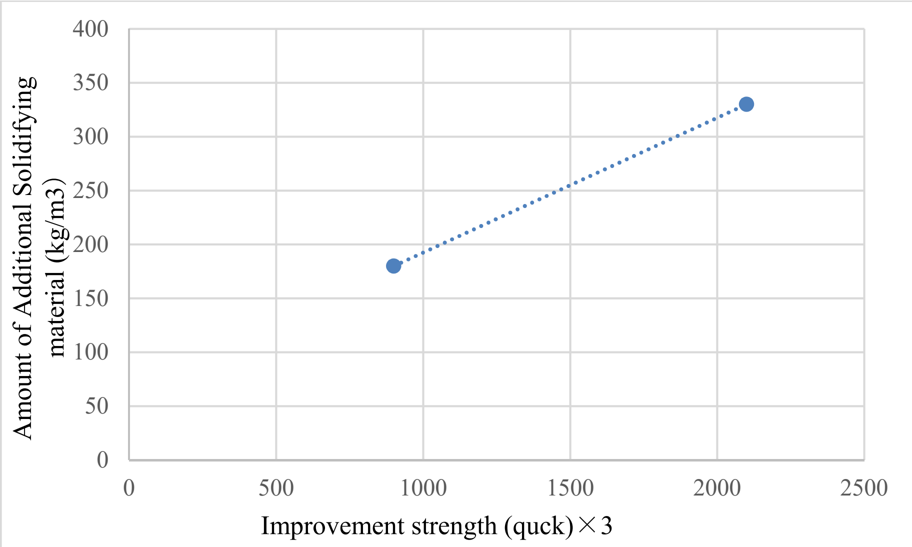
Back of abutment: Ratio of the improvement length D and the improvement width B shall be D:B = 1:0.5~1:1.

3) Amount of Additional Solidifying Material

The amount of additional solidifying material is set based on the result of the blending test at the time of the construction of the New Thaketa Bridge.

Figure 3.3.22 shows the relationship between the improvement strength at the New Thaketa Bridge and the amount of additional solidifying material.

As shown in Figure 3.3.22, the amount of additional solidifying material in the deep mixing method in this Project is set at 330 kg/m^3.



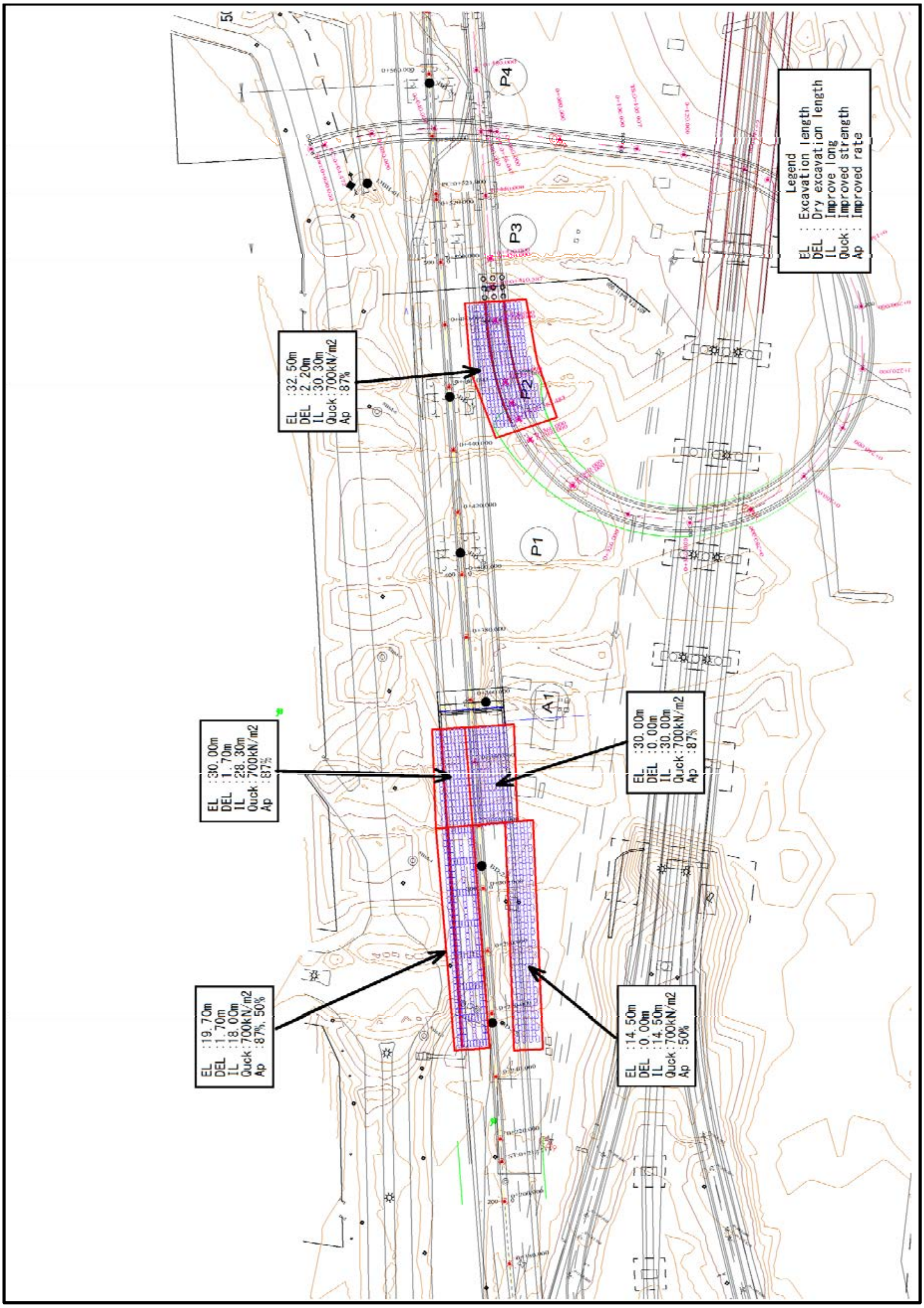
Source: JICA Study Team

Figure 3.3.22 Relationship between the Improvement Strength and the Amount of Additional Solidifying Material at the New Thaketa Bridge

(2) Result of the Ground Analysis after Design

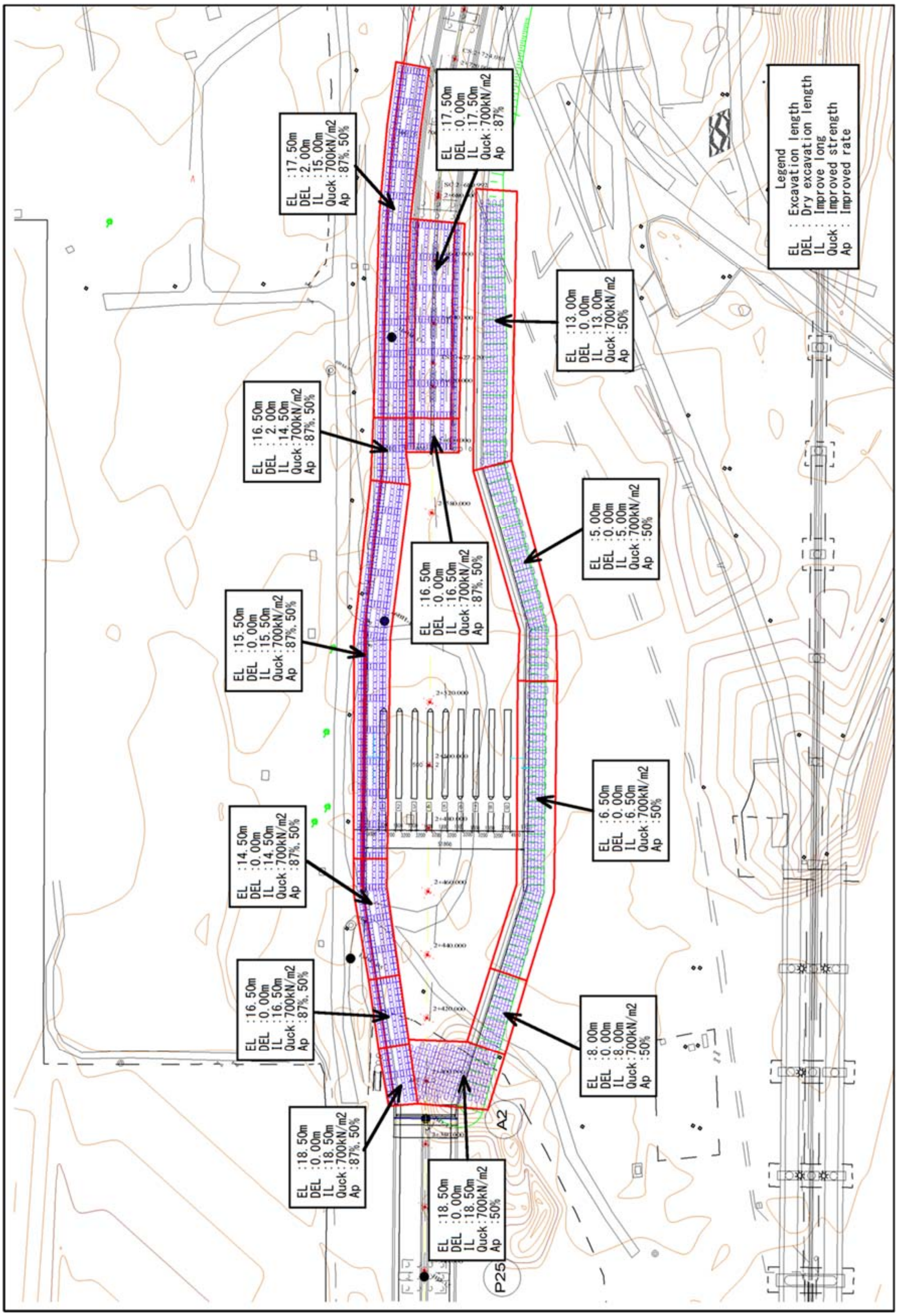
Based on the design principle above, the deep mixing processing method is carried out.

The column layout diagrams of the deep mixing method are shown in Figure 3.3.23 to Figure 3.3.24.



Source: JICA Study Team

Figure 3.3.23 Column Layout (Thanlyin Side)



Source: JICA Study Team

Figure 3.3.24 Column Layout (Thaketa Side)The analysis result of the ground after design is described below.

1) Block 2

The analysis results of Block 2 are shown in Table 3.3.46.

According to Table 3.3.46, the minimum safety factor is Fsmin = 2.433, and it can be confirmed that the planned safety factor Fs = 1.250 is satisfied.

Table 3.3.46 Result of Deep Mixing Method (Block 2)

Improvement Depth (m)	Improvement Width (m)	Minimum Safety Factor (Fsmin)
14.5	7.5	2.433

Source: JICA Study Team

2) Block 3

The analysis results of Block 3 are shown in Table 3.3.47.

According to Table 3.3.47, the minimum safety factor is Fsmin = 1.798, and it can be confirmed that the planned safety factor Fs = 1.250 is satisfied.

Table 3.3.47 Result of Deep Mixing Method (Block 3)

Improvement Depth (m)	Improvement Width (m)	Minimum Safety Factor (Fsmin)
30.0	22.0	1.798

Source: JICA Study Team

3) Block 4

The analysis results of Block 4 are shown in Table 3.3.48.

According to Table 3.3.48, the minimum safety factor is Fsmin = 1.825, and it can be confirmed that the planned safety factor Fs = 1.250 is satisfied.

Table 3.3.48 Result of Deep Mixing Method (Block 4)

Improvement Depth (m)	Improvement Width (m)	Minimum Safety Factor (Fsmin)
8.0	7.2	1.825

Source: JICA Study Team

4) Block 5

The analysis results of Block 5 are shown in Table 3.3.49.

According to Table 3.3.49, the minimum safety factor is Fsmin = 2.329, and it can be confirmed that the planned safety factor Fs = 1.250 is satisfied.

Table 3.3.49 Result of Deep Mixing Method (Block 5)

Improvement Depth (m)	Improvement Width (m)	Minimum Safety Factor (Fsmin)
13.0	7.4	2.329

Source: JICA Study Team

5) Block 8

The analysis results of Block 8 are shown in Table 3.3.50.

According to Table 3.3.50, the minimum safety factor is Fsmin = 1.312, and it can be confirmed that the planned safety factor Fs = 1.250 is satisfied.

Table 3.3.50 Result of Deep Mixing Method (Block 8)

Improvement Depth (m)	Improvement Width (m)	Minimum Safety Factor (Fsmin)
30.3	16.5	1.312

Source: JICA Study Team

3.3.5.2 Method of Loading Banking Load

A detailed research on the method of loading banking load is carried out.

The method of loading banking load is a construction method that removes extra banking after sufficiently advancing compaction by applying embankments at a height higher than the planned height and for the purpose of promoting consolidation of viscous lands and reducing the amount of residual settlement.

Table 3.3.51 shows the result of studying the construction embankment height.

From Table 3.3.51, the embankment will be 3.8~5.2 m in Thanlyin side and 4.7~5.8 m in Thaketa side from the construction base surface. However, for extra banking, it will be 1.6~2.8 m in Thanlyin side and 1.1~1.6 m in Thaketa side from the planned embankment level.

Table 3.3.51 Result of Construction Embankment Height Research

Side	Stationary point	Elevation of original ground H_{GL}(m)	Planned elevation of road H_{PL}(m)	Elevation of ground leveling (m)	Planned embankment height after ground Hp(m)	Computational planned embankment height H'(m)				Construction embankment height		Settlement S (m)	**Ratio of settlement amount S/H_E**
						H1'	H2'	H3'	Total H'	H'+S (m)	H_E' (m)		
Thanlyin Side	0+140	3.728	5.610	4.300	1.31	0.5	1.03	1.84	3.33	4.08	4.1	0.75	**0.184**
	0+160	3.148	5.681	4.300	1.38	0.5	1.03	1.84	3.40	4.17	4.2	0.77	**0.184**
	0+161.513	3.618	5.689	4.300	1.39	0.5	1.03	1.84	3.41	4.18	4.2	0.77	**0.184**
	0+180	3.800	5.968	4.300	1.67	1.0	0.85	1.32	3.13	3.85	3.9	0.71	**0.186**
	0+200	3.515	5.953	4.300	1.65	1.0	0.85	1.32	3.12	3.83	3.9	0.71	**0.186**
	0+212.713	4.195	6.076	4.300	1.78	1.1	0.85	1.32	3.24	3.97	4.0	0.74	**0.185**
	0+220	4.488	6.154	4.300	1.85	1.2	0.85	1.05	3.05	3.75	3.8	0.70	**0.186**
	0+240	4.260	6.399	4.300	2.10	1.4	0.85	1.05	3.30	4.05	4.1	0.75	**0.185**
	0+260	4.280	6.686	4.300	2.39	1.7	0.85	1.05	3.59	4.37	4.4	0.78	**0.178**
	0+280	3.620	7.017	4.300	2.72	2.0	0.85	0.79	3.65	4.44	4.5	0.79	**0.177**
	0+300	3.560	7.392	4.300	3.09	2.4	0.85	0.61	3.85	4.66	4.7	0.81	**0.173**
	0+320	3.330	7.809	4.300	3.51	2.8	0.85	0.61	4.27	5.12	5.2	0.85	**0.166**
Thaketa side	2+420	4.204	8.525	4.300	4.23	3.5	0.85	0.61	4.98	5.77	5.8	0.78	**0.136**
	2+440	4.304	8.254	4.300	3.95	3.3	0.85	0.61	4.71	5.48	5.5	0.77	**0.140**
	2+460	4.419	8.055	4.300	3.76	3.1	0.85	0.61	4.51	5.27	5.3	0.76	**0.144**
	2+480	4.037	7.930	4.300	3.63	3.3	0.39	0.61	4.33	5.08	5.1	0.75	**0.148**
	2+500	4.139	7.879	4.300	3.58	3.3	0.39	0.61	4.28	5.03	5.1	0.75	**0.149**
	2+520	3.820	7.900	4.300	3.60	3.3	0.39	0.61	4.30	5.05	5.1	0.75	**0.148**
	2+540	4.346	7.995	4.300	3.70	3.4	0.39	0.61	4.39	5.15	5.2	0.75	**0.147**
	2+560	3.911	8.164	4.300	3.86	3.2	0.85	0.61	4.62	5.39	5.4	0.77	**0.142**
	2+580	3.907	8.405	4.300	4.11	3.4	0.85	0.61	4.86	5.64	5.7	0.78	**0.138**
	2+600	3.307	8.510	4.300	4.21	3.5	0.85	0.61	4.97	5.75	5.8	0.78	**0.136**
	2+620	3.970	8.670	4.300	4.37	3.7	0.85	0.61	5.13	5.49	5.5	0.36	**0.066**
	2+627.42	4.000	8.632	4.300	4.33	3.6	0.85	0.61	5.09	5.45	5.5	0.36	**0.067**
	2+640	3.800	8.450	4.300	4.15	3.5	0.85	0.61	4.91	5.26	5.3	0.36	**0.068**
	2+660	3.330	7.856	4.300	3.56	2.9	0.85	0.61	4.31	4.65	4.7	0.33	**0.071**

H1' : Embankment height constituted by road / roabed
H2' : Embankment converted height of paving
H3' : Embankment converted height of traffic equivalent load

Source: JICA Study Team

3.3.5.3 Slow Loading Method

The slow loading method is investigated.

If embankment is rapidly applied on soft soil, slip failure and excessive transformation will occur in the embankment and foundation ground. The slow loading method is a construction method that aims to stabilize the ground by increasing the strength of the ground by embanking as slowly as possible over time as consolidation progresses instead of taking measure to the soft soil.

In this Project, the slow loading method will be applied in order to reduce the influence of embankment construction on Block 6, which is close to the pier of the flyover, and on Block 7, which is in the neighborhood of the pier of existing bridges.

In this Project, the embankment speed is set to 5 cm/day as basic, but for Block 6 and Block 7, the embankment speed is set to 3 cm/day by the slow loading method. However, based on information-oriented construction, confirming the embankment stability and controlling the embankment speed are important. Moreover, if it can be confirmed that the ground is more stable than predicted by observation during construction, the embankment speed can be increased.

3.3.5.4 Shallow Improvement

Shallow improvement is studied.

Shallow improvement is carried out by supplying improvement material such as cement to the ground all the way to secure the trafficability of the machine of the deep mixing method and forcibly solidifying by mixing and stirring with the base ground. For this improvement construction method, the improvement depth is set to 1.3 m.

The design policy is as follows:

- Since the improvement rate is total improvement, it is assumed that AP=100%.
- The depth of improvement was set at 1.3 m, which ensures the trafficability of the construction machine. (However, at the time of construction, re-examination is necessary with the equipment that is actually used.)
- The improvement range was set as the range of improvement and width (7 m) that the construction machine can be constructed and moved. (However, at the time of construction, re-examination is necessary with the equipment that is actually used.)
- Improvement strength was set to 420 kN/m^2 with reference to the laboratory soil test of the New Thaketa Bridge. However, it is necessary to conduct a laboratory soil test at the time of construction and to study the test result.
- The amount of cement added was 230 kg/m^3 based on the laboratory soil test of the New Thaketa Bridge. However, it is necessary to conduct a laboratory soil test at the time of construction and to study the test result.

3.3.6 Embankment Construction Plan

3.3.6.1 Embankment Construction Management

Construction management methods for embankment include "elevation management" and "thickness management". In general, when the settling time is fast and the amount of settlement is small, "elevation management" is used, and when the settling time is slow and the settlement amount is large, "thickness management" is used. Since the settling time is set to 390 days or 480 days in this Project, the construction management method for the embankment is set for each block taking into consideration the settlement amount (the ratio of the settlement amount to the embankment height).

Elevation management: Block in which the ratio of settlement amount to embankment height is 20% or less

Thickness management: Block in which the ratio of settlement amount to embankment height is 20% or more

Table 3.3.52 Construction Management Method for Embankment

Area	Block	Ratio of Settlement (%)	Management Method
Thanlyin side	Block 1	18.30	Elevation management
	Block 2	16.35	Elevation management
Thaketa side	Block 4	13.45	Elevation management
	Block 5	6.55	Elevation management
	Block 6	7.12	Elevation management
On-ramp	Block 7	26.67	Thickness management

Source: JICA Study Team

3.3.6.2 Precautions for Construction

(1) Additive Solidifying Material and Additional Amount of Ground Improvement Method

It is necessary to confirm the economical combination of additives and additional amount by preliminary compounding test. Especially for the amount to be added, it is necessary to carry out several blending tests to obtain an optimum additional amount.

(2) Review of Embankment Plan by Dynamic Observation and Dynamic Observation Analysis

Dynamic observation is not only for the construction management of embankment but also to carry out the dynamic observation analysis based on the result of dynamic observation in order to review the embankment plan as follows:

- Comparison between theoretical calculations and measured values
- Change of embankment amount (embankment thickness) based on review of settlement amount
- Check of residual settlement amount
- Review of construction process
- Confirmation of the influence on nearby structures (existing pier)

3.3.6.3 Dynamic Observation Plan

(1) Purpose

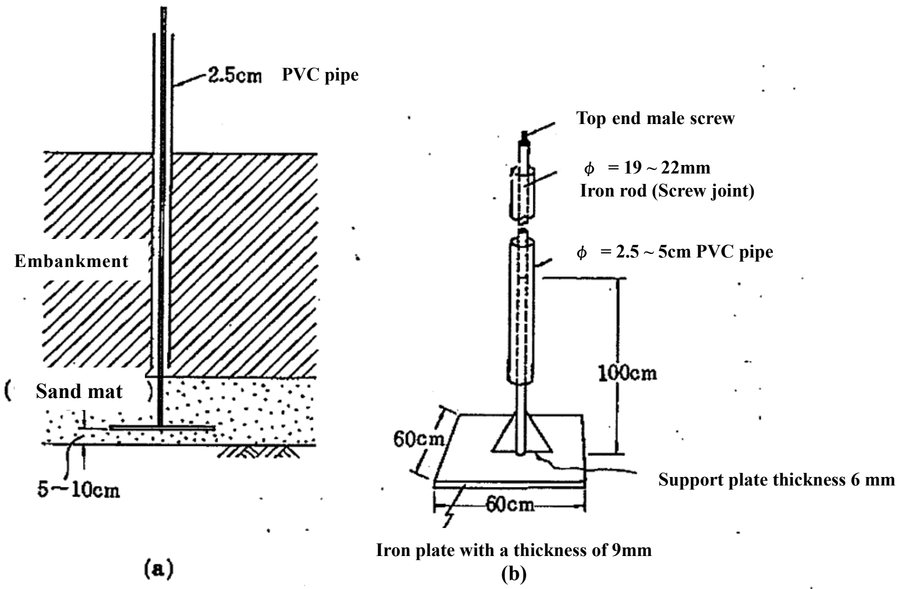
Generally, the settlement amount and settling time obtained from the theoretical calculation do not necessarily agree with the actual measured values. Therefore, in order to avoid problems during the construction stage, dynamic observation is carried out during embankment construction. Data can then be obtained for dynamic observation analysis. Specific purpose of dynamic observation and dynamic observation analysis includes the following:

- Comparison between theoretical calculations and measured values
- Change of embankment amount (embankment thickness) based on review of settlement amount
- Check of residual settlement amount
- Review of construction process
- Confirmation of the influence on nearby structures (existing pier)

(2) Meters Used for Dynamic Observation

1) Ground Surface Type Settlement Gage

The ground surface type settlement gage shown in Figure 3.3.25 will be set up to measure the settlement of the embankment part. It is made by welding a rod to the ground surface type settlement plate, and placing the settlement plate on the bottom of the embankment. A fixed point is set in the vicinity and the upper end of the rod is leveled with reference to that point to obtain the level of the settlement plate.

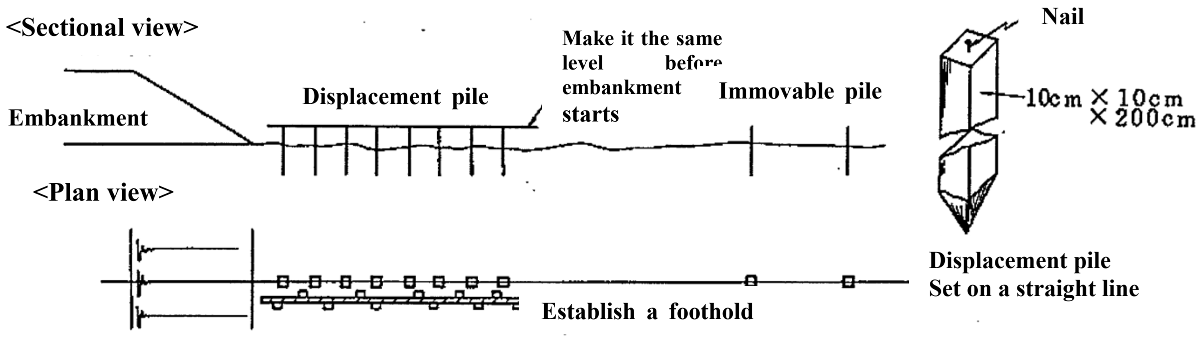


Source: JICA Study Team

Figure 3.3.25 Example of Installation of Ground Surface Type Settlement Gage

2) Displacement Pile

The displacement pile shown in Figure 3.3.26 is placed to measure the surface displacement of the nearby structure. The displacement pile is embedded in the ground with a length of 100 cm – 200 cm, 10 cm – 15 cm square, and the vertical and horizontal movement of the pile head is measured using the tape measure scale, level, and transit. The piles shall be aligned from the foot of the slope of the embankment, and if there is a nearby structure, it should be placed toward the target nearby structure.



Source: JICA Study Team

Figure 3.3.26 Example of Installation of Displacement Pile

3) Insertion Type Inclinometer

An insertion type inclinometer is installed to measure displacement of a proximate structure. The insertion type inclinometer measures the amount of horizontal displacement for each soil layer accompanying the advance of the embankment on the existing bridge piers. The insertion type inclinometer shall be installed at the same position as the distance from the foot of the slope of the embankment to the existing bridge piers.

(3) Observation Frequency

Table 3.3.53 shows the standards of observation frequency considering the embankment construction stage. However, the observation frequency shall be reviewed according to the situation of settlement. Observation shall be carried out until it is confirmed that the allowable residual settlement has been satisfied and the influence on the proximate structure is not a problem.

Table 3.3.53 Observation Frequency Standards

Stage of Embankment Construction	Observation Frequency
During the embankment construction	Once/day
Up to one month after start of embankment construction	Once/2 ~ 3 days
After 1 month from the completion of embankment	Once/week

Source: JICA Study Team

(4) Dynamic Observation Installation Plan

Dynamic observation meters will be installed as discussed below.

However, Block 3 and Block 8 have 78.5% improvement rate on the settlement target layer, and settlement and displacement of the surroundings are considered to hardly occur. Therefore, they are excluded from the targets of installation of dynamic observation meters.

1) Ground Surface Type Settlement Gage

The ground surface type settlement gage shall be installed in three places, i.e., one at the road center of each measurement point (interval of 20 m) in the area where the embankment construction is carried out and one each at the top of both slopes.

However, in the area close to the existing bridge piers of Block 7 (On-ramp), observation is additionally conducted at the spot between the stationary points at intervals of 10 m.

Table 3.3.54 Observation Point of Ground Surface Type Settlement Gage

Area	Block	Observation Range	Installation Location of Settlement Plate	Remarks
Thanlyin side	Block 1	STA.0+140 ~ STA.0+240	Center: 1 Top of slope: 2 in each stationary point	Intervals of 20 m
	Block 2	STA.0+260 ~ STA.0+320	Center: 1 Top of slope: 2 in each stationary point	Intervals of 20 m
Thaketa side	Block 4	STA.2+400 ~ STA.2+580	Center: 1 Top of slope: 2 in each stationary point	Intervals of 20 m
	Block 5	STA.2+600 ~ STA.2+660	Center: 1 Top of slope: 2 in each stationary point	Intervals of 20 m
	Block 6	STA.2+680 ~ STA.2+800	Center: 1 Top of slope: 2 in each stationary point	Intervals of 20 m
On-ramp	Block 7	STA.0+0 ~ STA.0+100	Center: 1 Top of slope: 2 in each stationary point	Intervals of 20 m
		STA.0+120 ~ STA.0+170	Center: 1 Top of slope: 2 in each stationary point	Intervals of 10 m
		STA.0+180 ~ STA.0+260	Center: 1 Top of slope: 2 in each stationary point	Intervals of 20 m
		STA.0+270 ~ STA.0+320	Center: 1 Top of slope: 2 in each stationary point	Intervals of 10 m
		STA.0+340 ~ STA.0+360	Center: 1 Top of slope: 2 in each stationary point	Intervals of 20 m

Source: JICA Study Team

2) Displacement Pile

The displacement piles shall be observed, one section at a time, in the ground analysis section of each block.

However, for Block 4 (Thaketa side), one block section shall be added near the toll gate center (STA.2+500) where the block extension is long and with the widest embankment width. Thus, the total observation sections are two.

Table 3.3.55 Observation Point of Displacement Pile

Area	Block	Number of Observation Section	Observation Point
Thanlyin side	Block 1	1	STA.0+240
	Block 2	1	STA.0+320
Thaketa side	Block 4	2	STA.2+420
			STA.2+500
	Block 5	1	STA.2+620
	Block 6	1	STA.2+680
On-ramp	Block 7	8	STA.0+120
			STA.0+140
			STA.0+160
			STA.0+200
			STA.0+280
			STA.0+300
			STA.0+320
			STA.0+360

Source: JICA Study Team

3) Insertion Type Inclinometer

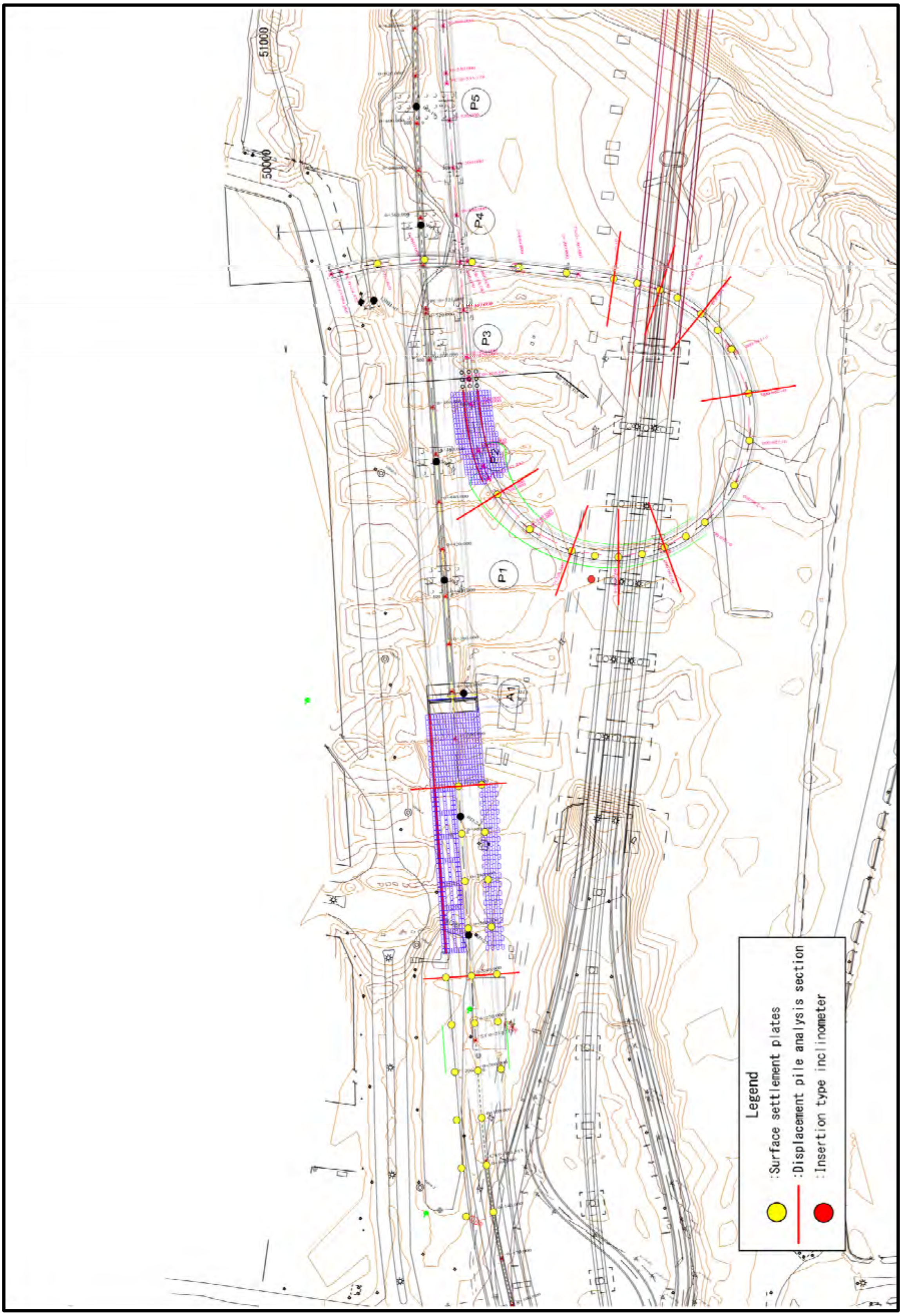
The insertion type inclinometer shall be observed in one cross section close to the existing piers of Block 7 (On-ramp).

Table 3.3.56 Observation Point of Insertion Type Inclinometer

Side	Block	Number of Observation Section	Observation Point
On-ramp	Block 7	1	STA.0+310

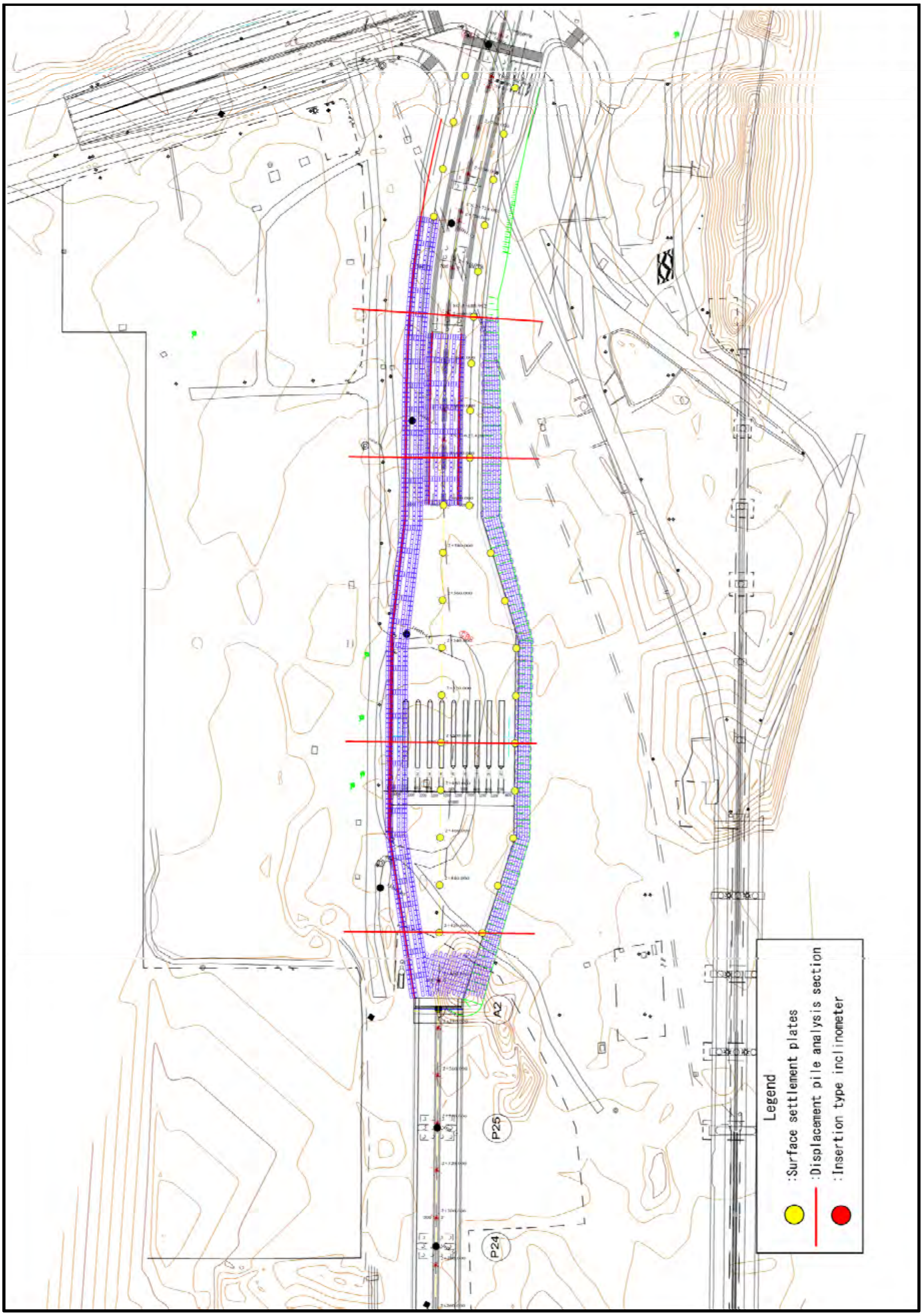
Source: JICA Study Team

The installation positions of dynamic observation instruments are shown in Figure 3.3.27 to Figure 3.3.28.



Source: JICA Study Team

Figure 3.3.27 Dynamic Observation Equipment Installation Site (Thanlyin Side)



Source: JICA Study Team

Figure 3.3.28 Dynamic Observation Equipment Installation Site (Thaketa Side)

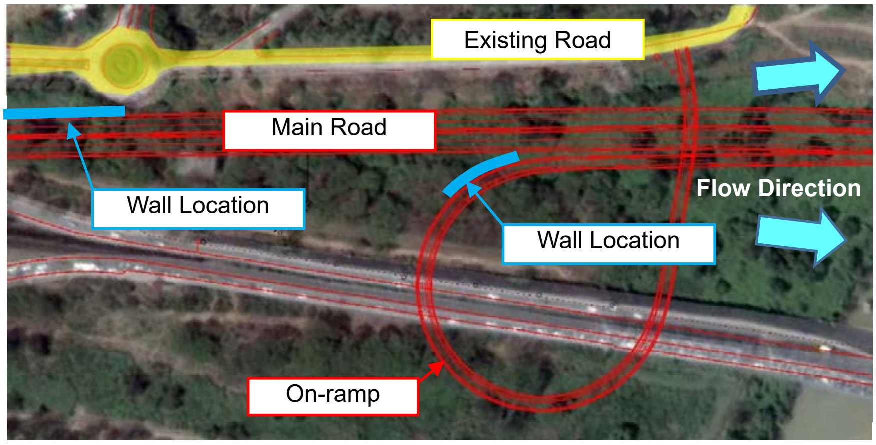
3.4 ROAD STRUCTURE DESIGN

3.4.1 Location of Road Structures

In the project site, there are some constraints. For example, it is necessary to maintain both functions of the existing roads and current drainage systems. Some structures are required to satisfy these constraints. In this chapter, some structures that do not affect the existing facilities will be studied.

The plan on the left bank side is shown in Figure 3.4.1 below. The project road is close and parallel to the current road. It is possible that the embankment slope will cover the current road. Therefore, retaining wall is necessary in this section. As the embankment slope of on-ramp will also cover the main road, retaining wall with length of about 30 m will be set up behind the ramp abutment. Rainwater is gathered and flows toward the river because of the low land in this area. It is necessary to secure the drainage system.

For this reason, the retaining walls are set up at the downstream side of the main road and behind the ramp abutment.

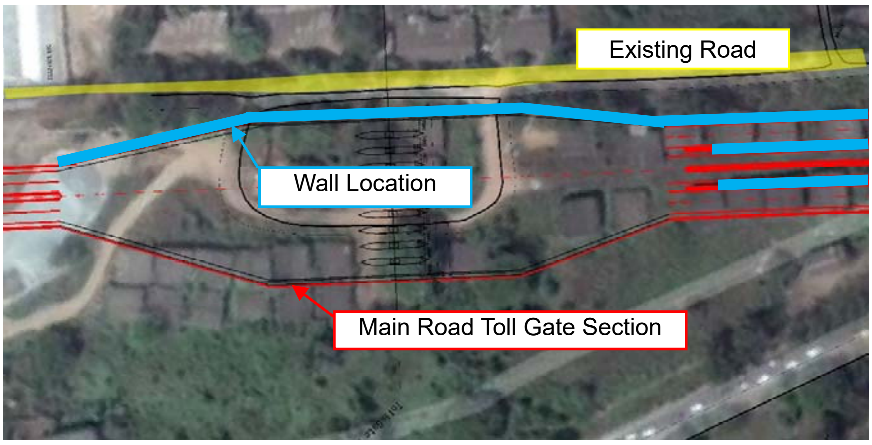


Source: JICA Study Team

Figure 3.4.1 Location of the Road Structures on the Left Bank

The plan on the right bank side is shown in Figure 3.4.2 below. The planned toll gate is close to the current road and border, and the opening is very narrow. Therefore, retaining wall structure is necessary on the project road.

For this reason, the retaining wall needs to be set up on the downstream side of the main road.



Source: JICA Study Team

Figure 3.4.2 Location of the Road Structures on the Right Bank

3.4.2 Design Conditions

Table 3.4.1 Design Conditions

THANLYIN MAIN DESIGN CONDITIONS

ITEM	DESIGN CRITERIA	(SUPPORTING DOCUMENTATION)	NOTE	ONSULTATION RESUI
1. Application criteria	1) Japan Road Association (2012). Road Earthwork: Retaining Wall Guidelines 2012. 2) Japan Road Association (2012). Specifications for Highway Bridges, the Commentary. 3) Civil Engineering Research Center (2014). Reinforced Soil (Terre Armee) Wall Design and Construction Manual, 4th Rev Ed. 4) American Association of State Highway and Transportation Officials (2012) . AASHTO LRFD Bridge Design Specifications, 6th Ed (US)	【Road Earthwork: Retaining Wall Guidelines P.66】		
2. Soil Condition 1). Embankment Material	γ=19kN/m², φ=30°, C=0kN/m²			
3. Earthquake resistant design	Horizontal Seismic Coefficient kh = Cz・kho = 1.0×0.18 = <u>0.18</u>			
4. Loading Conditions 1). Main Load Loading 2). Special Loads Collision Load	Road Department 11.6kN/m² (AASHTO LRFD 2012 Bridge) Mechanically -stabilised earth wall Handrail (A) 43kN/m (Only horizontal force taken into account)			
5. Study Model	Thanlyin Side 3750(Full Height) H=3730(Finished Height) + H4=1900 = 56 Thaketa Side H=4500(Full Height) H=4480(Finished Height) + H4=1700 = 6200 H=2250(Full Height) H=2230(Finished Height) + H4=2200 = 4450 Gravity Wall Hmax H=2100(Full Height) H=2061(Finished Height)	Concrete Barrier Height h=1150 Concrete Barrier Height h=1150 Concrete Barrier Height h=1400		

Table4-6 Unit weight of soil (kN/m³)

Ground	Soils and foundations	One with a loose	A dense
Natural Ground	Sand and gravel	18	20
	Sandy soils	17	19
	Cohesive soils	14	18
And the Backfill Soil embankment	Sand and gravel	20	
	Sandy soils	19	
	Cohesive soils (However wL < 50%)	18	

※ Groundwater less than weight per unit volume of soil is good as 9(kN/m³) minus the value in the table.

Table4-5 And the backfill soil strength parameters of embankment

And the Backfill Soil embankment	Shear Resistance Angle (φ)	Adhesive Force (C)[※2]
Gravelly soils	35°	–
Sandy soils[※1]	30°	–
Cohesive soils (However wL < 50%)	25°	–

※1 Little fines sand, gravelly soils of good use.

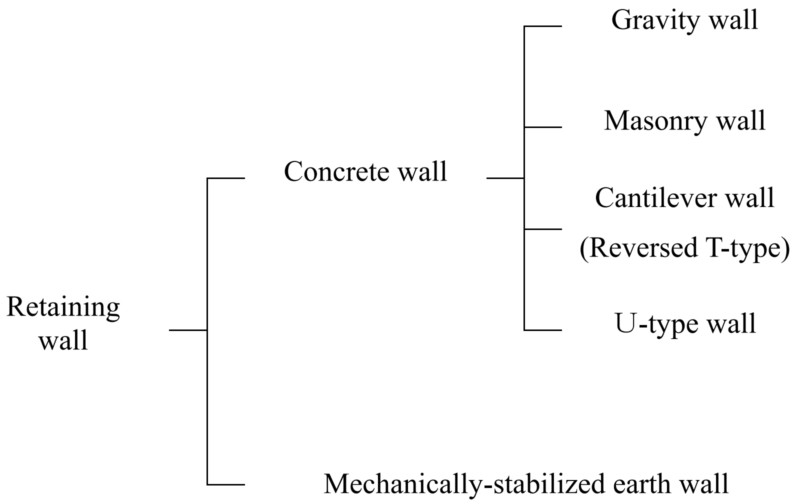
※2 To estimate the soil parameters from the table above to ignore sticky C.

Source: JICA Study Team

3.4.3 Design of Mechanically-Stabilized Earth Wall

3.4.3.1 General Form

In the study of the retaining wall structures, it is necessary to know the characteristics of various retaining wall structures and adopt the suitable wall for the purpose. The general types of retaining wall structures are shown in Figure 3.4.3 below.



Source: JICA Study Team

Figure 3.4.3 Types of Retaining Wall

The characteristics of each retaining wall are shown in Table 3.4.2 below. In this study, gravity wall, cantilever wall, and reinforced soil wall are the preferred retaining wall structures.

Table 3.4.2 Characteristics of Retaining Walls

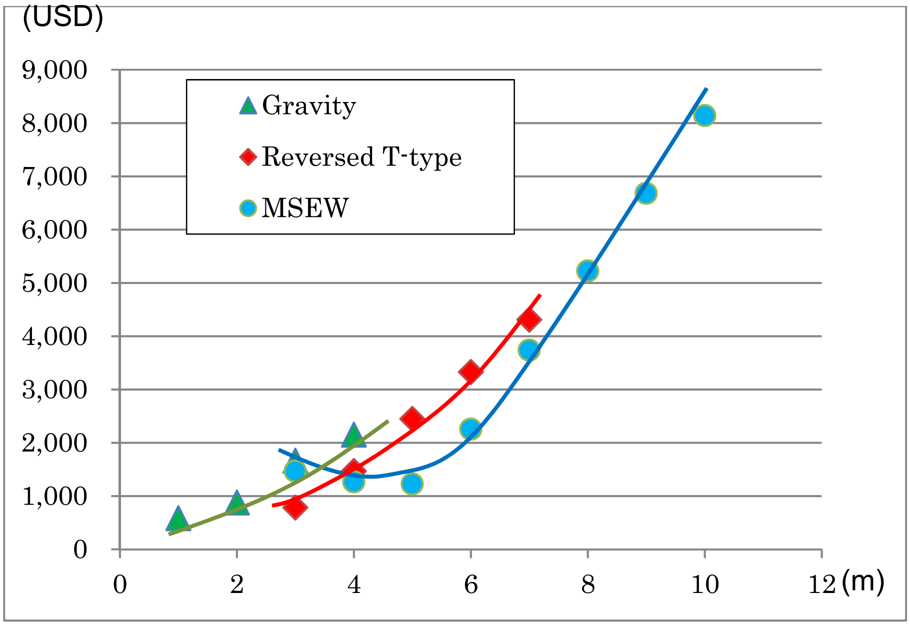
Wall Type	Features	Judgement
Gravity wall	Stabilized by its own weight. Suitable as a small retaining wall.	○
Masonry wall	Adopted to stabilize on the slope. Used as a slope protection.	
Cantilever wall (Reversed T-type)	Stabilized by own and soil weight. Applicable to 3 m to 10 m retaining wall.	○
U-type wall	In case of site constraints on both sides and adopted in excavated structures.	
Mechanically-stabilized earth wall	Reinforcing the soil with steel plates. Applicable to 3 m to 18 m high wall.	○

Source: JICA Study Team

Mechanically-stabilized earth wall is designed based on “Manual for Design and Construction for Mechanically-Stabilised Earth Wall Method forth (4) edition revision, August .4” from the construction result and the ease of material procurement.

3.4.3.2 Selection of Retaining Wall

For the selected retaining wall, the economics of each height are compared. As a result of the comparison, the mechanically-stabilized earth wall (hereinafter MSEW) or reinforced soil wall is the most economical among the walls. Moreover, the economical height is about 3 m or more. The height of the project road is 3 m to 6 m; therefore, the application of the reinforced soil wall is most preferable.



Source: JICA Study Team

Figure 3.4.4 Comparison of Retaining Walls

3.4.3.3 Foundation Method

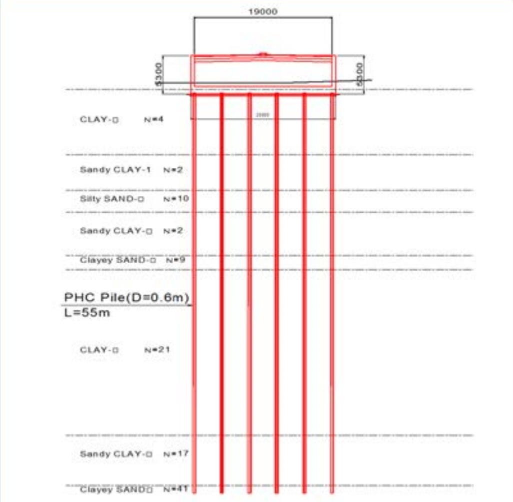
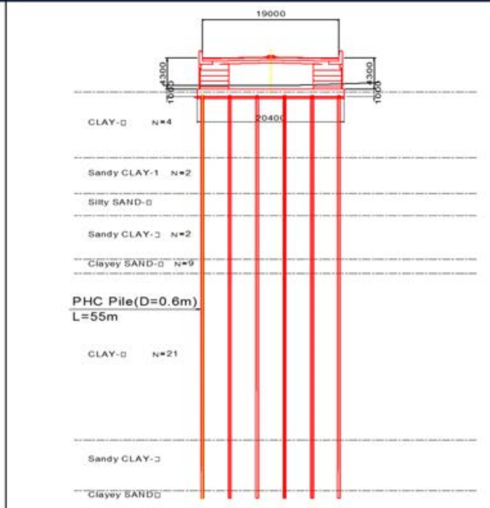
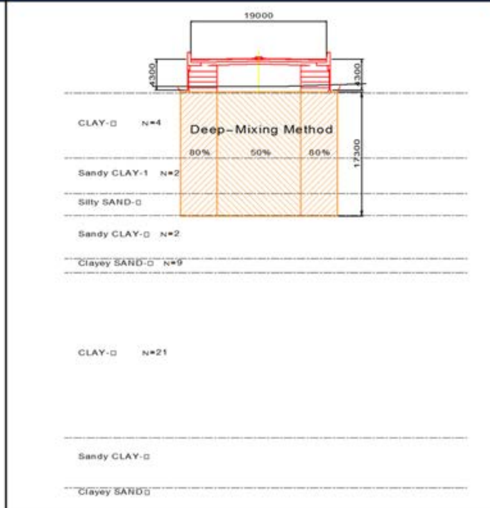
There are two types of foundation. The one is pile type and the other one is soft soil treatment. Each foundation has a different structure.

Pile type: The solid ground is the bearing layer. In this type, the piles are anchored on Clay Sand II. The left bank side pile length is about 55 m, and right bank side pile length is about 45 m.

Soft soil treatment: This type improves the weak layer. The weak layer is from the ground surface to the top of the sandy silt. The left bank side depth of treatment is about 20 m, while the right bank side depth is about 15 m.

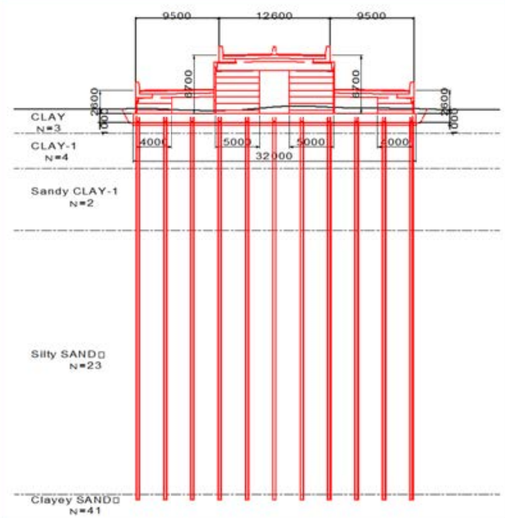
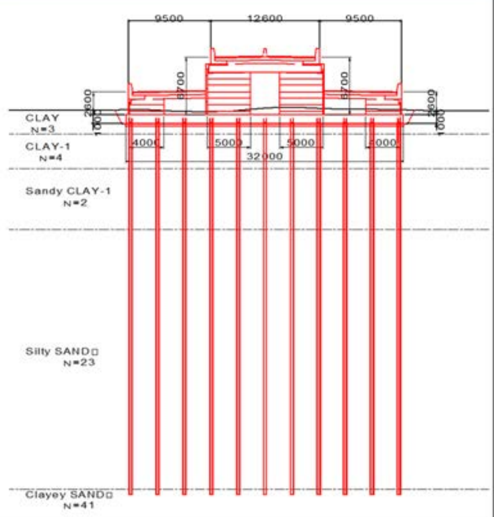
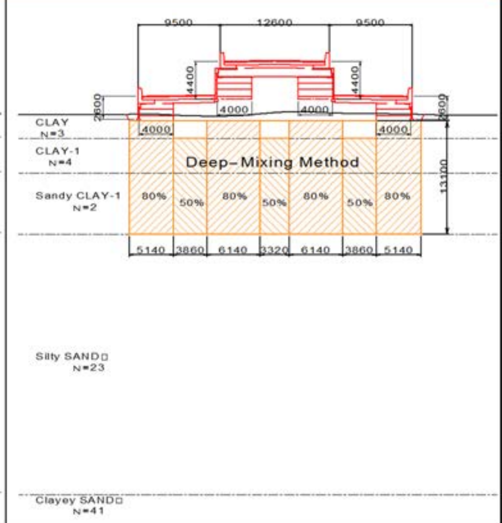
3.4.3.4 Selection of Road Structures

The selection of road structures considers the type of retaining wall and foundation method. The comparison is carried out including retaining wall with foundation, and three types are examined. The three types compared are shown in Figure 3.4.5 to Figure 3.4.7 below.

	Case 1 Retaing Concrete Wall with Piles		Case 2 Mechanically-Stabilided Earth Wall with Piles and Slab		Case 3 Mechanically-Stabilized Earth Wall with Deep-Mixing Method	
Schematic Section						
Construction Outline	Adapt U-Type Wall. Pile Foundation under the Wall Footing.		Adapt Reinforced Soil Wall. The foundation is PHC piles with slab.		Adapted Reinforced Soil Wall. The foundation is Deep-Mixing Method.	
Foundation Structal Feature	- The piles are PHC, diameter 500mm - The piles are anchored clay sand II, N value is more than 41.	○	- The piles are PHC, diameter 500mm - The piles are anchored clay sand II, N value is more than 41.	○	- Under the Reinforced Soil Wall has 80% rate of treatment. - The rate of other parts are 50%. - Deep-Mixing Method is anchored sandy silt, N value is more than 23.	○
Construction Period	6.9 days/m	○	5.9 days/m	◎	6.1 days/m	◎
Cost Ratio	1.44	○	1.03	○	1.00	◎
Evaluation					Most Recommended	

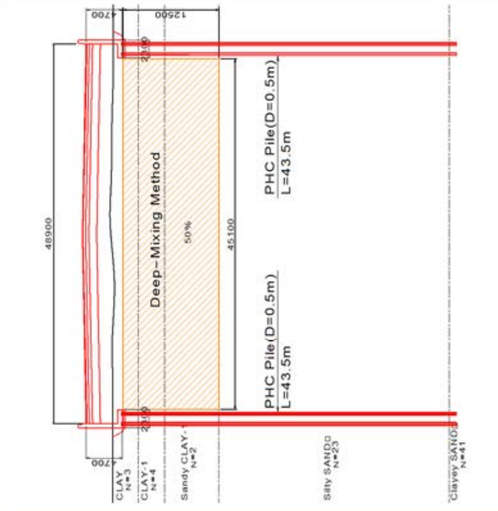
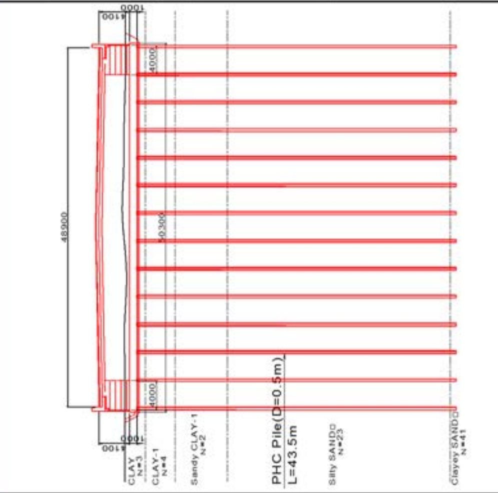
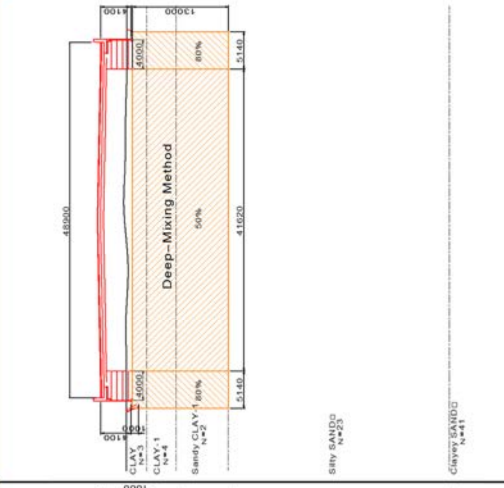
Source: JICA Study Team

Figure 3.4.5 Comparison of Retaining Wall with Foundation (Left Bank Side)

	Case 1　Retaing Concrete Wall with Piles		Case 2　Mechanically-Stabilided Earth Wall with Piles and Slab		Case 3 Mechanically-Stabilized Earth Wall with Deep-Mixing Method	
Schematic Section						
Construction Outline	Main road is adapted U-type Wall. The piles foundation under the walls. Other area inserts Deep-Mixing Method.		Main road and Ramp are adapted Reinforced soil wall. Foundation is PHC piles and slab.		Main road and Ramp are adapted Reinforced soil wall. Foundation is Deep-Mixing Method.	
Foundation Structal Feature	- Piles are PHC, diameter 500mm. - The piles are anchored clay sand II - N value is more than 41. - Deep-Mixing Method is anchored sandy silt.	○	- Piles are PHC, diameter 500mm. - The piles are anchored clay sand II - N value is more than 41.	○	- Under the Reinforced Soil Wall has 80% rate of treatment. - The rate of other parts are 50%. - Deep-Mixing Method is anchored sandy silt, N value is more than 23.	◎
Construction Period	10.6 days/m	○	10.6 days/m	○	9.8 days/m	◎
Cost Ratio	1.14	○	1.52	○	1.00	◎
Evaluation					Most Recommended	

Source: JICA Study Team

Figure 3.4.6　Comparison of Retaining Wall with Foundation (Right Bank Side)

	Case 1 Retaing Concrete Wall with Piles		Case 2 Mechanically-Stabilided Earth Wall with Piles and Slab		Case 3 Mechanically-Stabilized Earth Wall with Deep-Mixing Method	
Schematic Section						
Construction Outline	Adapt U-Type Wall. Pile Foundation under the Wall Footing.		Adapt Reinforced Soil Wall. The foundation is PHC piles with slab.		Adapted Reinforced Soil Wall. The foundation is Deep-Mixing Methodt.	
Foundation Structal Feature	- The piles are PHC, diameter 500mm - The piles are anchored clay sand II, N value is more than 41.	○	- The piles are PHC, diameter 500mm - The piles are anchored clay sand II, N value is more than 41.	○	- Under the Reinforced Soil Wall has 80% rate of treatment. - The rate of other parts is 50%. - Deep-Mixing Method is anchored sandy silt, N value is more than 23.	○
Construction Period	9.7 days/m	◎	12.2 days/m	○	10.1 days/m	◎
Cost Ratio	1.14	○	1.49	○	1.00	◎
Evaluation					Most Recommended	

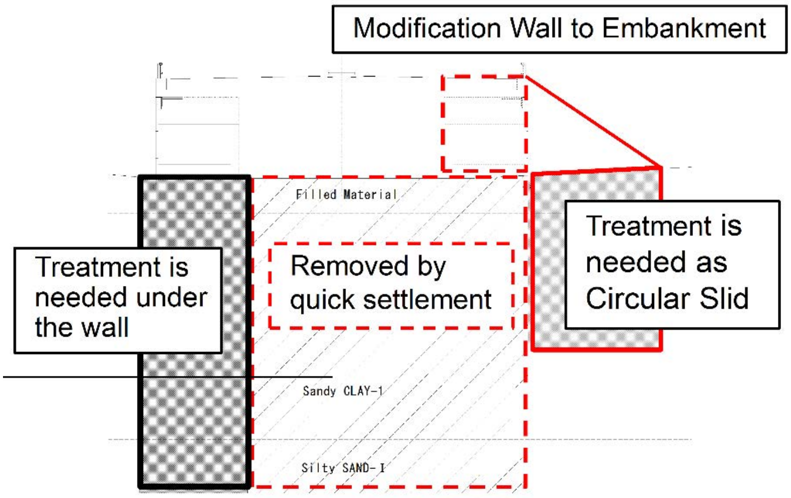
Source: JICA Study Team

Figure 3.4.7 Comparison of Retaining Wall with Foundation (Toll Gate Section)

As a result of the comparison, the reinforced soil wall with soft soil treatment is better than the other wall structures. In this Project, the road structure selects this wall and foundation structure.

3.4.3.5 Cost Estimation

The cost of the road structures will bear the high cost of the soft soil improvement. Therefore, the reduction of the area of soft soil improvement will be studied. The bottom of the wall needs soft soil improvement for stability, and soft soil improvement outside the wall is also needed for settlement measure. For this reason, it is possible to change the method from soft soil improvement to surcharge. Especially, the effect of cost reduction is expected in the toll gate section which has a wide width.

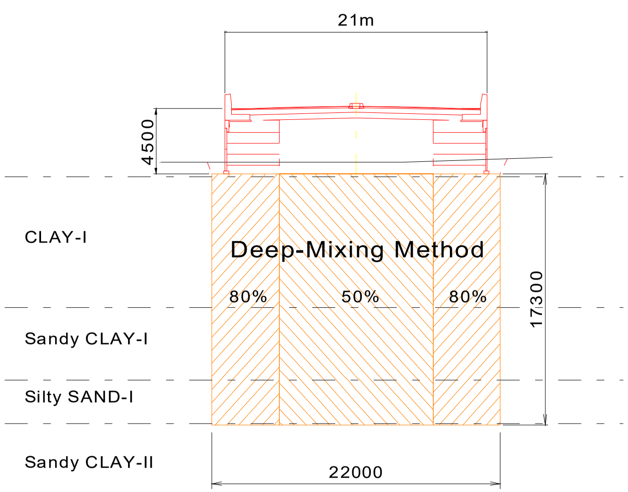
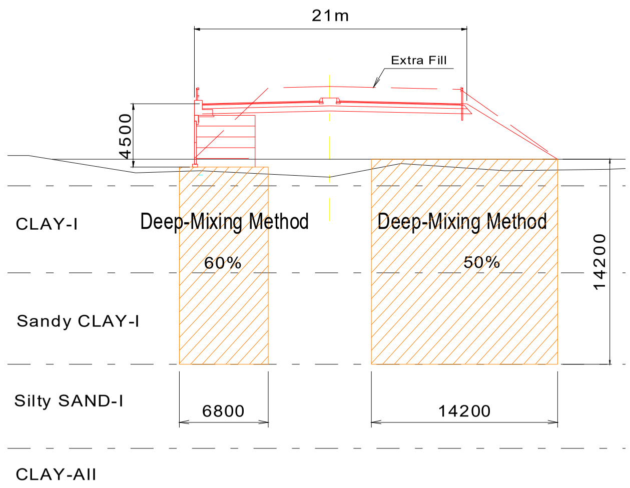


Source: JICA Study Team

Figure 3.4.8 Section Where to Apply the Surcharge Method

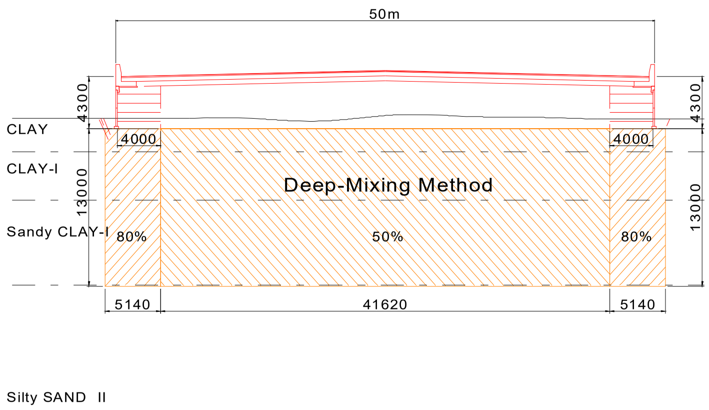
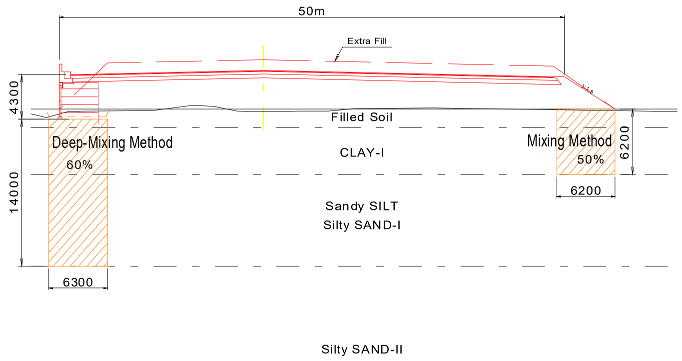
Surcharge method will be applied on both the left and right banks. The comparison of road structures with modified foundation structures is shown below.

Because cost reduction is expected, the surcharge method is recommended.

	Basic Design	Detail Design
Schematic Section		
Construction Outline	- Adapted Reinforced Soil Wall both side. - The Mixing Method is Full Width.	- Adapted Reinforced Soil Wall one side. - The Mixing Method is under the wall and Embankment Edge. - The Extra Fill for quick settlemet.
Foundation Structal Feature	- The rate of Soft Soil Treatment is 80% under the Wall. - The Embankment Wdge is 50% based on latest geological survey. ○	- Adapted Mixing Method at Embankmet Edge as Countermesure Circular Slip. - The rate of Soft Soil Treatment is 60% under the Wall. - The Embankment Edge is 50% based on latest geological survey. ○
Construction Period	6.1 days/m ○	5.1 days/m ◎
Cost Ratio	1.37 ○	1.00 ◎
Evaluation		Most Recommended

Source: JICA Study Team

Figure 3.4.9 Changing to Surcharge Method (Left Bank)

	Basic Design		Detail Design	
Schematic Section				
Construction Outline	- Adapted Reinforced Soil Wall both side. - The Mixing Method is Full Width.		- Adapted Reinforced Soil Wall one side. - The Mixing Method is under the wall and Embankment Edge. - The Extra Fill for quick settlemet.	
Foundation Structal Feature	- The rate of Soft Soil Treatment is 80% under the Wall. - The Embankment Wdge is 50% based on latest geological survey.	○	- Adapted Mixing Method at Embankmet Edge as Countermesure Circular Slip. - The rate of Soft Soil Treatment is 60% under the Wall. - The Embankment Edge is 50% based on latest geological survey.	○
Construction Period	10.1 days/m	○	4.4 days/m	◎
Cost Ratio	4.19	○	1.00	◎
Evaluation			Most Recommended	

Source: JICA Study Team

Figure 3.4.10 Changing to Surcharge Method (Right Bank)

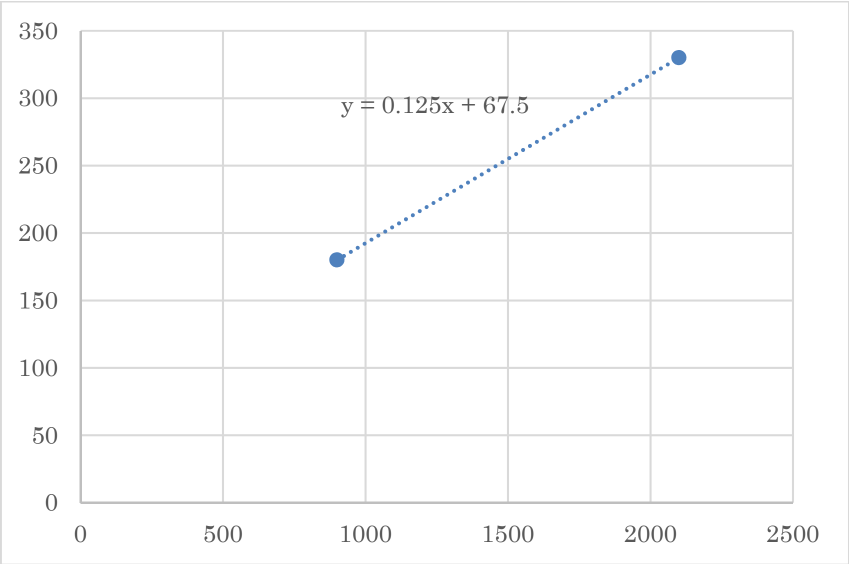
3.4.3.6 Gravity Wall

In the Thaketa section, the retaining wall near the intersection is gravity wall, The height is less than H=2.0 m. Shallow improvement is proposed to ensure the bearing capacity of the Clay-1 layer. The amount of cement added is based on the result in the New Thaketa Bridge. The amount of cement to be added is 110 kg/m^3, but during construction, it is necessary to secure more than 290 kN/m^2.

Table 3.4.3 Amount of Cement Added in the New Thaketa Bridge

Design Strength (quck)	300	700
Design Strength x 3 (safety rate)	900	2100
Amount of cement added(kg/m3)	180	330

Source: JICA Study Team



Source: JICA Study Team

Figure 3.4.11 Amount of Cement Added in the New Thaketa Bridge

Ground Reaction (Ordinary Condition) 96.156 kN/m^2

Safety Rate 3 x 96.156 = 288.468 kN/m^2

Amount of Cement Added = 0.125 x 288.468 + 67.5 = 103.559 kg/m^3 ---> 110 kg/m^3

3.5 FLYOVER AND WIDENING OF THANLYIN CHIN KAT ROAD

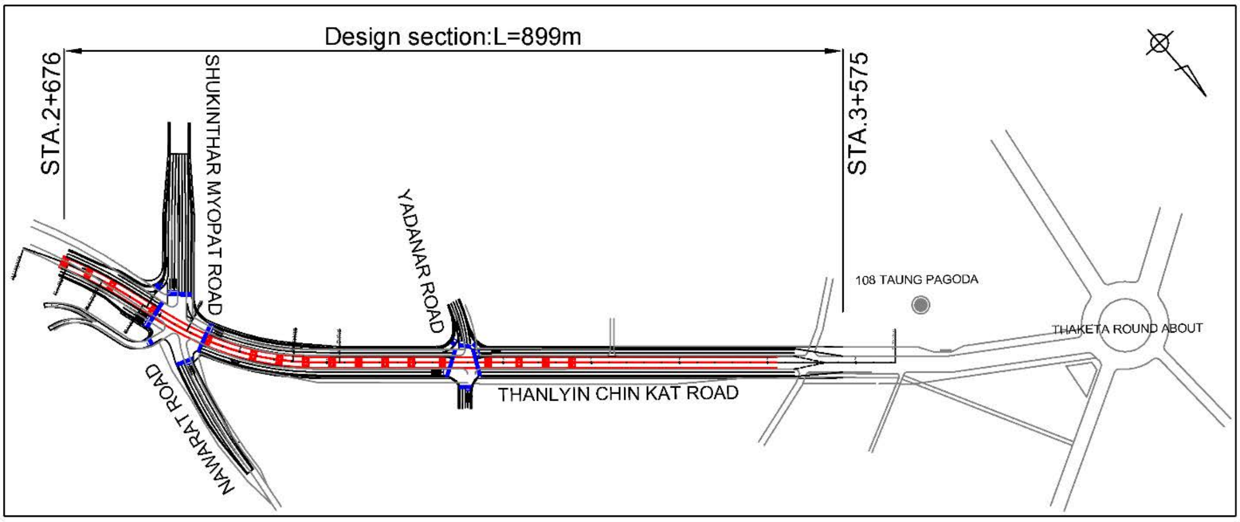
3.5.1 Design Conditions

(1) Project Site for Flyover and Widening of Thanlyin Chin Kat Road

The site for the Thanlyin Chin Kat Road flyover and widening project is as follows:

- Beginning point (STA.2+676): The beginning point is the A1 abutment of the flyover.
- End point (STA.3+575): The end point is the taper end merged to the existing Thanlyin Chin Kat Road after the flyover connects to the at-grade road.

The design section for the Thanlyin Chin Kat Road flyover and widening project is shown in Figure 3.5.1.



Source: JICA Study Team

Figure 3.5.1 Design Section for Flyover and Widening of Thanlyin Chin Kat Road

(2) Design Criteria for the Flyover Design

The geometric design criteria for the flyover design are shown in Table 3.5.1, which was prepared based on the ASEAN Highway Standards and Japanese Road Design Criteria.

Table 3.5.1 Geometric Design Criteria for the Flyover Design

Item	Criteria			Adopted Value	Remark
	Desirable	Standard	Absolute		
Design Speed		60 km/h		60 km/h	
Min. Horizontal Curve Radius (m)	200	150	120	320	
Min. Horizontal Curve Length (m)		700/ø	100	150.231	
Min. Transition Curve Length (m)		50		52.813	
Min. Radius without Transition Curve (m)		1000	500	-	
Min. Radius without Superelevation (m)		2000		-	Straight section: 2%
Max. Grade (%)		5.0	7.0	3.0	
Min. Vertical Curve Length (m)		50		60	
Min. K value (Crest)	20	14		40	

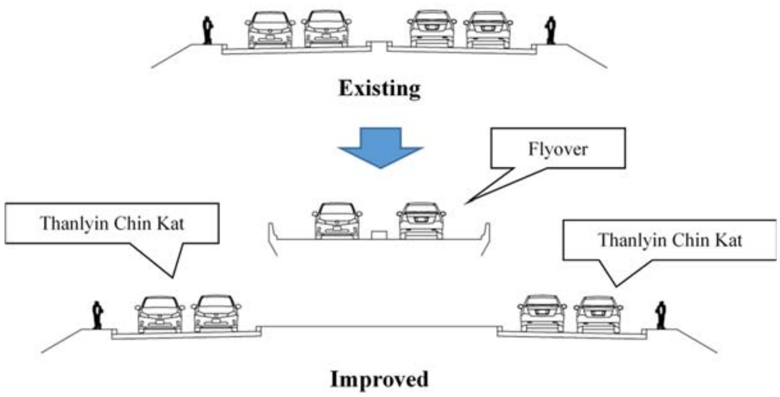
Item	Criteria			Adopted Value	Remark
	Desirable	Standard	Absolute		
Min. K value (Sag)	15	10		24	
Max. Superelevation (%)		6.0		6.0	
Superelevation to Horizontal Curve (%)		6.0			R=270-330 m
		5.0			R=330-420 m
		4.0			R=420-560 m
		3.0			R=560-800 m
		2.0			R=800-2000 m
Max. Ratio for Superelevation Development		1/125		1/165	
Stopping Sight Distance (m)		75		>75	

Source: JICA Study Team

(3) Design Criteria for the Thanlyin Chin Kat Road Widening Design

The flyover shall be constructed at the center of the existing Thanlyin Chin Kat Road in the Project. According to this flyover construction, Thanlyin Chin Kat Road shall be improved.

Thanlyin Chin Kat Road shall be widened on both sides of the flyover as shown in Figure 3.5.2.



Source: JICA Study Team

Figure 3.5.2 Widening of Thanlyin Chin Kat Road

The geometric design criteria such as minimum horizontal curve radius, maximum vertical grade, etc. are determined in accordance with the Road Design Criteria of Myanmar, Department of Highways, Ministry of Construction, 2015 (hereinafter called as "MOC Road Design Criteria").

(4) Typical Cross Section for the Thanlyin Chin Kat Road Widening

The typical cross section for the Thanlyin Chin Kat Road widening, i.e., frontage road of the flyover, was determined in accordance with the MOC Road Design Criteria.

1) Road Classification

"Local Road (Urban)" is applied, as the road runs within a township.

Table 3.5.2 Road Classification

Function	Local Road	Collector Road	Sub Arterial Road	Main Arterial Road	Expressway
Name	Township Road	District Road	Regional/State Road	Union (National)	Expressway

				Highway	
Feature	Run within a Township	Run within a District	Run within a Region/State	Across Region/State	High Speed with Access Control

Source: MOC Road Design Criteria

2) Design Speed

According to the MOC Road Design Criteria, the standard design speed for a local road in urban areas is 50 km/h. However, the MOC Road Design Criteria states that if necessary, design speeds can be 20 km/h lower than the standard speeds.

There is Ta Yah Shit Taung Pagoda along Thanlyin Chin Kat Road, and thus, many pedestrians walk along the road.

In order to ensure pedestrian safety, “design speed of 40 km/h” is applied for the Thanlyin Chin Kat Road widening (frontage road of the flyover).

Table 3.5.3 Design Speed (Minimum)

Class	Terrain	Local Road	Collector Road	Sub Arterial Road	Main Arterial Road	Expressway
Rural	Flat	60	70	80	100	120
	Rolling	50	60	70	80	100
	Mountainous	40	50	50	60	80
Urban		50	60	70	80	100

Note: If necessary, design speeds can be 20 km/h lower than the standard speeds.

Source: MOC Road Design Criteria

3) Number of Lanes

Four lanes will be provided, which is the same number of existing lanes at this time.

In the intersection, an exclusive left turn lane and channelized right turn lane will be provided, if necessary.

4) Lane

According to the MOC Road Design Criteria, the minimum lane width for “Local Road (Urban)” is 3.00 m. However, as many heavy vehicles from Bago Bridge will utilize Thanlyin Chin Kat Road and the existing lane width is 3.50 m, “3.50 m” will be applied as the lane width.

Table 3.5.4 Lane Width (Minimum)

Class	Local Road	Collector Road	Sub Arterial Road	Main Arterial Road	Expressway
Rural	3.00	3.25	3.50	3.50	3.60
Urban	3.00	3.00	3.25	3.50	3.50

Note: If it is a single lane, the width of the lane should be at least 3.7 m.

Source: MOC Road Design Criteria

5) Right Shoulder

"1.5 m" will be applied basically in accordance with the MOC Road Design Criteria. However, the right shoulder will be narrowed down to 0.5 m if required due to private land.

Right shoulder of 0.5 m will be applied in the road section between Yadanar Intersection and the end point. Because of this narrowing, land acquisition as well as demolition of a private wall can be avoided.

Table 3.5.5 Right Shoulder Width (Minimum)

Class	Terrain	Local Road	Collector Road	Sub Arterial Road	Main Arterial Road	Expressway
Rural	Flat	1.5	2.0	2.5	2.5	3.0
	Rolling	1.5	2.0	2.5	2.5	3.0
	Mountainous	1.0	1.5	2.0	2.0	2.5
Urban		1.5	1.5	2.0	2.0	2.0

Note: If a sidewalk is placed, the right shoulder can be reduced to 0.5 m as minimum

Source: MOC Road Design Criteria

6) Left Shoulder

"0.5 m" will be applied in accordance with the MOC Road Design Criteria.

Table 3.5.6 Left Shoulder Width (Minimum)

Class	Terrain	Local Road	Collector Road	Sub Arterial Road	Main Arterial Road	Expressway
Rural	Flat	0.5	0.5	0.75	0.75	1.0
	Rolling	0.5	0.5	0.75	0.75	1.0
	Mountainous	0.5	0.5	0.75	0.75	1.0
Urban		0.5	0.5	0.75	0.75	1.0

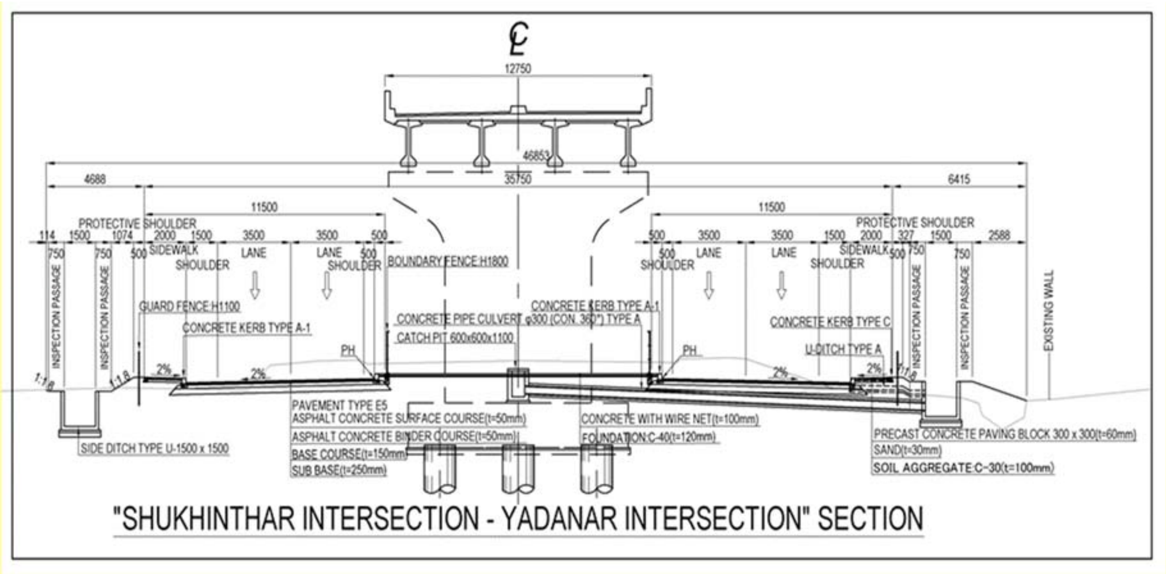
Source: MOC Road Design Criteria

7) Sidewalk

"2.0 m" will be applied as the sidewalk width in accordance with the Japanese Road Design Criteria. Pedestrians as well as wheelchairs can pass each other in the 2.0 m wide sidewalk.

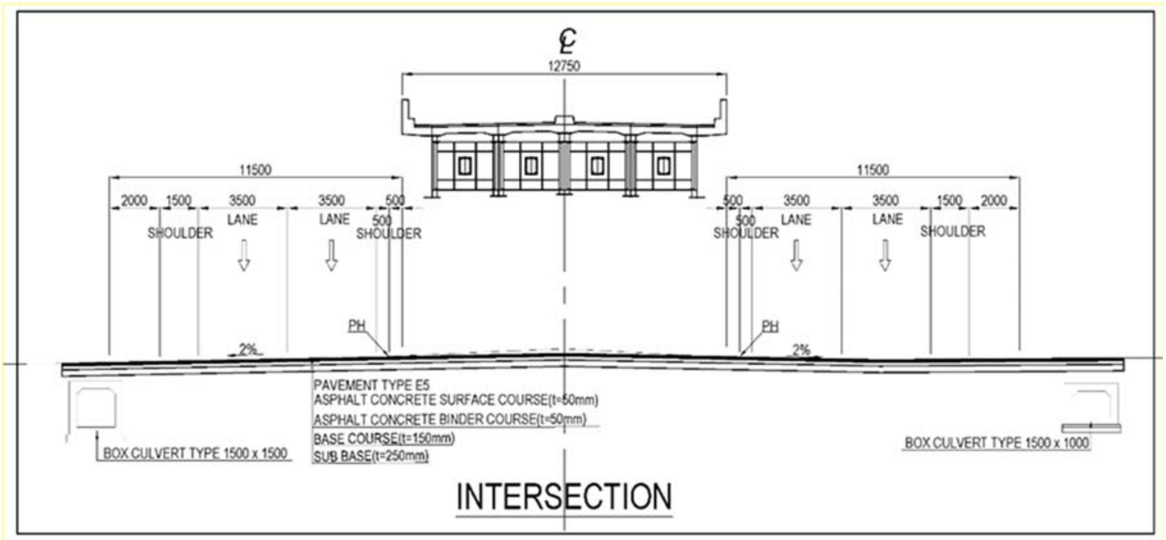
8) Typical Cross Section

Typical cross section for the Thanlyin Chin Kat Road widening is shown in Figure 3.5.3 to Figure 3.5.6. Typical cross section for the flyover is the same as that of the main route.



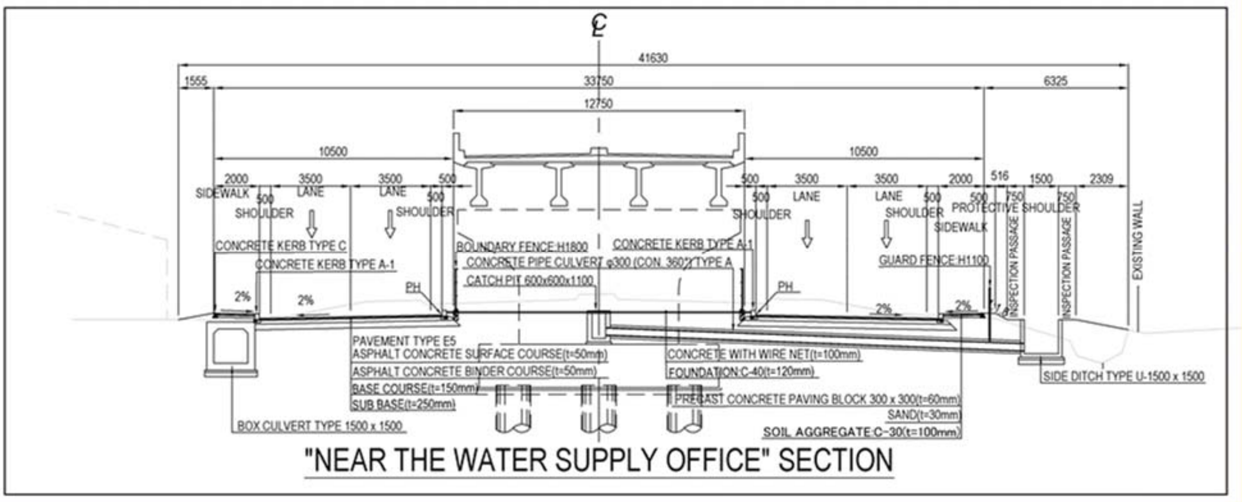
Source: JICA Study Team

Figure 3.5.3 Typical Cross Section: “Shukhinthar Intersection-Yadanar Intersection”



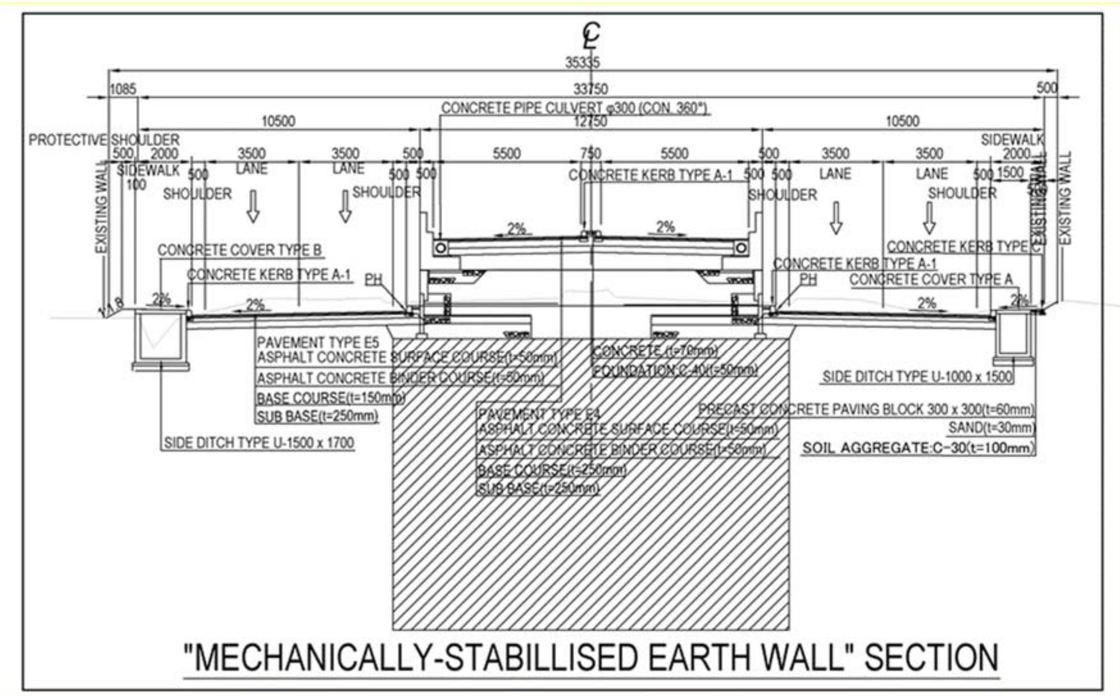
Source: JICA Study Team

Figure 3.5.4 Typical Cross Section: “In the Intersection”



Source: JICA Study Team

Figure 3.5.5 Typical Cross Section: "Near the Water Supply Office"



Source: JICA Study Team

Figure 3.5.6 Typical Cross Section: "Mechanically-stabilized Earth Wall"

9) Type of Median Strip

“Raised median strip” was selected as the median strip on the flyover from the following reasons;

- Physical separation is necessary to prevent the deviation from oncoming traffic lane in consideration of driving manner in Myanmar
- “Rigid barrier” is inferior to “Raid median strip” in emergency use of flyover (emergency cars cannot pass over the oncoming lane)

Table 3.5.7 Type of Median Strip on the Flyover

Item		Flat	Raised	Barrier
Schematic Picture				
Structural Feature		Continuous asphalt plane with rubber poles / delineators	Concrete curb with height of 250mm	Rigid concrete wall barrier or steel railing
Function	Separation of lane	Semi-separated by pole or line	Physically separated by curb	Physically separated by barrier
	Anti-Deviation	Low	Medium	High
	Emergency use*1	Possible	Possible	Impossible
Oppression to drivers		Low	Low	High
Suitable/Applicable Site Condition*2		-Min.curve radius : R≧300m -Design speed : V ≦ 60km -Vertical gradient : i < 4%	-Min.curve radius : R≧300m -Design speed : V ≦ 60km -Vertical gradient : i < 4%	-Min.curve radius : R<300m -Design speed : V≧80km/h -Vertical gradient : i ≧ 4%
Installation Cost		Low	Moderate	Very High*3

*1 Emergency cars can physically pass over the median
*2 Minimum curve radius : R=320m, Max. i = 3% shall be applied to flyover
*3 Construction cost would be very high since width of F/O have to be widen due to incursion to artificial limit on S-curve section

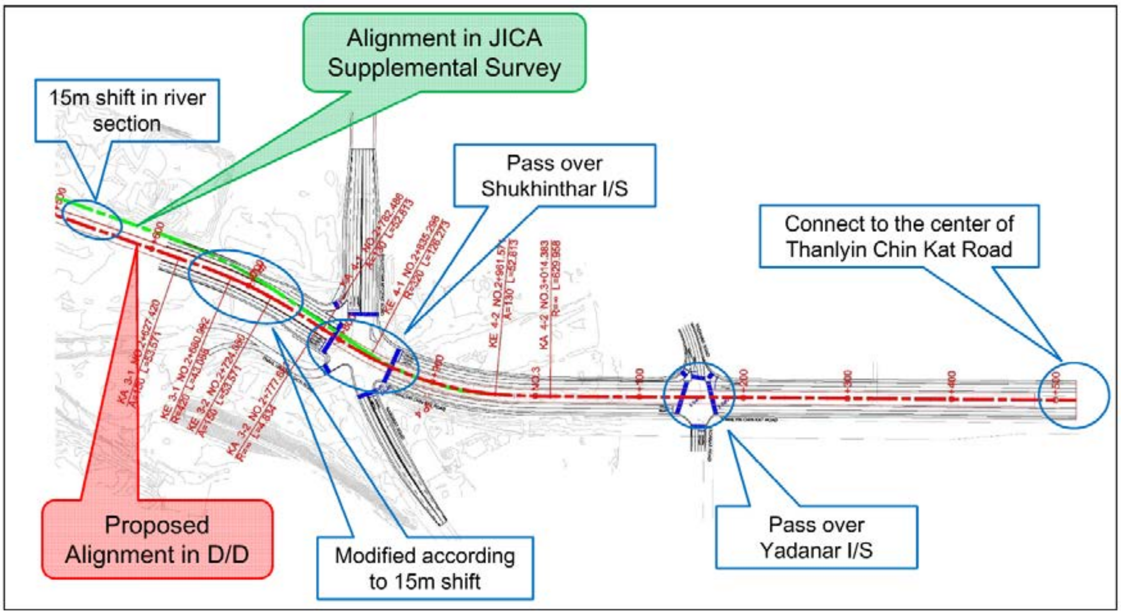
Source: JICA Study Team

3.5.2 Alignment of the Flyover

(1) Horizontal Alignment

Horizontal alignment of the flyover was determined taking into account the following conditions:

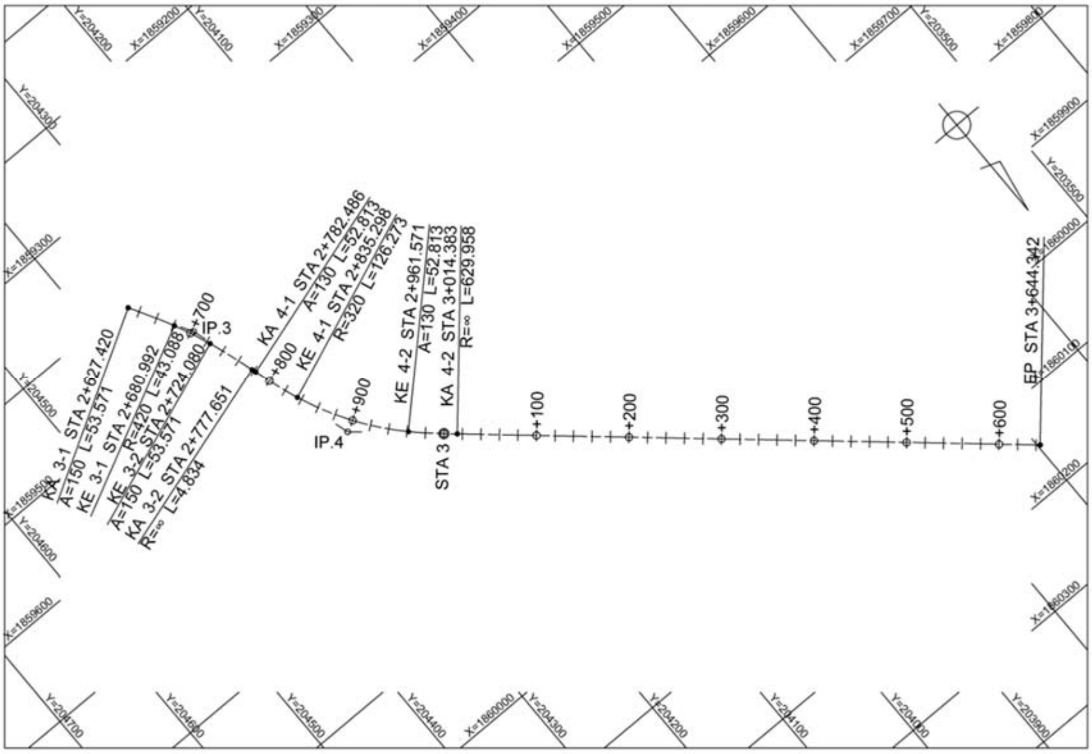
- Connecting to the centerline of the main route at the beginning point. Alignment of the main route is shifted 15 m in the detailed design (D/D) from that of the supplemental F/S.
- Passing over Shukhinthar Intersection.
- Passing over Yadanar Intersection.
- Connecting to the center of Thanlyin Chin Kat Road at the end point.
- Minimum radius of 310 m is applied for the horizontal curve. It is the radius which does not need the widening of carriageway.



Source: JICA Study Team

Figure 3.5.7 Outline of the Horizontal Alignment of the Flyover

The horizontal alignment of the flyover section is shown in Figure 3.5.8.



Source: JICA Study Team

Figure 3.5.8 Horizontal Alignment of the Flyover Section

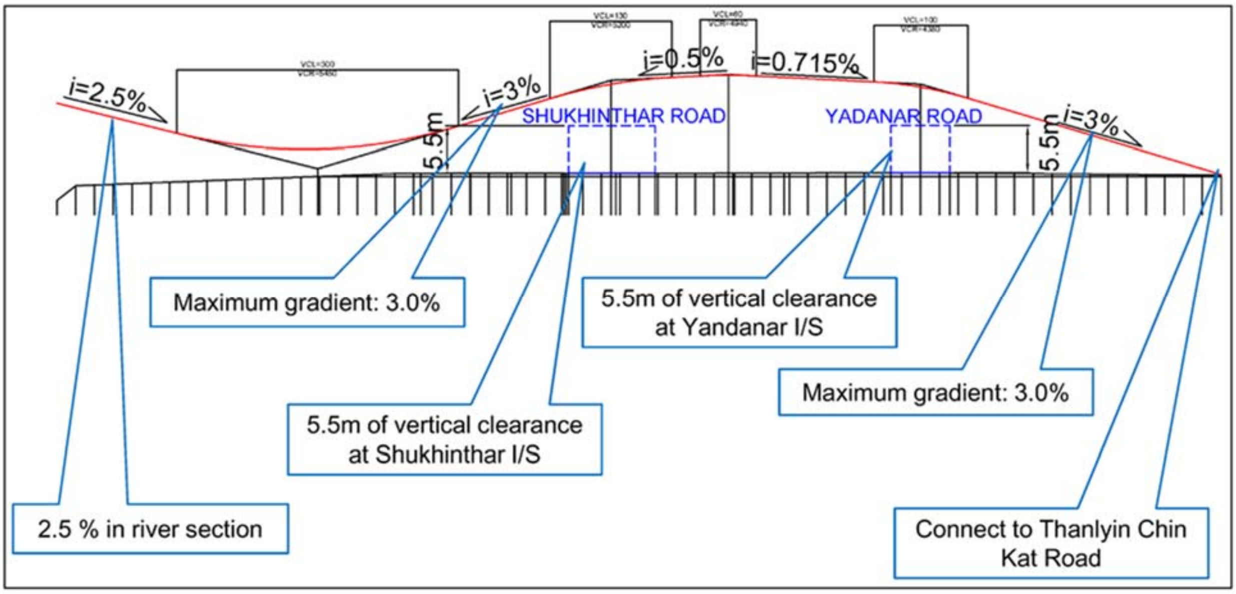
(2) Vertical Alignment

The vertical alignment of the flyover was determined taking into account the following conditions:

- Applying 3.0% as maximum vertical gradient in consideration of smooth driving of heavy vehicles.
- Applying 0.5% as minimum vertical gradient in consideration of discharge of rainwater from the road surface.
- Applying 5.5 m of vertical clearance under the flyover based on the request of YCDC.

  Ensuring 5.5 m of vertical clearance at Shukhintar Intersection

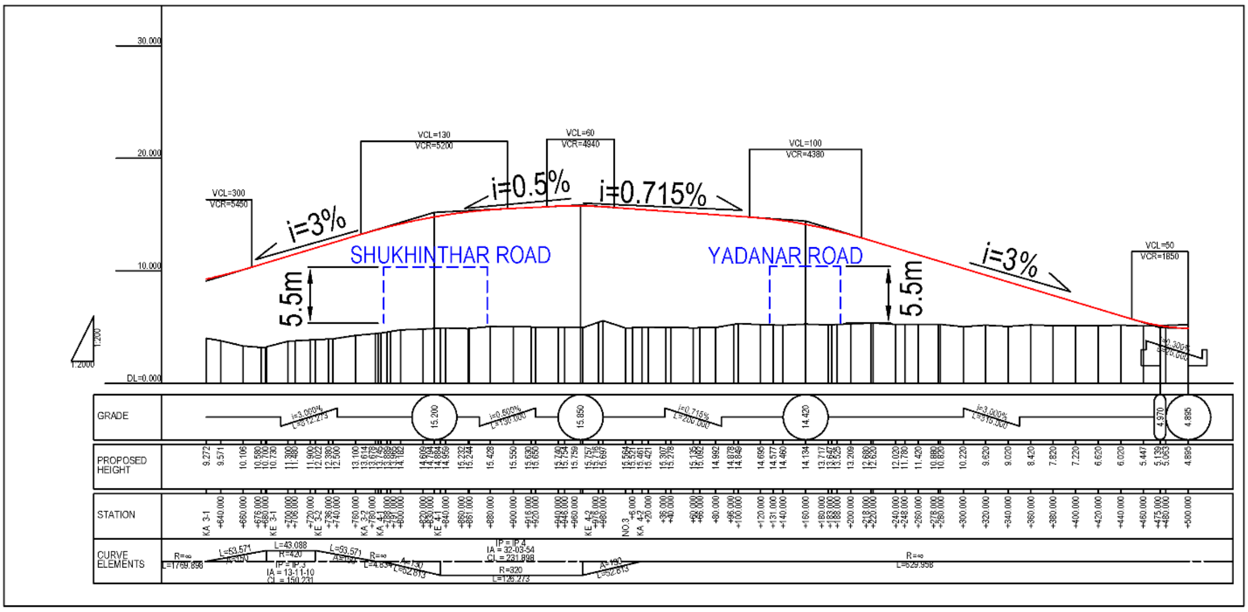
  Ensuring 5.5 m of vertical clearance at Yadanar Intersection



Source: JICA Study Team

Figure 3.5.9 Outline of the Vertical Alignment of the Flyover

The vertical alignment of the flyover section is shown in Figure 3.5.10.



Source: JICA Study Team

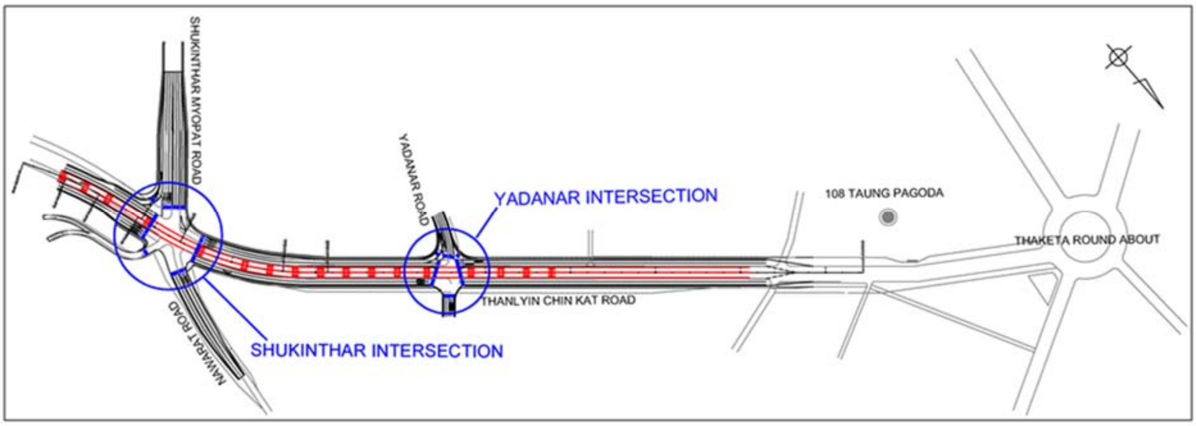
Figure 3.5.10 Vertical Alignment of the Flyover Section

3.5.3 Intersection Design

(1) Introduction

The following two intersections in Thanlyin Chin Kat Road will be improved in the Project:

- Shukhinthar Intersection around STA. 2+830
- Yadanar Intersection around STA. 3+160



Source: JICA Study Team

Figure 3.5.11 Improvement of Two Intersections in Thanlyin Chin Kat Road

(2) Traffic Volume

The daily traffic volume in 2035 forecasted in the "Project for Comprehensive Urban Transport Plan of the Greater Yangon, JICA, 2014 (YUTRA)" is used for the intersection capacity analysis.

The daily traffic volumes for each direction in Shukhinthar Intersection and Yadanar Intersection are shown in Table 3.5.8, Table 3.5.9, and Figure 3.5.12.

Table 3.5.8 Daily Traffic Volume in 2035 at Shukhinthar Intersection

Unit: PCU/day

Direction Code	2016	2017	2018	2019	2020	2021	2022	2023	2024	2025
3	740	930	1,120	1,320	1,510	2,210	2,920	3,630	4,340	5,050
4	9,380	9,320	9,260	9,200	9,140	9,960	10,770	11,590	12,410	13,220
5	8,480	9,910	11,350	12,790	14,220	12,680	11,140	9,600	8,060	6,510
6	1,050	1,160	1,270	1,370	1,480	2,010	2,540	3,070	3,610	4,140
7	4,370	4,780	5,180	5,590	5,990	5,750	5,500	5,250	5,010	4,760
8	100	100	100	100	90	310	520	740	950	1,160
9	110	110	120	120	120	350	570	800	1,020	1,250
10	8,820	8,540	8,260	7,990	7,710	8,530	9,350	10,170	10,990	11,810
11	1,270	1,200	1,130	1,060	990	920	840	770	700	620
12	1,570	1,460	1,350	1,240	1,140	1,130	1,130	1,120	1,110	1,110
13	6,350	7,310	8,270	9,220	10,180	9,660	9,150	8,630	8,110	7,590
14	8,350	9,060	9,780	10,490	11,200	9,550	7,900	6,240	4,590	2,940
Direction Code	**2026**	**2027**	**2028**	**2029**	**2030**	**2031**	**2032**	**2033**	**2034**	**2035**
3	4,830	4,610	4,400	4,180	3,960	3,930	3,900	3,870	3,840	3,810
4	12,860	12,490	12,120	11,760	11,390	11,620	11,860	12,090	12,320	12,560
5	6,600	6,680	6,760	6,840	6,930	7,550	8,170	8,780	9,400	10,020
6	4,090	4,030	3,970	3,920	3,860	3,840	3,820	3,800	3,790	3,770
7	5,070	5,370	5,680	5,990	6,290	6,530	6,770	7,010	7,250	7,480
8	1,220	1,280	1,340	1,400	1,460	1,510	1,560	1,610	1,660	1,710
9	1,150	1,060	960	870	770	770	770	770	760	760
10	11,460	11,110	10,750	10,400	10,050	10,170	10,300	10,420	10,550	10,670
11	670	710	750	800	840	870	900	920	950	980
12	1,120	1,140	1,160	1,170	1,190	1,220	1,260	1,300	1,340	1,370
13	8,090	8,580	9,080	9,570	10,070	10,470	10,870	11,280	11,680	12,080
14	3,230	3,530	3,820	4,120	4,410	4,780	5,150	5,510	5,880	6,250

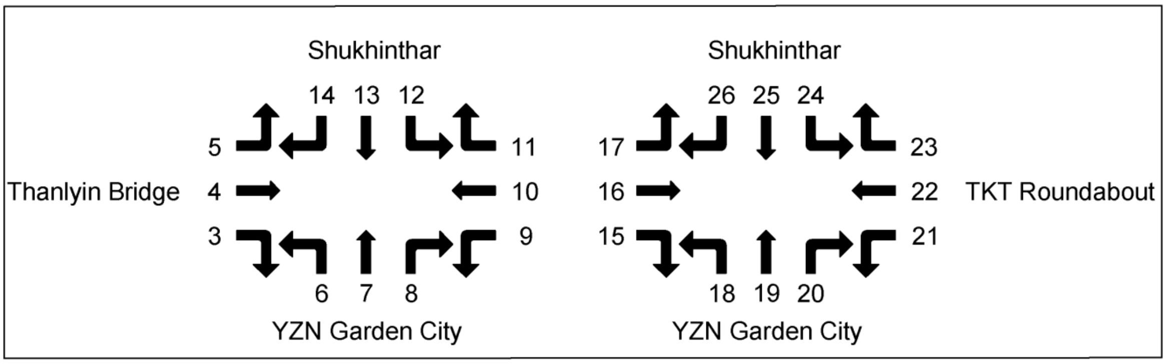
Source: YUTRA

Table 3.5.9 Daily Traffic Volume in 2035 at Yadanar Intersection

Unit: PCU/day

Direction Code	2016	2017	2018	2019	2020	2021	2022	2023	2024	2025
15	180	280	380	470	570	620	670	720	770	820
16	11,060	10,390	9,730	9,060	8,390	9,190	9,980	10,780	11,570	12,370
17	570	510	450	380	320	500	680	860	1,050	1,230
18	110	150	200	240	280	400	510	630	750	860
19	8,740	9,710	10,690	11,660	12,630	13,090	13,540	13,990	14,440	14,900
20	3,550	4,300	5,050	5,800	6,550	6,030	5,510	4,990	4,470	3,950
21	3,210	3,650	4,090	4,530	4,970	4,700	4,430	4,160	3,900	3,630
22	9,560	9,320	9,070	8,820	8,570	9,220	9,870	10,520	11,170	11,820
23	3,990	4,160	4,320	4,490	4,650	4,360	4,070	3,780	3,490	3,200
24	3,190	3,190	3,200	3,200	3,210	3,180	3,160	3,140	3,110	3,090
25	6,140	6,880	7,620	8,360	9,100	9,580	10,060	10,540	11,020	11,500
26	770	740	720	700	670	790	910	1,030	1,160	1,280
Direction Code	**2026**	**2027**	**2028**	**2029**	**2030**	**2031**	**2032**	**2033**	**2034**	**2035**
15	730	640	560	470	380	390	400	400	410	420
16	12,400	12,430	12,460	12,490	12,520	12,730	12,930	13,140	13,350	13,560
17	1,110	980	860	740	620	630	650	670	680	700
18	730	600	470	340	200	210	220	230	230	240
19	15,310	15,730	16,140	16,560	16,980	17,430	17,880	18,330	18,790	19,240
20	4,220	4,490	4,760	5,040	5,310	5,380	5,460	5,530	5,600	5,680
21	3,810	3,990	4,170	4,350	4,530	4,530	4,520	4,510	4,510	4,500
22	11,560	11,310	11,050	10,800	10,540	10,660	10,780	10,890	11,010	11,130
23	3,540	3,870	4,210	4,550	4,890	4,830	4,760	4,700	4,640	4,570
24	3,320	3,550	3,770	4,000	4,230	4,240	4,250	4,260	4,270	4,280
25	11,640	11,780	11,920	12,060	12,200	12,520	12,840	13,160	13,480	13,800
26	1,200	1,110	1,030	950	870	900	920	950	970	1,000

Source: YUTRA



Source: YUTRA

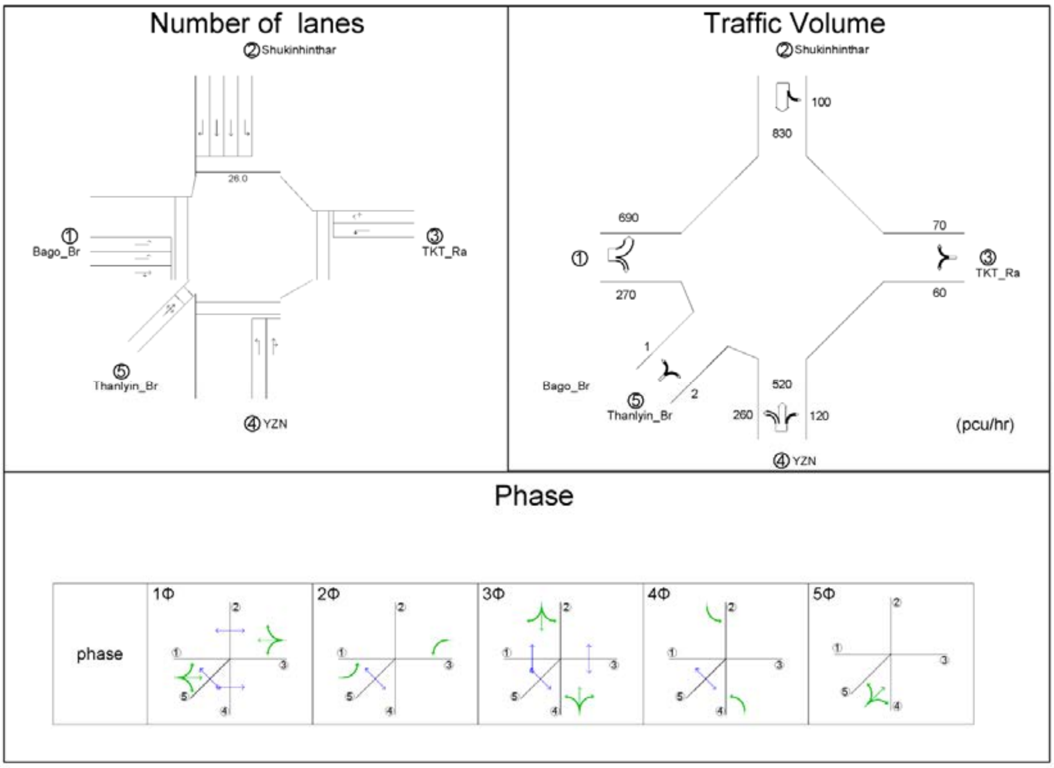
Figure 3.5.12 Direction Code for Traffic Volume in Shukhinthar Intersection and Yadanar Intersection

The hourly traffic volume utilized for the intersection capacity analysis is calculated with peak ratio of 6.9%.

(3) Intersection Capacity Analysis

1) Shukhinthar Intersection

The conditions for capacity analysis of Shukhinthar Intersection are shown in Figure 3.5.13.



Source: JICA Study Team

Figure 3.5.13 Conditions for Capacity Analysis of Shukhinthar Intersection

The result of capacity analysis of Shukhinthar Intersection is shown in Table 3.5.10.

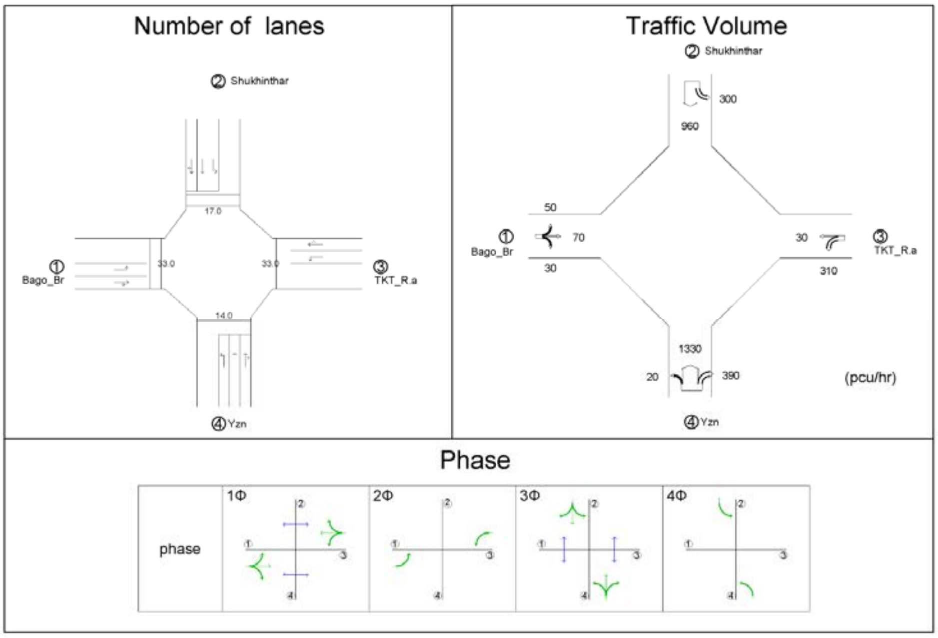
Table 3.5.10 Result of Capacity Analysis of Shukhinthar Intersection

Intersection name		Shukhinthar Intersection											
Entry		① : Bago_Br		② : Shukinhinthar			③ : TKT_Ra		④ : YZN		⑤ : Thanlyin_Br		
Direction		TH + RT	LT	RT	TH	LT	TH + RT	LT	TH + RT	LT	LT + TH + RT		
Number of Lane:		1	2	1	2	1	1	1	1	1	1		
Basic value of saturation flow rate (PCU/hr):SB		2000	1800	1800	2000	1800	2000	1800	2000	1800	2000		
Reduction coefficient: αw (Lane width: m)		1.000 (3.50)	1.000 (3.50)	1.000 (3.50)	1.000 (3.50)	1.000 (3.50)	1.000 (3.00)	1.000 (3.50)	1.000 (3.50)	1.000 (3.50)	1.000 (3.50)		
Reduction coefficient: αG (Gradient: %)		1.000 (0.00)	1.000 (0.00)	1.000 (0.00)	1.000 (0.00)	1.000 (0.00)	1.000 (0.00)	1.000 (0.00)	1.000 (0.00)	1.000 (0.00)	1.000 (0.00)		
Reduction coefficient: αT (Share of large vehicle: %)		1.000 (0.00)	1.000 (0.00)	1.000 (0.00)	1.000 (0.00)	1.000 (0.00)	1.000 (0.00)	1.000 (0.00)	1.000 (0.00)	1.000 (0.00)	1.000 (0.00)		
Reduction coefficient: αLT (Share of right turn: %)L% fp sec sec αL		0.901		1.000			0.901		0.980 (18.8)		0.932 (66.7)		
Reduction coefficient: αRT (Share of left turn: %)R% f sec KER : os./cycle K: nos./cycle						3(98)				3(98)	0.965 (33.3) 1.000 6		
Saturation flow ratio: SA		1802	3600	1800	4000	1800	1802	1800	1960	1800	1799		
Traffic volume (pcu/hr): q		270 (270+0)	690	0	830	100	70 (70+0)	60	640 (120+520)	260	3 (2+0+1)		
Traffic volume with compensation of left turn (pcu/hr): qR-N						2				162			
Flow ratio: ρ		0.150	0.192	0.000	0.207	0.001	0.039	0.033	0.327	0.090	0.002	Demand ratio of Phase	Demand ratio of intersection
Phase ratio	1φ	0.150	0.192				0.039	0.033				0.192	0.611
	2φ		****					****				****	
	3φ			0.000	0.207				0.327			0.327	
	4φ					0.001				0.090		0.090	
	5φ										0.002	0.002	
Current green time (sec):	1φ	31	31				31	31				Current cycle length (sec)	
	2φ		6					6				110	
	3φ			39	39				39				
	4φ					13				13			
	5φ										6		
Signal blue time ratio: G/C		31/110	37/110	39/110	39/110	13/110	31/110	37/110	39/110	13/110	6/110		
Capacity (pcu/hr): Ci		508	1211	638	1418	311	508	605	695	311	98		
Degree of Saturation: q/Ci		0.531	0.570	0.000	0.585	0.322	0.138	0.099	0.921	0.836	0.031		
Check		OK	OK	OK	OK	OK	OK	OK	OK	OK	OK		
Storage length: Ls(m)			94.9	0.0		36.5		24.2		76.3			

Source: JICA Study Team

2) Yadanar Intersection

The conditions for capacity analysis of Yadanar Intersection are shown in Figure 3.5.14.



Source: JICA Study Team

Figure 3.5.14 Conditions for Capacity Analysis of Yadanar Intersection

The result of capacity analysis of Yadanar Intersection is shown in Table 3.5.11.

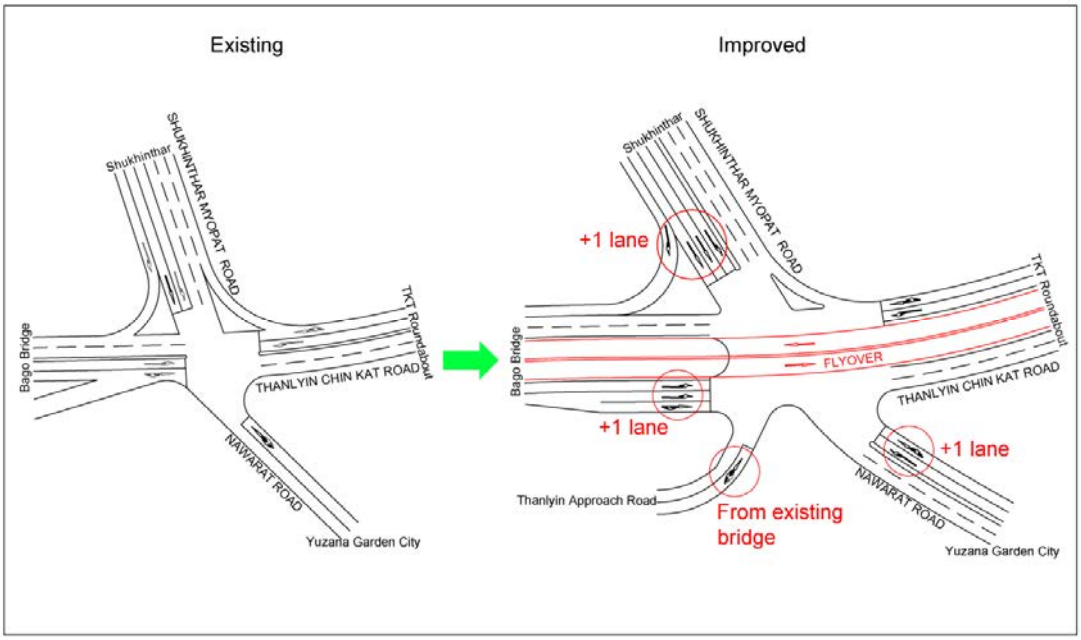
Table 3.5.11 Result of Capacity Analysis of Yadanar Intersection

交差点名		ヤダナー交差点											
流入部		①		②			③		④				
車線の種類		右折・直進	左折	右折・直進	直進	左折	右折・直進	左折	右折・直進	直進	左折		
車線数		1	1	1	1	1	1	1	1	1	1		
飽和交通流率の基本値	S B	2000	1800	2000	2000	1800	2000	1800	2000	2000	1800		
車線幅員による補正率 （車線幅員）	α w m	1. 000 (3. 50)	1. 000 (3. 50)	1. 000 (3. 00)	1. 000 (3. 00)	1. 000 (3. 00)	1. 000 (3. 50)	1. 000 (3. 50)	1. 000 (3. 00)	1. 000 (3. 00)	1. 000 (3. 00)		
縦断勾配による補正率 （縦断勾配）	α G %	1. 000 (0. 00)	1. 000 (0. 00)	1. 000 (0. 00)	1. 000 (0. 00)	1. 000 (0. 00)	1. 000 (0. 00)	1. 000 (0. 00)	1. 000 (0. 00)	1. 000 (0. 00)	1. 000 (0. 00)		
大型車混入による補正率 （大型車混入率）	α T %	1. 000 (0. 00)	1. 000 (0. 00)	1. 000 (0. 00)	1. 000 (0. 00)	1. 000 (0. 00)	1. 000 (0. 00)	1. 000 (0. 00)	1. 000 (0. 00)	1. 000 (0. 00)	1. 000 (0. 00)		
右折車混入による補正率 （右折率） （歩行者による低減率） （有効青時間） （歩行者用青時間） 横断歩行者による補正率	α L T L % f p 秒 秒 α L	0. 968 (30. 0)		1. 000 (0. 0)			1. 000 (0. 0)		0. 953 (45. 3)				
左折車混入による補正率 （左折率） （左折車の通過確率） （有効青時間） （現示変り目のさばけ台数増分） KER：台/サイクル （交差点内滞留台数） K：台/サイクル	α R T R % f 秒		3 (108)			3 (108)		3 (108)			3 (108)		
飽和交通流率	S A	1936	1800	2000	2000	1800	2000	1800	1906	2000	1800		
設計交通量	q	100 (30+70)	50	960 (0+960)		300	30 (0+30)	310	1720 (390+1330)		20		
左折補正交通量	q R − N		0			192		202			0		
交差点流入部の需要率	ρ	0. 052	0. 000	0. 240		0. 107	0. 015	0. 112	0. 440		0. 000	現示の需要率	交差点の需要率
必要現示率	1 φ	0. 052					0. 015					0. 052	0. 711
	2 φ		0. 000					0. 112				0. 112	
	3 φ			0. 240					0. 440			0. 440	
	4 φ					0. 107					0. 000	0. 107	
有効青時間（秒）	1 φ	22					22					サイクル長（秒）	
	2 φ		12					12				100	
	3 φ			45					47				
	4 φ					11					11		
信号青時間比	G / C	22/100	12/100	45/100		11/100	22/100	12/100	47/100		11/100		
可能交通容量	C i	426	324	1800		306	440	324	1836		306		
交通容量比	q / C i	0. 235	0. 154	0. 533		0. 980	0. 068	0. 957	0. 937		0. 065		
交通処理案のチェック		OK	OK	OK		OK	OK	OK	OK		OK		
滞留長	L s (m)		18. 3			79. 0		81. 1			7. 3		

Source: JICA Study Team

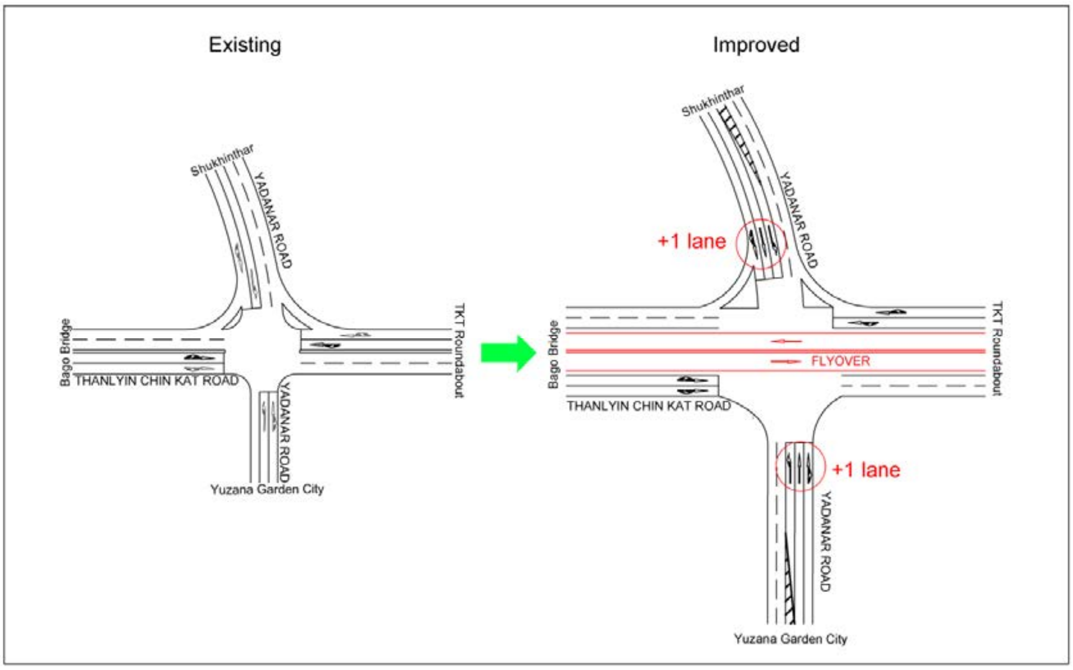
(4) Improvement of Intersection

Based on the intersection capacity analysis, Shukhinthar Intersection and Yadanar Intersection will be improved as shown in Figure 3.5.15 and Figure 3.5.16.



Source: JICA Study Team

Figure 3.5.15 Improvement of Shukhinthar Intersection



Source: JICA Study Team

Figure 3.5.16 Improvement of Yadanar Intersection

3.5.4 Earthwork

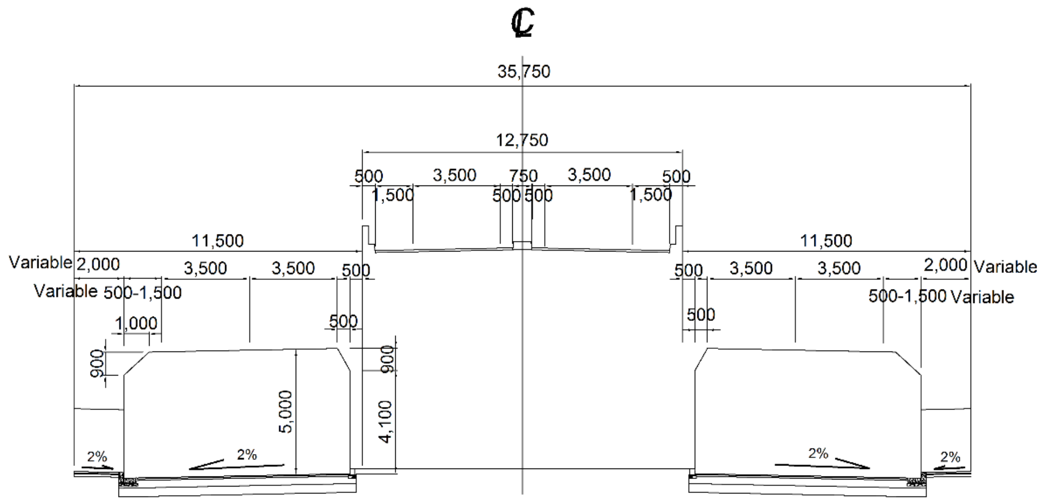
Based on the analysis result in Section 2.2.3, the slope gradient for the embankment is set at 1:1.8.

3.5.5 Embankment Design behind Abutment

In Myanmar, a large gap on the road surface caused by a large settlement behind the abutment has been observed, and the deformation is mainly caused by defects during construction and/or consolidation settlement. According to the soil investigation results in the Supplemental F/S, it was observed that the soft ground layer is spread out in the project area; thus, the necessity of countermeasures should be examined. When an earthquake occurs, moreover, the settlement of the approach road embankment would cause critical damage as well as can be damaging to the bridge, in which roads will be temporarily closed to traffic for rehabilitation. Considering these points, the necessary soft soil ground treatment and retaining wall type will be studied in this section.

3.5.5.1 Selection of Retaining Wall Structure Type

A vertical wall type should be applied as the retaining wall structure for the approach road in the flyover section in order to minimize the road width as well as land acquisition. Some alternatives are prepared considering the maximum wall height (approximately 7 m) and the ground condition (soft soil ground). The appropriate structure type will be determined considering construction cost, structural stability, and construction period.



Source: JICA Study Team

Figure 3.5.17 Typical Cross Section for Approach Road on Flyover Section

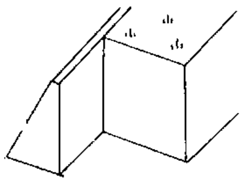
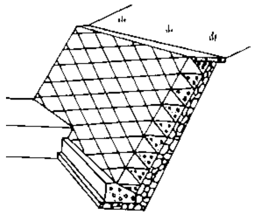
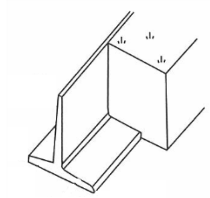
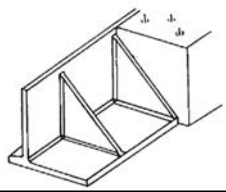
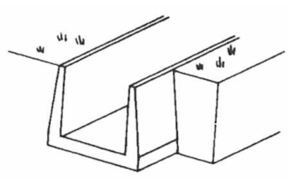
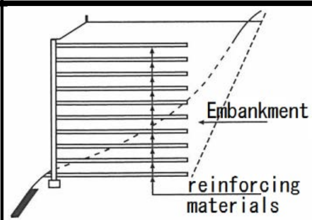
(1) Alternatives for Retaining Wall Structure

1) Retaining Wall Structure

Three alternatives, namely: “Cantilever Retaining Wall”, “U-shaped Retaining Wall”, and “Mechanically-stabilized Earth Wall” are prepared considering the following conditions:

- Maximum wall height: 7 m
- Ground condition: Soft soil
- Desirable wall slope: Vertical

Table 3.5.12 Alternatives for Retaining Wall Structure

Items	Schematic View	Generally Applicable Wall Height	Generally Applicable Wall Slope	Applicable Ground Condition
Gravity Retaining Wall		H ≦5 m	1:0.2～	Good-quality bearing layer (Inapplicable to soft soil ground)
Concrete Block Retaining Wall		H ≦7 m	1:0.3～	Good-quality bearing layer (Inapplicable to soft soil ground)
Cantilever Retaining Wall (L or Reversed T-shape)		**3 m≦H≦10 m**	**Vertical**	**Any (Applicable even to soft ground if pile foundation or soft ground treatment are applied)**
Counterfort Retaining Wall		10 m≦H	Vertical	Any (Applicable even to soft ground if pile foundation or soft ground treatment are applied)
U-shaped Retaining Wall		**Any height**	**Vertical**	**Any (Applicable even to soft ground if pile foundation or soft ground treatment are applied)**
Mechanically-stabilized Earth Wall		**3 m≦H≦10 m**	**Vertical**	**Any (Applicable even to soft ground if pile foundation or soft ground treatment are applied)**

Source: Prepared by JICA Study Team based on JSHB

2) Foundation Structures

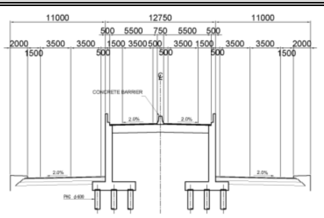
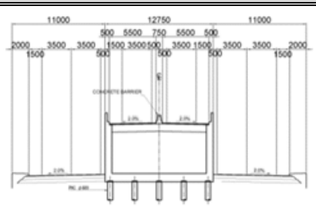
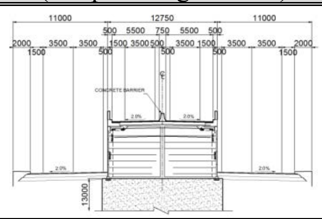
"Pile Foundation" or "Combination of Spread Foundation and Soft Soil Treatment" can be applied as the foundation type of retaining wall on soft soil ground. In case of "U-shape Retaining Wall" and "Cantilever Retaining Wall", since they are rigid structures, the expected ground reaction is larger than that of "Mechanically-stabilized Earth Wall" which is a flexible structure. If soft ground treatment is combined with these rigid retaining walls, the required improvement strength tends to be much larger, thus the combination may result in an uneconomical option. Therefore, the pile foundation is generally adopted for the foundation type of rigid retaining walls. On the other hand, the combination with soft ground treatment is recommended for “Mechanically-stabilized Earth Wall” since piled slab should be installed under a reinforced earth wall for structural stability, which is obviously an uneconomical option in case of combination with pile foundation. “Reversed T-shape Retaining Wall” is recommended for the “Cantilever Retaining Wall” since it is superior in terms of structural stability compared to other types such as “L-shape” or “Reversed L-shape” and has no constraint in front of retaining walls. Alternatives for retaining wall structures are listed below.

Alternative-1: Cantilever Retaining Wall (T-shape) + Pile Foundation Alternative-2: U-shape Retaining wall + Pile Foundation (Original plan in JICA Supplemental Preparatory Survey) Alternative-3: Mechanically-stabilized Earth Wall + Soft Soil Ground Treatment (Deep Mixing Method)

(2) Evaluation

As a result of the comparative study, “Alternative-3: Mechanically-stabilized Earth Wall + Soft Soil Ground Treatment by Deep Mixing Method” is selected for the retaining wall of the approach road in the flyover section as given in Table 3.5.13.

Table 3.5.13 Selection of Retaining Wall Structure

Evaluation Item	Alt-1 Cantilever Retaining Wall (T-shape) + Pile Foundation		Alt-2 U-shape Retaining Wall + Pile Foundation (Plan at F/S)		Alt-3 Mechanically-stabilized Earth Wall + Soft Ground Treatment (Deep Mixing Method)	
Schematic View						
Structural Aspect	Applicable wall height: 3-10 m Supported by piles for structural stability	Fair	Applicable wall height: Any Supported by piles for structural stability No. of piles is less than Alt-1 due to less uneven earth pressure	Good	Applicable span length: 3-18 m Soft ground treatment is necessary	Fair
Construction Cost	Ratio = 1.94	Poor	Ratio = 1.33	Fair	Ratio = 1.00	Good
Construction Period	3.7 months / 20 m	Poor	3.9 months / 20 m	Poor	1.1 months / 20 m	Good
Evaluation					**Recommended**	

Source: JICA Study Team

3.5.5.2 Soft Ground Treatment

(1) Verification of Possibility of Consolidation Settlement by New Embankment

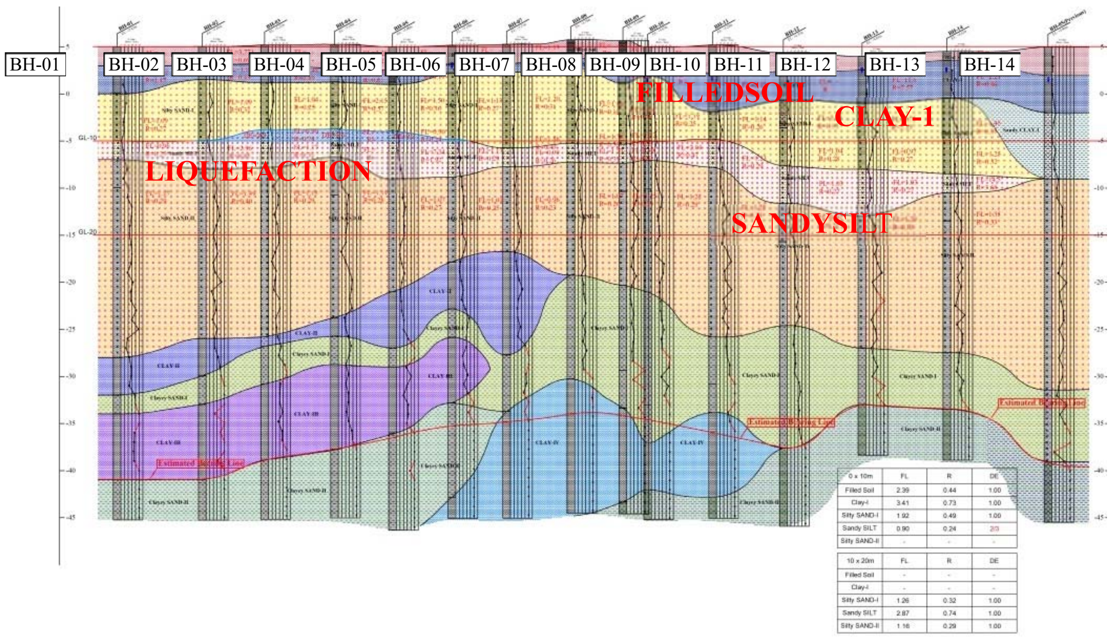
Considering the soft ground in the project area, consolidation settlement of cohesive soil due to the new embankment is a concern. Concerned layers are Filled Soil, Clay-I, and Sandy Silt among the identified soil layers based on the soil investigation survey as shown in Figure 3.5.18 Since the N value of Filled Soil and Clay-I is relatively small (N=4), the possibility of consolidation settlement was examined by utilizing the laboratory consolidation tests. In the verification, the proposed embankment height was considered at each boring location as well as vertical load equivalent to live load. As a result of the calculation, the total settlement including immediate settlement is from 28 cm to 73 cm for these layers which indicates that proper countermeasures are necessary as shown in Table 3.5.14.

On the other hand, as for the Sandy Silt layer, it can be concluded that the possibility of consolidation settlement is quite low since the mean N value is 9 and sufficient strength improvement (bearing capacity) is expected to be secured by the accumulated load of the existing embankment and upper soil layers.

Table 3.5.14 Study Results of Consolidation Settlement

Borehole No.	Embankment Height (m)	Vertical Load (kN/m^2)	Immediate Settlement (cm)	Total Settlement (cm)	Borehole Location	
BH-01	1.750	30.0	5.83	27.83	NO.3+421.5	Approach section
BH-02	4.400	11.6	13.16	36.68	NO.3+331.1	Approach section
BH-03	6.300	11.6	11.26	45.88	NO.3+265.9	Bridge section
BH-12	6.400	11.6	7.21	73.30	NO.2+714.4	Bridge section
BH-13	3.900	11.6	11.49	29.90	NO.2+631.0	Approach section
BH-14	3.500	11.6	11.85	23.18	NO.2+541.0	Approach section

Source: JICA Study Team



Source: JICA Study Team

Figure 3.5.18 Soil Profile

(2) Assessment of Soil Liquefaction

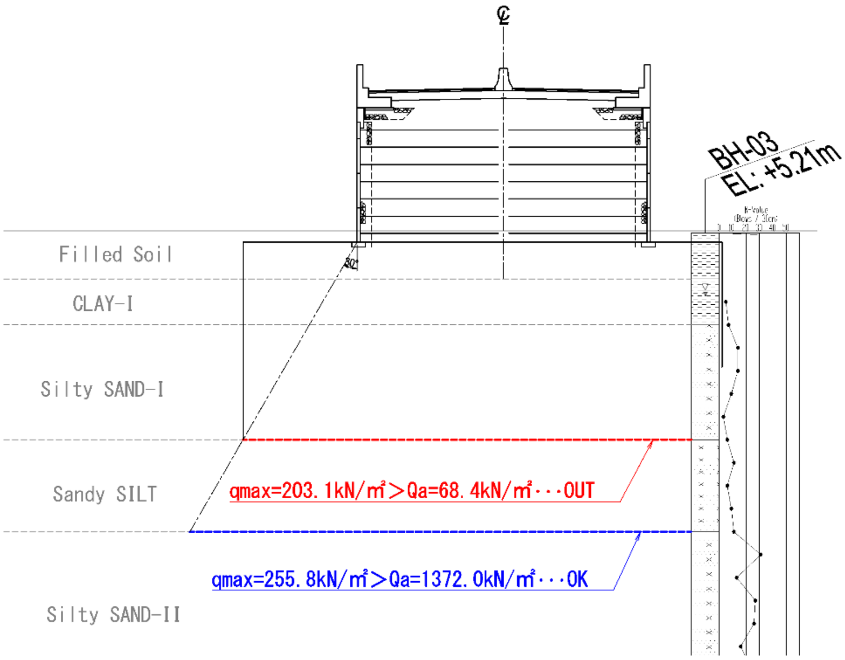
The possibility of soil liquefaction was assessed for saturated soil layers less than 20 m in depth from the existing ground surface. The assessment method specified in JSHB-V is applied. The ground type in the flyover section is classified as "Type III" in the specification and the ground water level was observed at less than -5.0 m from the existing ground level according to the soil investigation results for the flyover section. For each layer of the 14 boreholes, the reduction coefficient, DE, was calculated using the mean value of the resistivity against liquefaction, FL, and the dynamic shear strength ratio, R, as specified in JSHB-V. As a result, for the upper Sandy Silt layer up to -10 m from the existing ground level, the reduction factor (DE) is calculated as 0.677 (less than 1.0), which indicates that there is a possibility of liquefaction. Therefore, soft ground treatment is necessary for the foundation ground of the mechanically-stabilized earth wall on the approach road section. The detailed assessment results of liquefaction are described in Section 4.6.4.3(2).

(3) Applied Soft Ground Treatment Method and Depth

As described above, consolidation settlement of cohesive soil due to the new embankment and liquefaction due to earthquakes are a concern in the flyover section and soft ground treatment is required for the approach road in the flyover section.

"Deep mixing method" is applied as the soft ground treatment in the flyover section as well as other sections.

The depth of soft ground treatment can be determined by the required bearing capacity under the mechanically-stabilized earth wall as given in Figure 3.5.19. As a result, soft ground treatment should be applied to the bottom level of the Sandy Silt layer.

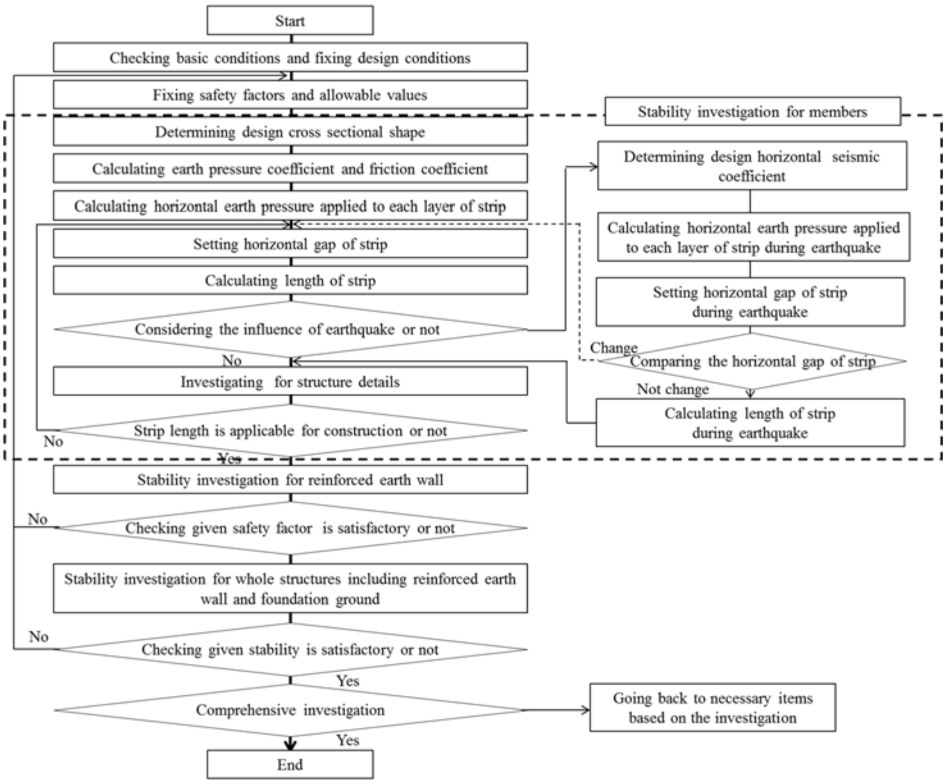


Source: JICA Study Team

Figure 3.5.19 Verification Result of Bearing Capacity of Mechanically-stabilized Earth Wall

3.5.5.3 Design Method of Mechanically-stabilized Earth Wall

Figure 3.5.20 shows the design procedure for the mechanically-stabilized earth wall. The design criteria specified in the "Manual for Design and Construction of Mechanically-stabilized Earth Wall, 4th Edition" issued by the Public Works Research Center (Japan) is applied to this design.



Source: Guideline for Retaining Wall Structures, Japan Road Association (2012)

Figure 3.5.20 Design Procedure for Mechanically-stabilized Earth Wall

(1) Determination of Required Performance

Table 3.5.15 shows the required performance for the assumed actions.

Table 3.5.15 Required Performance

Load Status \ Degrees of Importance		Level 1
Ordinary		Performance 1
Rainfall		Performance 1
Earthquake	Level 1 Earthquake	Performance 1
	Level 2 Earthquake	Performance 2

Source: JICA Study Team

Where, Performance 1: Keeping sound functions

Performance 2: Limited damage and capable of recovering required function in a short period

Performance 3: No critical damage on the soundness of reinforced earth wall

(2) Design Horizontal Seismic Coefficient

Design horizontal seismic coefficients are described as follows:

1) Safety Verification for Wall Members

$$k_{h1} = C_z \cdot k_{h0} = 0.18$$

Where, k_{h1} is the design horizontal seismic coefficient for safety verification

2) External Stability Verification for Mechanically-stabilized Earth Wall

$k_{h2} = C_z \cdot k_{h0} \cdot \upsilon = 0.13$

Where, k_{h2} is the design horizontal seismic coefficient for external stability verification

υ is the correction coefficient = 0.7.

3) Entire Stability Verification for Mechanically-stabilized Earth Wall

$k_{h3} = C_z \cdot k_{h0} = 0.12$

Where, k_{h3} is the design horizontal seismic coefficient for entire stability verification

C_z is the standard value of horizontal seismic coefficient = 0.12 (Ground type III, Level 1 earthquake)

(3) Allowable Stress and Design Safety Factors

Table 3.5.16 shows the allowable stress and design safety factors under ordinary and earthquake load.

Table 3.5.16 Allowable Stress and Type of Safety Factors

Allowable Stress and Type of Safety Factors	Ordinary	Earthquake
Tensile allowable stress of strip (N/mm^2)	σa=185.0	σaE=277.0
Shear allowable stress of bolt (N/mm^2)	τa=200.00	τaE=300.00
Safety factor for pulling of strip	Fs≧2.00	FsE≧1.20
Safety factor for sliding	Fs≧1.50	FsE≧1.20
Safety condition for overturning	e≦L/6	e≦L/3
Safety factor for bearing capacity	Fs≧3.00	FsE≧2.00
Safety factor for circular slip	Fs≧1.20	FsE≧1.00

Source: RA Highway Bridge Specification V

(4) Material

The details of the materials for mechanically-stabilized earth wall are shown in Table 3.5.17.

Table 3.5.17 Details of Material

<table>
<tr><th>Members</th><th>Item</th><th>Code</th><th>Unit</th><th>On land
(In water)</th></tr>
<tr><td rowspan="2">Bolt</td><td>Nominal diameter</td><td>d</td><td>mm</td><td>M12</td></tr>
<tr><td>Effective cross section area</td><td>Ae</td><td>mm^2</td><td>84.3</td></tr>
<tr><td colspan="2">Number of bolt per one joint connection</td><td>n</td><td>Nos</td><td>1</td></tr>
<tr><td colspan="2">Number of bolt in width direction of strip</td><td>n’</td><td>Nos</td><td>1</td></tr>
<tr><td colspan="2">Number of shear</td><td>j</td><td>Nos</td><td>2</td></tr>
<tr><td rowspan="3">Strip</td><td>Plate width</td><td>b</td><td>Mm</td><td>60.0</td></tr>
<tr><td>Plate thickness</td><td>t</td><td>Mm</td><td>4.0</td></tr>
<tr><td>Corrosion allowance</td><td>cm</td><td>Mm</td><td>1.0
(1.5)</td></tr>
</table>

Source: RA Highway Bridge Specification V

3.5.5.4 Basic Design Result

The basic design results are shown in the following tables.

Table 3.5.18 Basic Design Calculation Results of Mechanically-stabilized Earth Wall

(1) Safety Verification for Strip Length and Pulling

No. of Column (i)	Depth zi (m)	Vertical Distance ΔH(m)	Horizontal Distance ΔB(m)	Design Length L(m)	Necessary Length		Safety Factor for Drawing		Evaluation
					Ordinary Lr (m)	Earthquake LrE (m)	Ordinary Fs	Earthquake FsE	
1	1.725	0.750	0.750	6.000	5.298	5.407	**2.423 (>2.00)**	**1.451 (>1.20)**	**OK**
2	2.475	0.750	0.750	6.000	5.404	5.320	**2.348 (>2.00)**	**1.497 (>1.20)**	**OK**
3	3.225	0.750	0.750	6.000	5.529	5.330	**2.265 (>2.00)**	**1.492 (>1.20)**	**OK**
4	3.975	0.750	0.750	6.000	5.274	4.868	**2.393 (>2.00)**	**1.682 (>1.20)**	**OK**
5	4.725	0.750	0.750	6.000	5.006	4.391	**2.512 (>2.00)**	**1.859 (>1.20)**	**OK**
6	5.475	0.750	0.750	6.000	4.782	3.961	**2.593 (>2.00)**	**1.993 (>1.20)**	**OK**
7	6.225	0.750	0.750	6.000	4.527	3.960	**2.685 (>2.00)**	**2.131 (>1.20)**	**OK**

(2) Stress of Wall Components

No. of Column (i)	Ordinary			Earthquake			Evaluation
	σ_{ti} (N/mm^2)	σ_{0ti} (N/mm^2)	τ_{0i} (N/mm^2)	σ_{tEi} (N/mm^2)	σ_{0tEi} (N/mm^2)	τ_{0Ei} (N/mm^2)	
1	59.1 (<185.0)	59.1 (<185.0)	47.3 (<200.0)	65.9 (<277.0)	65.9 (<277.0)	52.8 (<300.0)	OK
2	75.2 (<185.0)	75.2 (<185.0)	60.2 (<200.0)	84.7 (<277.0)	84.7 (<277.0)	67.8 (<300.0)	OK
3	89.6 (<185.0)	89.6 (<185.0)	71.7 (<200.0)	101.7 (<277.0)	101.7 (<277.0)	81.4 (<300.0)	OK
4	102.1 (<185.0)	102.1 (<185.0)	81.7 (<200.0)	116.8 (<277.0)	116.8 (<277.0)	93.5 (<300.0)	OK
5	112.8 (<185.0)	112.8 (<185.0)	90.3 (<200.0)	130.1 (<277.0)	130.1 (<277.0)	104.2 (<300.0)	OK
6	121.7 (<185.0)	121.7 (<185.0)	97.5 (<200.0)	141.5 (<277.0)	141.5(<277.0)	113.3 (<300.0)	OK
7	131.2 (<185.0)	131.2 (<185.0)	105.1 (<200.0)	153.4 (<277.0)	153.4(<277.0)	122.8 (<300.0)	OK

Where, σ_{ti} and σ_{tEi} are tensile stresses at the general parts of strip.

σ_{0ti} and σ_{0tEi} are tensile stresses of strip at joint connections between skin and strip.

τ_{0i} and τ_{0Ei} are shear stresses of bolt at joint connections between skin and bolt.

(3) External Stability of Mechanically-stabilized Earth Wall

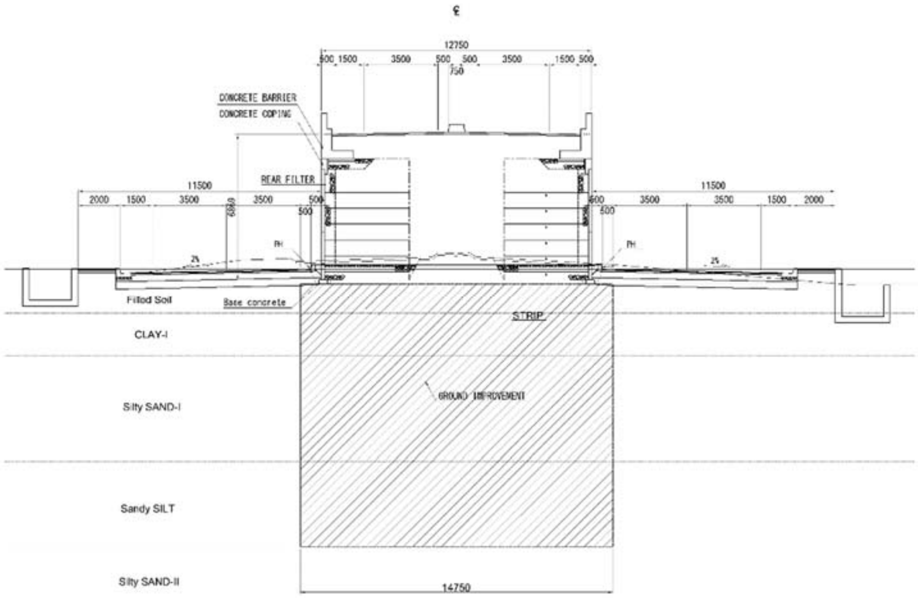
The value inside ( ) means an allowable value.

Item		Under Stationary		During Earthquake	
		Calculation Result	Evaluation	Calculation Result	Evaluation
Stability for sliding	Safety factor for sliding Fs	3.930 (>1.500)	OK	2.023 (>1.200)	OK
Stability for overturning	Eccentricity distance e (m)	0.063 (< 1.023)	OK	0.459 (<2.047)	OK
Stability for bearing capacity underneath embankment	Vertical counterforce qs (kN/m^2)	**146.197 (< 150.000)**	**OK**	138.840 (< 225.000)	OK
Stability for bearing capacity of foundation of wall underneath embankment	Vertical counterforce qos (kN/m^2)	372.817 (< 729.873)	OK	365.460 (< 967.885)	OK
Stability for bearing capacity underneath walls	Vertical counterforce qw (kN/m^2)	**120.164 (< 150.000)**	**OK**	108.462 (< 225.000)	OK

(4) Entire Stability Including Mechanically-stabilized Earth Wall (Safety Verification by Circular Slip Method)

Reinforcement Area (Inside/Outside)	Case	Center Coordinate of Circular Arc		Radius R (m)	Fsmin	Fs	Evaluation
		X (m)	Y (m)				
Inside of reinforcement area	Under stationary-3	-16.500	20.000	25.354	**1.423**	**1.200**	**OK**
	During earthquake-3	-7.000	20.500	20.954	1.576	1.000	OK
Outside of reinforcement area	Under stationary-1	-2.000	7.500	11.068	1.791	1.200	OK
	During earthquake-1	-3.000	10.000	13.548	1.459	1.000	OK

Source: JICA Study Team



Source: JICA Study Team

Figure 3.5.21 Typical Cross Section of Mechanically-stabilized Earth Wall

3.5.6 Detailed Design of Retaining Wall

3.5.6.1 Major Updates in Detailed Design from Basic Design

(1) Modification on Design Soil Parameter

Since the number of boring results used to determine the parameters was increased in D/D, the design soil parameter were updated in D/D. Table 3.5.19 shows the soil modulus in the project area.

Table 3.5.19 Soil Modulus

Layer	N Average *1	Unit Weight "γ" (kN/m3)	Cohesion "c" (kN/m2)	Friction Angle "φ" *5 (°)	Modulus of Deformation "E" (kN/m2)
FILLED SOIL	4	18 *3	24 *4	0	1300 *6
CLAY-I	4	18 *2	24 *1	0	1300 *6
SILTY SAND-I	10	18 *2	0 *4	32	5000 *8
SANDY SILT	8	17 *3	48 *4	0	5600 *7
SILTY SAND-II	22	19 *3	0 *4	33	15400 *7
CLAY-II	21	18 *3	126 *4	0	14700 *7
CLAYEY SAND-I	35	19 *3	0 *4	33	24500 *7
CLAY-III	35	18 *3	210 *4	0	24500 *7
CLAYEY SAND-II	50	19 *3	0 *4	37	35000 *7
CLAY-IV	50	18 *3	300 *4	0	35000 *7

Source: JICA Study Team

*1 Maximum N value is 50
*2 Average values obtained by each tests
*3 Referenced by Japanese Standard (NEXCO)
*4 Calculated by C=6N (referenced by Japanese Standard (NEXCO)). The value of sandy soil is 0.
*5 Calculated with N value using effective overburden pressure
*6 Test value obtained by unconfined compression test
*7 E=700N according to the worth value obtained by borehole lateral load test
*8 E=500N according to the worth value obtained by borehole lateral load test

Source: JICA Study Team

(2) Area of Soil Treatment

In the section of mechanically stabilized wall, the area of soft soil treatment will be up to the bottom of SANDY SILT and the deep mixing method will be applied. In the section of L-shaped retaining wall, the area will be up to the bottom of CLAY-I and the medium-deep mixing method will be applied.

3.5.6.2 Design Conditions

(1) Design Criteria

- Japan Road Association (2012), Road Earthwork: Retaining Wall Guidelines 2012.
- Japan Road Association (2012), Specifications for Highway Bridges, the Commentary.
- Civil Engineering Research Center (2014), Reinforced Soil (Terre Armee) Wall Design and Construction Manual, 4th Rev Ed.
- American Association of State Highway and Transportation Officials (2012), AASHTO LRFD Bridge Design Specifications, 6th Ed (US)

(2) Soil Condition

- Embankment Material: γ γ ban kN/m^3,C=0 kN/m^2
- Foundation Ground:

Mechanically-stabilized earth wall: γ γ infor kN/m^3, φ=33.0, C=0.0 kN/m^2 (SILTY SAND-Ⅱ)

L-shaped retaining wall: γ γ shape kN/m^3, φ=32.0, C=0.0 kN/m^2 (SILTY SAND-Ⅰ)

(3) Materials

- Concrete (Protective fence foundation, L-type retaining wall): σ ck=24 N/mm^2
- Levelling Concrete: σ ck=18 N/mm^2
- Re-bar: SD345

(4) Dimensions of Mechanically-stabilized Earth Wall

- Bolt: Nominal diameter M12 (mm)
- Strip: PL-SS400(Width, b=80.0 mm × Thickness, t=4.0 mm × Corrosion Allowance, Cm=1.0 mm)

(5) Load Condition

1) Dead Loads

- Plain concrete: 23.0 kN/m^3
- Reinforced concrete: 24.5 kN/m^3
- Backfilling material: 18.0 kN/m^3
- Water: 10.0 kN/m^3

2) Working Load

- Roadway 11.6 kN/m^2 (AASHTO LRFD 2012 Bridge)

3) Collision Load

- Rigid protection fence (SC Type) P=43 kN/m

(6) Allowable Stress

1) Concrete Allowable Stress

The allowable compressive stress level and allowable shear strength of concrete values are shown in the following Table 3.5.20.

Table 3.5.20 Allowable Compressive Stress Level of Concrete and Allowable Shear Strength (N/mm^2)

Types of Stress Intensity \ Concrete Design Standard Strength (σ_{ck})		21	24
Compressive stress	Bending compressive stress	7.0	8.0
	Axial compressive stress	5.5	6.5
Shear stress	In case of bearing shear force only with concrete ($\tau a1$)	0.22	0.23
	In case of bearing shear force with concrete and diagonal tension bar ($\tau a2$)	1.6	1.7
	Punching shear stress intensity ($\tau a3$)	0.85	0.90

Source: JICA Study Team

The following Table 3.5.21 shows the allowable unit bond stress of concrete for re-bar with a diameter of 5 mm or less.

Table 3.5.21 Allowable Unit Bond Stress of Concrete

(N/mm^2)

Concrete Design Standard Strength (σ_{ck})	21	24
Bond Stress (Deformed re-bars)	1.4	1.6

Source: JICA Study Team

2) Allowable Stress Level of Re-bar

The following Table 3.5.22 shows the allowable stress level of re-bar with a diameter of 51 mm or less.

Table 3.5.22 Allowable Stress Level of Re-bar

(N/mm^2)

Stress Level, Types of Materials \ Types of Re-bars			SD345
Tensile stress	Load combination includes neither impact load nor influence of earthquake.	1) General Materials	180
		2) Materials in water or under groundwater level	160
	3) Basic value of allowable stress. Load combination includes collision load or influence of earthquake.		200
	4) Calculating lap joint length or fixing length of re-bar		200
5) Compressive stress			200

Source: JICA Study Team

Table 3.5.23 Allowable Stress and Design Safety Factor

Type of Allowable Stress and Design Safety Factor	Ordinary	Earthquake
Strip Tensile Stress（N/mm^2）	σa=140.0	σaE=210.0
Bolt Shear Stress（N/mm^2）	τa=200.00	τaE=300.00
Strip Pull Out	Fs≧2.00	FsE≧1.20
Sliding	Fs≧1.50	FsE≧1.20
Overturning	e≦L/6	e≦L/3
Bearing Capacity	Fs≧3.00	FsE≧2.00
Circular Slip	Fs≧1.20	FsE≧1.00

Source: JICA Study Team

Table 3.5.24 Bolt and Strip Dimensions

Item	Item	Code	Unit	Ground (Underwater)
Bolt	Nominal Diameter	d	mm	M12
	Bolt Thread Stress Area	Ae	mm^2	84.3
Number of Bolts per Connection		n	Piece	1
Number of Bolts across the Width		n'	Piece	1
Number of Shear		j	Point	2
Strip	Strip Width	b	mm	80.0
	Strip Thickness	t	mm	4.0
	Corrosion Allowance	Cm	mm	1.0 (1.5)

Source: JICA Study Team

(7) Horizontal Seismic Coefficient

The horizontal seismic coefficient is as follows:

1) Safety of Elements (Internal Stability) (Note: Same for L-type retaining wall)

$k_{h1} = C_z \cdot k_{h0} = 0.18$

k_{h1} : Horizontal seismic coefficient allowing for safety of elements

2) Safety of Mechanically-stabilized Earth Wall（External Stability）

$k_{h2} = C_z \cdot k_{h0} \cdot \upsilon = 0.13$

k_{h2} : Horizontal seismic coefficient allowing for safety of Terre Armee

υ : Correction factor = 0.70

3) Total Stability including Mechanically-stabilized Earth Wall（Stability for Circular Slip）

$k_{h3} = C_z \cdot k_{h0} = 0.18$

k_{h3} : Design horizontal seismic coefficient

k_{h0} : Standard value of design horizontal seismic coefficient = 0.18

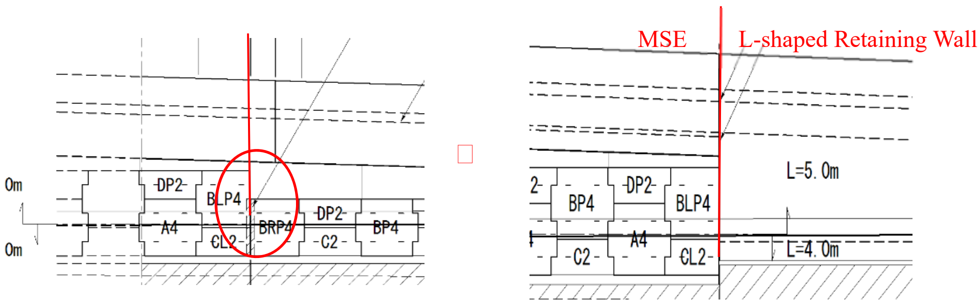
[Ground type : Level 1, Earthquake ground motion: Type III]

Source: Road Earthwork: Retaining Wall Guidelines 2012.

3.5.6.3 Mechanically-stabilized Earth Wall Structure Design

(1) Installation Area for Mechanically-stabilized Earth Wall

A mechanically-stabilized earth wall will be installed in the embankment section behind the abutment of the flyover. However, an L-shaped retaining wall will be installed in the low embankment section, where the number of mechanically-stabilized earth wall panel is one or less.

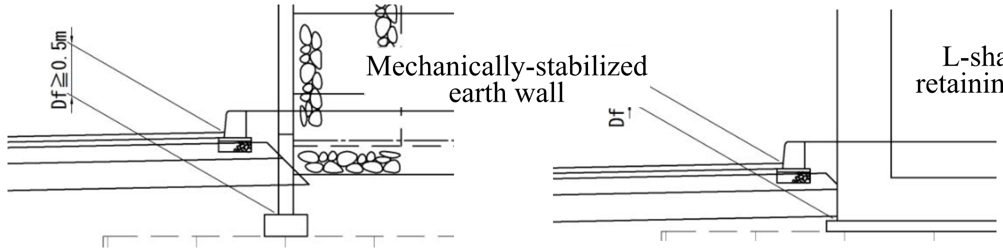


Source: JICA Study Team

Figure 3.5.22 Area for Mechanically-stabilized Earth Wall

(2) Foundation Embedment

Embedded depth of the foundation for both mechanically-stabilized earth wall and L-shaped retaining wall is set as 0.5 m or more.



Source: JICA Study Team

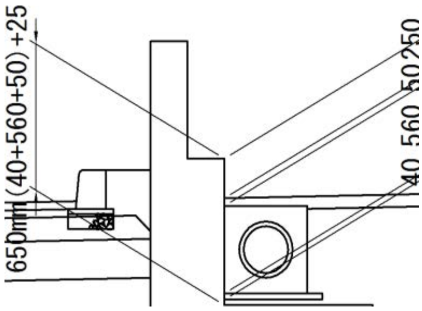
Figure 3.5.23 Depth of Foundation Embedment

(3) Adoption of L-shaped Retaining Wall

The drainage facility in the approach section is installed at a depth of 650mm, which consists of 50mm (pavement) +560mm (the height of drainage) +40mm (mortar layer). The required height of retaining wall is 900mm that the total of 560mm and 250mm of the curb height.

Gravity retaining wall is generally applied if the height of wall is less than 3m. However, L-shaped retaining wall was applied because it is more stable than gravity retaining wall when the 1/3 or 1/2 of the cross section is reduced due to the drainage facility.

The interval of joint was basically determined as 10m similar to the shrinkage joint on the parapet.

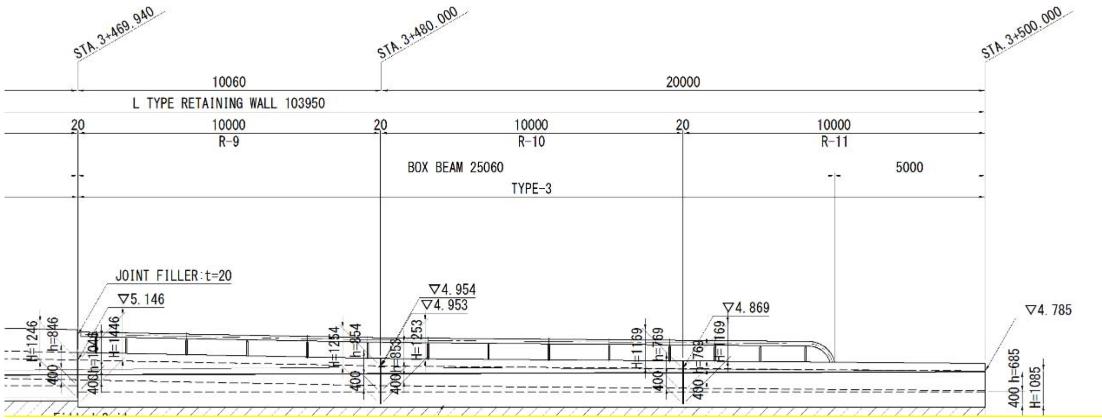


Source: JICA Study Team

Figure 3.5.24 Shape of Drainage Facility and Retaining Wall

(4) Concrete Barrier and Box Beam

A concrete barrier is basically installed in the approach section of the flyover as with the flyover section. However, a steel box beam is installed in the 30 m section before connecting to the at grade section so that enough visibility can be provided to the driver at the merging area.



Source: JICA Study Team

Figure 3.5.25 Side View of the Box Beam

3.5.6.4 Design Calculation

(1) Design Calculation for Mechanically-stabilized Earth Wall

Table 3.5.25 Strip Length and Safety Factor of Pull Out

Stage (i)	Depth zi(m)	Vertical ΔH(m)	Horizontal ΔB(m)	Design Length L(m)	Calculation Length		Safety Factor Pull Out		Evaluation
					Ordinary Lr(m)	Earth-quake LrE(m)	Ordinary Fs	Earth-quake FsE	
1	2.185	0.750	0.750	6.500	5.961	6.297	2.280	1.268	OK
2	2.935	0.750	0.750	6.000	5.476	5.544	2.312	1.395	OK
3	3.685	0.750	0.750	6.000	5.114	5.026	2.574	1.688	OK
4	4.435	0.750	0.750	5.000	4.667	4.440	2.215	1.481	OK
5	5.185	0.750	0.750	5.000	4.325	4.000	2.422	1.731	OK
6	5.935	0.750	0.750	5.000	4.040	4.000	2.570	1.925	OK
7	6.685	0.750	0.750	4.000	4.000	4.000	2.250	1.758	OK

Source: JICA Study Team

Table 3.5.26 Degree of Material Stress

Stage (i)	Ordinary			Earthquake			Evaluation
	σti (N/mm^2)	σoti (N/mm^2)	τoi (N/mm^2)	σtEi (N/mm^2)	σotEi (N/mm^2)	τoEi (N/mm^2)	
1	80.1	74.0	85.5	99.6	91.9	106.3	OK
2	82.4	76.1	88.0	97.0	89.5	103.5	OK
3	84.6	78.1	90.3	95.4	88.1	101.8	OK
4	90.3	83.3	96.4	103.9	95.9	111.0	OK
5	96.3	88.9	102.8	111.6	103.0	119.2	OK
6	101.2	93.5	108.1	118.3	109.2	126.3	OK
7	111.3	102.8	118.9	129.7	119.7	138.5	OK

Source: JICA Study Team

σti , σtEi : Tensile stress of general section of strip (N/mm^2)

σoti, σotEi : Tensile stress of strip in section joined to skin element (N/mm^2)

τoi , τoEi : Bolt shear stress in section joined to skin element (N/mm^2)

Table 3.5.27 Examination of Stability of Mechanically-stabilized Earth Wall (Examination of

External Stability)

( ) Allowable Value

Item		Ordinary		Earthquake	
		Result	Judge	Result	Judge
Sliding	Sliding Safety Factor Fs	7.642 (1.500)	OK	2.339 (1.200)	OK
Overturning	Eccentricity e（m）	-1.024 (0.655)	OK	-0.310 (1.310)	OK
Bearing Capacity under R/E Back Fill	Vertical eaction qs（kN/m^2）	223.045 (223.770)	OK	201.455 (340.000)	OK
Bearing Capacity for Deep Layer Mixing under R/E Back Fill	Verticel eaction qos（kN/m^2）	354.527 (1012.063)	OK	340.247 (1296.009)	OK
Bearing Capacity under Concrete Foundation	Vertical eaction qw（kN/m^2）	208.838 (223.770)	OK	186.596 (340.000)	OK

Source: JICA Study Team

Table 3.5.28 Overall Stability Examination Including Mechanically-stabilized Earth Wall (Concerning Circular Slip)

Classification	Case	Coordinate of Circular Center		Radius R(m)	Fsmin	Fs	Evaluation
		X(m)	Y(m)				
Within Reinforced Area	Ordinary-3	-12.500	18.000	21.303	1.414	1.200	OK
	Earthquake-3	-5.000	19.000	18.923	1.345	1.000	OK
Outside Reinforced Area	Ordinary-15	-6.500	5.500	20.524	3.112	1.200	OK
	Earthquake-12	-7.500	21.500	35.288	1.795	1.000	OK

Source: JICA Study Team

(2) Design Calculation for L-shaped Retaining Wall

1) Overturning

$$d = \frac{\Sigma M_r - \Sigma M_t}{\Sigma V}$$

Where,

d : Distance from toe to the point of force (m)

ΣMr: Moment of resistance around the toes (kN.m)

ΣMt: Overturning moment around the toes (kN.m)

ΣV : Base design surface in all vertical load (kN)

$$e = \frac{B}{2} - d$$

Where,

e : From the bottom of the middle of force eccentricity (m)

B : Width of bottom (m), B = 2.500

$$e_a = {}^{B}\!/_{n}$$

Where,

e_a : Permissible eccentricity (m)

n : Factor of safety

Table 3.5.29 Examination of Overturning

Case	ΣMr (kN.m)	ΣMt (kN.m)	ΣV (kN)	d (m)	e (m)		e_a (m)	Evaluation
Ordinary (Top Loading)	246.537	56.371	208.023	0.914	0.336	≦	0.417	OK
Ordinary (Back)	211.737	56.371	184.823	0.841	0.409	≦	0.417	OK
Earthquake	261.083	108.858	204.587	0.744	0.506	≦	0.833	OK
Collision	211.737	92.502	184.823	0.645	0.605	≦	0.833	OK
Wind	209.637	41.476	184.823	0.910	0.340	≦	0.833	OK

Source: JICA Study Team

1) Sliding

$$F_s = \frac{\Sigma V \cdot \mu + C_B \cdot B'}{\Sigma H}$$

Where,

ΣV: Base design surface in full load (kN)

ΣH: All horizontal loading in the bottom design (kN)

μ: Coefficient of friction between the floor and the bearing, μ=0.600

$C_{B:}$ Bond strength between the floor and the bearing (kN/m^2), C_B = 0.000

B': Loading width (m), B' = B - 2e

B : Width of bottom (m), B = 2.500

e : Eccentricity (m)

Table 3.5.30 Examination of Sliding

Case	Eccentricity e (m)	Loading Width B'(m)
Ordinary (Top Loading)	0.336	1.828
Ordinary (Back)	0.409	1.682
Earthquake	0.506	1.488
Collision	0.605	1.290
Wind	0.340	1.820

Case	Vertical Load ΣV(kN)	Horizontal Load ΣH(kN)	Safety Factor F_s Required Safety Factor F_{sa}	Evaluation
Ordinary (Top Loading)	208.023	49.754	2.509 ≧ 1.500	OK
Ordinary (Back)	184.823	49.754	2.229 ≧ 1.500	OK
Earthquake	204.587	77.166	1.591 ≧ 1.200	OK
Collision	184.823	48.074	2.307 ≧ 1.200	OK
Wind	184.823	36.607	3.029 ≧ 1.200	OK

Source: JICA Study Team

2) Bearing Capacity

Case A: The point of resultant force is in 1/3 width of the central bottom plate.

$$q_1 = \frac{\Sigma V}{B} \cdot \left(1 + \frac{6e}{B}\right)$$

$$q_2 = \frac{\Sigma V}{B} \cdot \left(1 - \frac{6e}{B}\right)$$

Case B: The point of resultant force is in 2/3 width of the central bottom plate.

$$q_1 = \frac{2\Sigma V}{3 \cdot (B/2 - e)}$$

Where,

ΣV : Total load pressure on bottom design (kN)

B : Width of bottom plate (m), B=2.500

e : Eccentricity (m)

Table 3.5.31 Stability Calculation

Load Case	Applied width of subgrade reaction(m)	Shape of subgrade reaction	Subgrade Reaction (kN/m^2) q max q min	Result
Design Load (vertical)	2.500	Trapezium	150.278 ≧ 150.700	OK

Design Load (Back)	2.500	Trapezium	146.566 ≧ 150.700	OK
Earthquake	2.232	Triangle	183.322 ≧ 226.050	OK
Impact	1.935	Triangle	191.032 ≧ 226.050	OK
Wind	2.500	Trapezium	134.282 ≧ 226.050	OK

Source: JICA Study Team

Table 3.5.32 Stability Calculation of Bottom Surface Treated

Load Case	Subgrade Reaction (kN/m^2) q max q min	Result
Design Load (vertical)	1705.782 ≧ 532.023	OK
Design Load (Back)	1687.532 ≧ 508.823	OK
Earthquake	2212.718 ≧ 528.587	OK
Impact	2552.467 ≧ 508.823	OK
Wind	2702.338 ≧ 508.823	OK

Source: JICA Study Team

3.5.7 Road Surface Drainage

(1) Introduction

Rainwater must be removed from the road surface as soon as possible, since surface water has negative impact on road performance.

- Rainwater reduces the effectiveness of the tire grip on the carriageway, which increases the stopping distance.
- The rainwater sprayed by car tires reduces visibility.

In addition, if rainwater penetrates into the pavement structure, the pavement as well as embankment will be damaged.

Rainwater is removed from the road surface through the drainage facilities such as curbs, ditches, culverts, pipes and catch basins, and then discharged to existing rivers or streams.

(2) Rainfall Intensity

"148.9 mm/hr" was applied as the rainfall intensity for the drainage design as shown in Chapter 2.4.

(3) Road Surface Drainage

Rainwater on the road surface is collected by catch basins installed at the edge of the shoulders. Rainwater collected by catch basins is drained to the drainage facilities installed on/under the existing ground such as u-ditch, pipe drainage, and box culvert. Rainwater is finally discharged to the existing rivers or streams through drainage facilities.

The discharge amount on the road surface is calculated according to the catchment area. The type and size of each drainage facility are determined to provide efficient capacity for the discharge amount.

Calculation of discharge amount is shown in Chapter 3.7.

YCDC recommends adopting the open channel with size of 1.5 m x 1.5 m in consideration of easy maintenance including cleaning.

The following types of drainage facilities are applied:

- Standard Section: Open channel with W:1.5 m x H:1.5-1.7 m.
- Narrow Section: Open channel with W:1.0 m x H:1.5 m.

  <= Narrowed to ensure the space for public facility installation. Necessary capacity is ensured.
- Road Crossing Section: Box culvert with W:1.5 m x H:1.5 m.
- Road Crossing Section (Small earth covering section): Box culvert with W:1.0-1.5 m x H:1.0 m.

  <= Necessary capacity is ensured.
- Mechanically-stabilized Earth Wall Section: Pipe culvert with 0.3 m diameter.
- For Road Crossings: Covered U-ditch with W:0.5 m x H:0.5 m.

  Covered U-ditch with W:0.5 m x H:0.85 m.

  Covered U-ditch with W:0.8 m x H:0.8 m.
- Crossing Section: W:0.3 m or 0.2 m diameter.
- End of Flyover: Pipe culvert with 0.3 m diameter.

3.5.8 Design of Ancillary Works

3.5.8.1 Guard Fence

A 1.1 m high guard fence will be installed between the sidewalk and open drainage channel to prevent the pedestrians from falling into the channel.

3.5.8.2 Box Beam

In the mechanically-stabilized earth wall, a steel box beam will be installed in the 30 m section before connecting to the at grade section so that enough visibility can be provided to the driver at the merging area.

3.5.8.3 Boundary Fence

Through the discussion with MOC and YCDC, it was determined that 1.8 m high boundary wall will be installed to prevent the people from entering under the flyover.

3.5.8.4 Traffic Signs and Road Markings

The following traffic signs and road markings will be provided:

(1) Traffic Signs

1) Regulation Sign

Regulation signs of "Maximum speed 40 km/h" and "Stop" will be installed on Thanlyin Chin Kat Road and the road crossings.

2) Warning Sign

Warning signs of "Signal", "Slow down", and "School" will be installed on Thanlyin Chin Kat Road and the road crossings.

3) Informatory Sign

Informatory signs showing the direction to each area will be installed on Thanlyin Chin Kat Road and the road crossings.

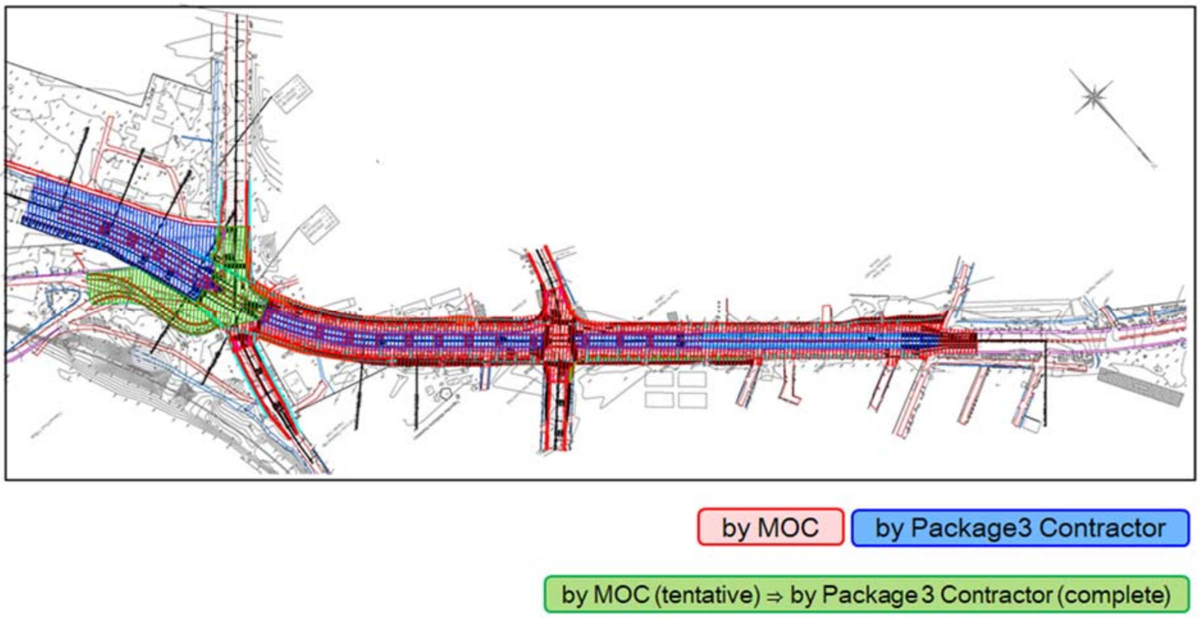
(2) Road Markings

Road markings of "Toll Ahead" and "Maximum speed 60 km/h" will be painted on the road surface of the flyover and approach road.

3.5.9 Demarcation between Yen Loan and Myanmar for Package 3

3.5.9.1 Outline

Construction of flyover and widening of Thanlyin Chin Kat Road were originally done by Package 3 contractor. However, in the discussion with MOC on 22th June 2017, it was determined that widening of Thanlyin Chin Kat Road shall be directly completed by MOC prior to the commencement of flyover construction by the Package 3 contractor as shown in Figure 3.5.29. In this section, design modification due to the demarcation change is introduced.



Source: JICA Study Team

Figure 3.5.26 Demarcation between Package 3 Contractor and MOC for Package 3

Detailed demarcation between Package 3 Contractor and MOC is shown in Table 3.5.33.

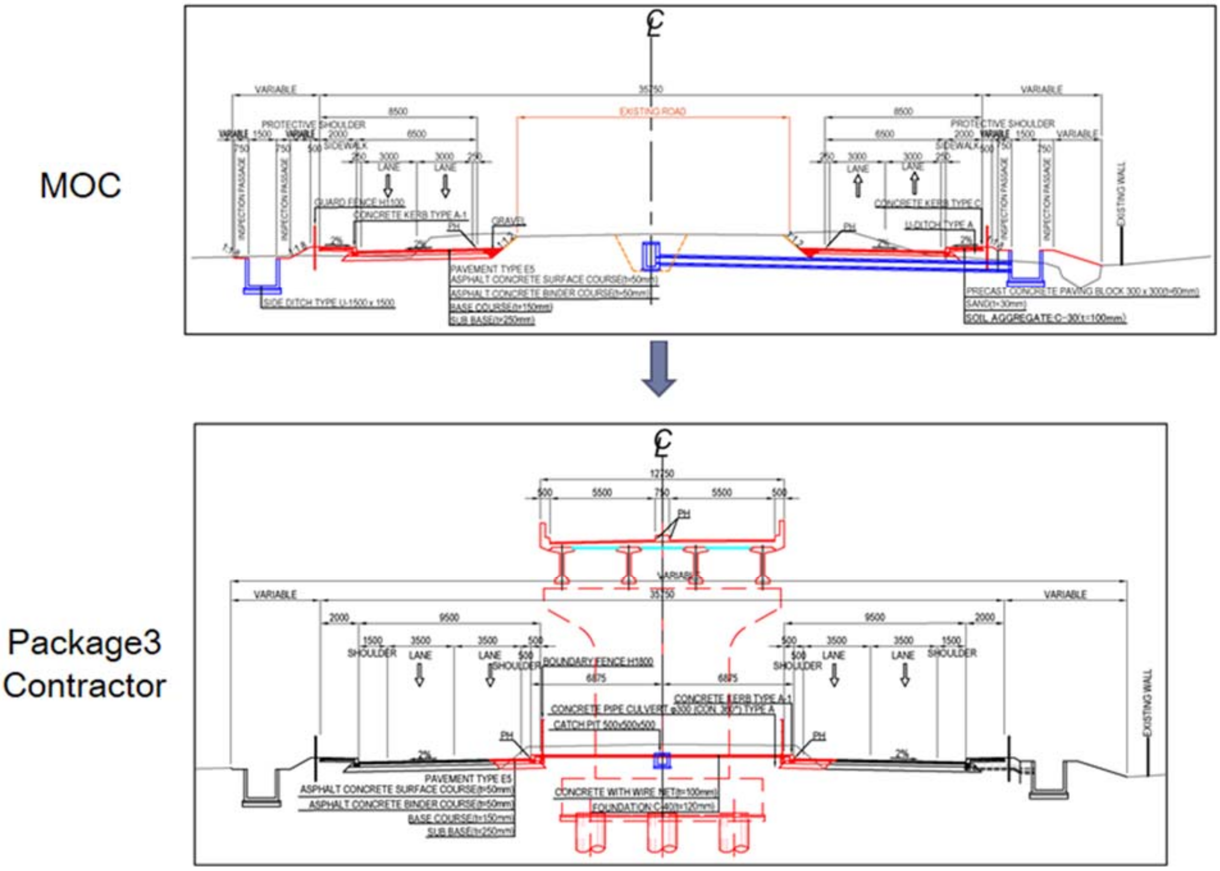
Table 3.5.33 Detailed Demarcation between Package 3 Contractor and MOC

Work Item	MOC	Package 3 Contractor
Site Clearing	➢ Outside construction yard for flyover	➢ Within construction yard for flyover
Road Works	➢ Earth Work & Pavement (6.5m) ➢ Sidewalk ➢ Intersection (Tentative) ➢ Tentative approach road to/from existing Thanlyin Bridge	➢ Earth Work & Pavement (1.5m or 2.5m) within construction yard ➢ Intersection (Complete) ➢ Approach road (Complete) to/from existing Thanlyin Bridge (Package 2)
Drainage Works	➢ Side ditches & Box culverts ➢ Concrete pipe culvert ➢ Catch pits on Thanlyin Chin Kat Rd.	➢ Bridge drainage
Miscellaneous Works	➢ Road lighting on Thanlyin Chin Kat Rd. ➢ Concrete kerb outside construction yard for flyover ➢ Road markings (Tentative) ➢ Regulation / Warning sings ➢ Traffic signals on I/S (Tentative) ➢ Guard fence on sidewalk	➢ Road lighting on Flyover ➢ Concrete kerb within construction yard for flyover ➢ Informatory sign board ➢ Road markings (Complete) ➢ Traffic signals on I/S (Complete) ➢ Boundary fence under flyover
Flyover Works	N/A	➢ Flyover Bridge ➢ Approach Road

Source: JICA Study Team

3.5.9.2 Typical Cross Section

Typical cross section for each construction stage is shown in Figure 3.5.27.



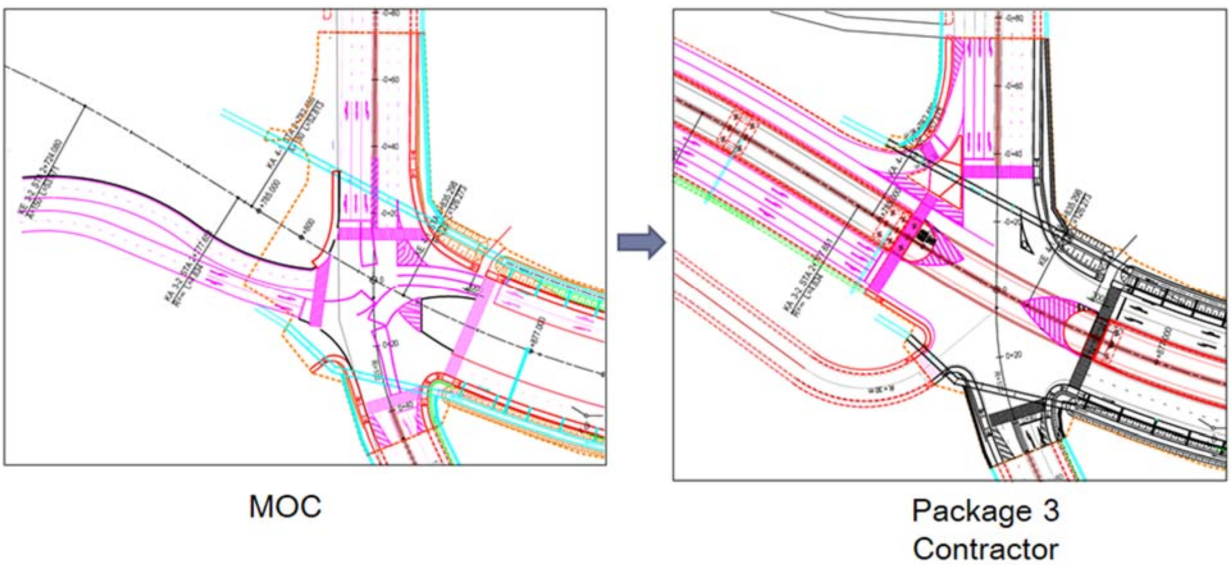
Source: JICA Study Team

Figure 3.5.27 Typical Cross Section

3.5.9.3 Intersection Design

(1) Shukhinthar Intersection

The main traffic flow generates to/from the existing Thanlyin Bridge until Bago River Bridge and flyover is completed. Therefore Shukinthar intersection and its approach road shall be tentatively adjusted for smooth traffic flow by MOC and then shall be completed after the completion of flyover by Package 3 contractor as shown in Figure 3.5.28.

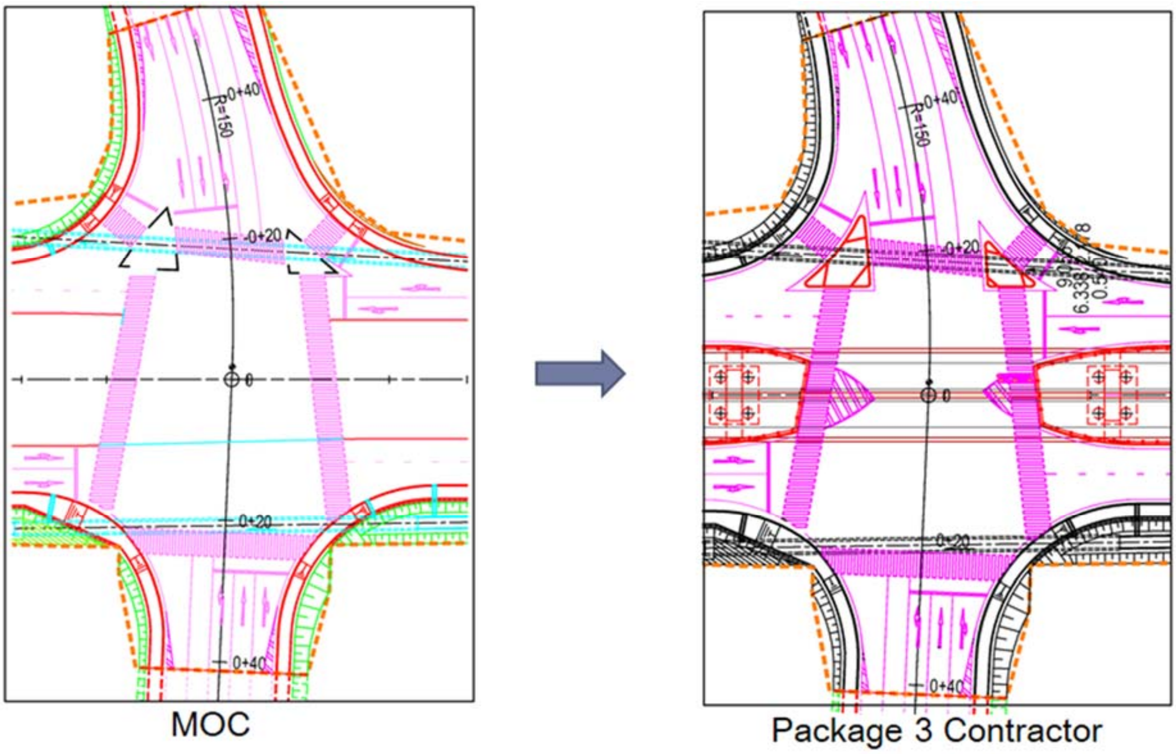


Source: JICA Study Team

Figure 3.5.28 Design Modification for Shukhintar Intersection

(2) Yadanar Intersection

At Yadanar intersection, the works except for traffic islands, road markings and medians under girders can be completed during widening of Thanlyin Chin Kat Road by MOC, and then remain works for completion shall be done by Package 3 contractor as shown in Figure 3.5.29.

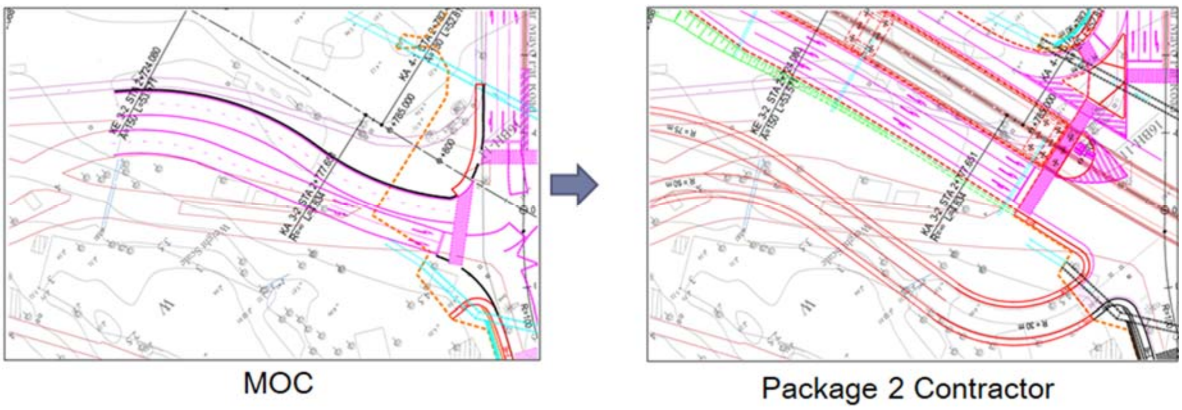


Source: JICA Study Team

Figure 3.5.29 Design Modification for Yadanar Intersection

3.5.9.4 Approach Road to / from the Existing Thanlyin Bridge

Approach road to/from the existing Thanlyin Bridge shall be adjusted in accordance with the tentative Shukinthar intersection by MOC and shall be completed by Package 2 Contractor as shown in Figure 3.5.30.

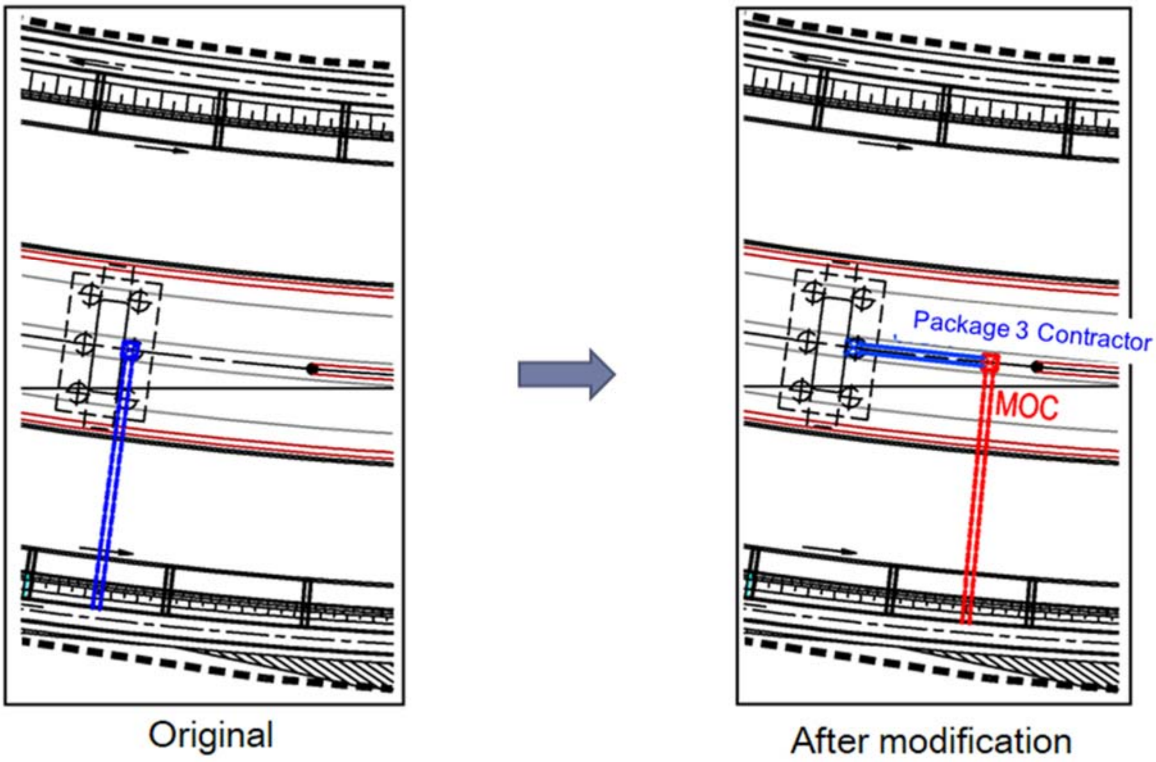


Source: JICA Study Team

Figure 3.5.30 Design Modification for Approach Road to/from the Existing Thanlyin Bridge

3.5.9.5 Bridge Drainage

The drainage system of the bridge should be changed from the initial plan in consideration of the construction order. Changes are shown in Figure 3.5.35.



Source: JICA Study Team

Figure 3.5.34 Design Modification for Bridge Drainage (Flyover).

3.5.9.6 Pavement Design for Tentative Approach Road to the Existing Thanlyin Bridge

As for the pavement design, "pavement of detour path" of Japanese standard method is used. From the design calculation, the total pavement thickness: t = 40 cm is adopted. This is the same as the pavement on the Thanlyin side on-ramp (Type E 6, total thickness 40 cm).