8. STUDY ON LAND SLIDE AREA

In the hilly area of southern Malakal Island Road, there are two locations which have arc cracks and sea side road surface depression. As the beneath of road, pressurized sewer pipe, which conveys waste water from whole Koror area to the sewage works, is embedded, it is required to study the areas and provide countermeasures, if necessary.

<u>Present conditions</u>

Arc cracks on the road surface are both located in sea side, and form a top of horseshoe line as shown in two photos below. The mass inside horseshoe line is depressed from the adjacent road surface and seems moved to sea side along with the installed guardrail.

Location : STA. 0+395~0+415
Length : Approximately 20 m
Width : Approximately 2 m
Depression : Some 10 cm or less

Location : STA. 0+458~0+468
Length : Approximately 20 m
Width : Approximately 2 m
Depression : Some 10 cm or less

There are big scale earth works site of sewage works expansion project at hill side of Malakal Island Road, adjacent to these land slide area. As shown in the following two photos, fresh cut slopes were being left with vinyl sheet covers. This expansion project was implemented by CIP, and the works were carried out by local general contractor, BlackMicro.

The study was carried out based on the following two assumptions:

Assumption-1: Land slide and depression have been occurred locally at sea side of road.

Assumption-2: A big scale of land slide is there which starts from hill side and includes whole Malakal Island Road. This assumption is not negligible since fresh hill side cut slopes are exposed to precipitation rather long period.

In order to confirm which of the above assumptions represent the real situation, the further investigation schedule for detailed design stage was established as shown in Table 9-1, with the consultation of geotechnical expert. However it was judged at the moment that Assumption-2 would not be the case due to the following reasons:

1. In case a big scale of land slide has been occurred, which involves two land slide traces on the road surface in 30 m distance, there should be lateral cracks on the road with depressions abutting to these cracks. However such cracks or depressions were not found at the site.

2. There is an old concrete observation post on the hill side of Malakal island Road. It was not observed any trace of move of the observation post, or any cracks of the building walls.

3. No trace of land slide cracks on cut slopes of sewage works expansion project site.

However, since the sewage works expansion project seems not to provide any special countermeasures for land slide or slope collapse, the possibility of big scale land slide collapse would be still remained when the project site receives heavy rain in future. It is understood that the said foreseen land slide, originated from sewage works expansion project, is beyond this project scope, and should be prevented by sewage works expansion project providing appropriate countermeasures.

Assumption-1 would be supported by the following explanation. It is understood that Malakal Island Road in this section was constructed along steep hill slope, by cutting hill side and filling sea side. Therefore it may cause a sliding face between original hill ground and fill material.

It was observed that repair works were carried out in two locations several times before. Although the sliding behavior or depression is in progress, the road is passable with the current depressed level at the moment. However, there is a possibility to cause a serious land slide collapse when underground water from original hill ground, increased by heavy rain, flow through sliding face between original ground and fill material. In order to eliminate this possibility, it was proposed to provide countermeasures by removing the existing fill material by bench cutting, refilling with selected material by every 20 cm compaction utilizing geotextile reinforced embankments method, protecting sea side fill slope by stone masonry, and providing asphalt pavement.

Investigation schedule as shown in Table 9-1 was planned to finalize the countermeasures to cope with the observed sliding traces on the existing road surface. To ensure the rightness of said assumption and safety by the proposed countermeasures, it is proposed to carry out the site investigation by the Geotechnical Expert at the detailed design stage as the first site investigation. When the first site investigation concluded to necessitate the further detailed investigation, then the second site investigation would be carried out including boring works as noted in Table 8-1.

Table 8-1 Proposed Further Investigation Schedule

	Investigation Item	Period	Purpose	Output
First Site Investigation	Site investigation by the Geotechnical Expert	Palau 7 days Japan 10 days Total 17 days	- Study on basic design of countermeasures - Study on terrain condition and geotechnical condition of sites, required for planning of the further detailed investigations	Identify the mechanism of giving sliding trace on the road surface
Second Site Investigation				
- for fill material sliding	Boring works A: 10 m×2 locations B: 15 m×1 location Total 35 m	One week	A: At the shoulder edge of probable sliding area B: At the hill side cut slope to see the stableness of slope	Confirmation of sliding face between original hill ground and fill material
- for big scale land slide	Boring works 15 m×5 locations Total 75 m Geodetic Survey - Profile 60 m - Cross Section 60 m	Three weeks	- Three locations on centerline of land slide area - 2 locations at shoulder edge of existing road	Plan drawing 60 m×80 m=4,800 m^2
Detailed design				
- for fill material sliding	Detailed design of countermeasures	Reconstruction of fill sections Design: a week	- Geotextile reinforced embankments	
- for big scale land slide	Conduct Stability Analysis	Countermeasure against big scale land slide: One month		Provide design and specifications of protection works by earth anchor or large scale steel piling

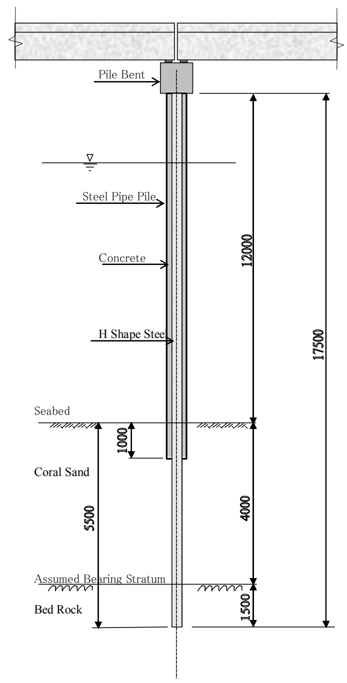
9. STRUCTURAL ANALYSIS OF EXISTING PILES OF PIERS,
(Stability & Sectional Force Analysis of Piles of Piers Minato Bashi Bridge**)**

9.1 CONDITIONS & METHOD OF ANALYSIS

- Dimensions of piles, pile bent, and other bridge members were assumed based on the collected data & site inspection.
- Embedded depth of piles into the seabed & bearing stratum, and soil properties were assumed based on the collected construction drawings and visual inspection of seabed.
- For loading condition, seismic forces in longitudinal & transverse directions were applied.
- The pile consists of H shape beam, concrete, and steel pipe pile. It was assumed that those 3 components share the bending moment with the proportion of their stiffness, namely, the young modulus multiplied by moment of inertia.

The present condition of piles was regarded as the projecting pile, and the Japanese Specification for Highway Bridges was applied as the standard.

The general features and dimensions of the pile applied for the analysis was shown in the right figure.



9.2 SUMMARY OF WORKING LOADS AT THE BOTTOM OF PILE BENT

Summary of working loads under seismic case was as shown in the following table. The value of seismic coefficient was defined as 0.06.

Table 9-1 Summary of Working Loads

Working Load	Longitudinal	Transverse
V(kN)	2,404.0	2,404.0
H(kN)	144.0	144.0
M(kN·m)	149.0	157.0

9.3 RESULTS OF ANALYSIS

9.3.1 Stability Analysis

- For the stability analysis, only H shape beam was considered for the calculation of bearing capacity. This is because only this component of pile was assumed to be embedded into the bearing stratum.

- The weight of the concrete and steel pipe pile covering the H shape beam was regarded as the dead loads. The weight of concrete was 24kN, and that of steel pipe pile was 14kN per 1 pile, respectively.

- The results of stability analysis were summarized as below. In conclusion, the stability of existing pile foundation was satisfied.

Tbale 9-2 Results of Stability Analysis

	Longitudinal	Transverse
Compressive Force, PNmax (kN)	839 < 1,036	1,022 < 1,036
Tensile Force, PNmin (kN)	839 < -201	656 < -201
Evaluation	OK	OK

9.3.2 Sectional Force Analysis

Analysis of Maximum Moment

In the design calculation, maximum moment was induced at about 1m from the tip of pile. At that point, all components of the pile, namely, H shape beam, concrete, and steel pipe pile were sound and able to resist the maximum moment. The value of maximum moment was divided to those three components with the proportion of their stiffness, and section analysis for each component was conducted separately. Because of the proportion of stiffness, it was assumed that the steel pipe pile charges 62% of moment. Results of the analysis were summarized in the following table. It was certified that whole of the induced compressive & tensile stresses in each component were under the allowable stresses.

Table 9-3 Results of Sectional Force Analysis of Piles (for Maximum Moment)

<table>
<tr><td colspan="2">Component</td><td colspan="2">H shape beam</td><td colspan="2">Concrete</td><td colspan="2">Steel Pipe Pile</td></tr>
<tr><td colspan="2">Dimension</td><td colspan="2">W14×119</td><td colspan="2">5000psi, #11bar×4</td><td colspan="2">Dia. 28 in. t=1/4 in.</td></tr>
<tr><td colspan="2">Share of Moment</td><td colspan="2">31%</td><td colspan="2">7%</td><td colspan="2">62%</td></tr>
<tr><td rowspan="3">Stress</td><td></td><td>σt</td><td>σta</td><td>σc
(σt)</td><td>σca
(σta)</td><td>σt</td><td>σta</td></tr>
<tr><td>Compressive (N/mm²)
(Ratio)</td><td colspan="2">-</td><td colspan="2">2.0 < 15.6
(0.13)</td><td colspan="2">-</td></tr>
<tr><td>Tensile (N/mm²)
(Ratio)</td><td colspan="2">97.7 < 210.0
(0.46)</td><td colspan="2">46.6 < 270.0
(0.17)</td><td colspan="2">202.3 < 210.0
(0.96)</td></tr>
<tr><td colspan="2">Evaluation</td><td colspan="2">OK</td><td colspan="2">OK</td><td colspan="2">OK</td></tr>
</table>

Analysis of Pile at Corroded Portion

- The bending moment at 4.5 m under the bottom of pile bent was estimated. This location was regarded as the portion where the steel pipe piles were corroded and not effective to resist the induced moment. Accordingly, only two components of piles, namely, H shape beam and concrete were applied for the sectional analysis. Results of the analysis were summarized as below. It was clarified that whole of the induced compressive & tensile stresses in each component were under the allowable stresses.

- Consequently, it was concluded that the sectional capacity of piles at corroded portion was satisfied.

Table 9-4 Results of Sectional Force Analysis of Piles (at Corroded Portion)

<table>
<tr><td colspan="2">Component</td><td>H shape beam</td><td>Concrete</td><td>Steel Pipe Pile</td></tr>
<tr><td colspan="2">Dimension</td><td>W14×119</td><td>5000psi, #11bar×4</td><td rowspan="6"></td></tr>
<tr><td colspan="2">Share of Moment</td><td>31%</td><td>20%</td></tr>
<tr><td rowspan="3">Stress</td><td></td><td>σt σta</td><td>σc σca
(σt) (σta)</td></tr>
<tr><td>Compressive (N/mm^2)
(Ratio)</td><td>-</td><td>1.9 < 15.6
(0.12)</td></tr>
<tr><td>Tensile (N/mm^2)
(Ratio)</td><td>186.2 < 210.0
(0.89)</td><td>22.6 < 270.0
(0.08)</td></tr>
<tr><td colspan="2">Evaluation</td><td>OK</td><td>OK</td></tr>
</table>

10 PAVEMENT DESIGN

10. 1 DESIGN CONDITIONS

10.1.1 General

Pavement design was conducted by AASHTO methods and checked by Japanese standard. AASHTO design methods for asphalt Pavement are done by the procedures shown in Fig 10-1.

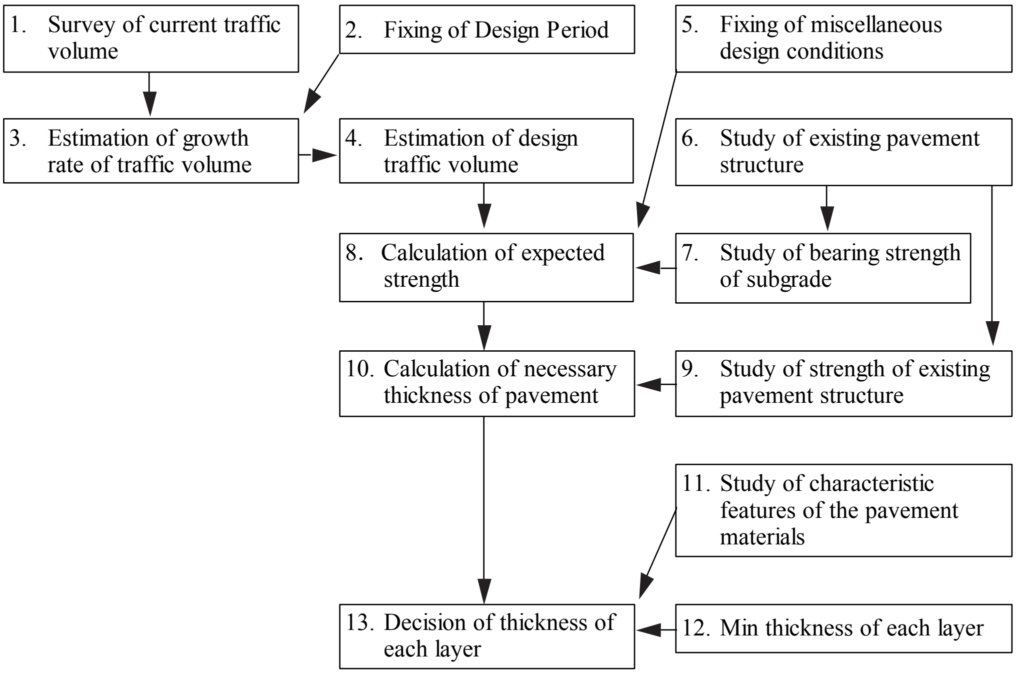


Fig 10-1 Asphalt pavement design procedure by AASHTO

Necessary strength of the pavement, called Structural Number（SN）, is calculated by the following formula.

Asphalt Pavement

$$\log_{10} W_{18} = Z_R * S_0 + 9.36*\log_{10}(SN+1) - 0.20 + \frac{\log_{10}\frac{\Delta PSI}{(4.2-1.5)}}{0.4+\frac{1094}{(SN+1)^{5.19}}} + 2.32*\log_{10} M_R - 8.07$$

Cement Concrete Pavement

$$\log_{10} W_{18} = Z_R * S_0 + 7.35*\log_{10}(D+1) - 0.06 + \frac{\log_{10}\frac{\Delta PSI}{(4.5-1.5)}}{1+\frac{16240000}{(D+1)^{8.46}}} +$$

$$(4.22-0.32*p1)*\log_{10}\left[\frac{S'c*Cd*(D^{0.75}-1.132)}{215.63*J[D^{0.75}-\frac{18.42}{(Ec/k)^{0.25}}}\right]$$

This formula is the statistical summary of the large scale of experiment of 10 years by AASHTO where, W_{18} : predicted number of 18 kip (=8.2 t) equivalent single axle load application of during the design period.

Z_R : Standard normal deviate=-1.037（correspond to Reliability probability R=85%. 75~95% is for urban roads）

S_0 : Combined standard error of the traffic prediction and performance predictions=0.45（normally 0.45 is applied for flexible pavement）

D : Thickness of Concrete slab

M_R : Resilient Modulus of Subgrade=CBR×(750~3000)＝CBR×1500

ΔPSI : Difference between the initial design seviceabilty index P_0 and the design terminal serviceability index, P_t
(ex.: P_0=4.2, terminal value: P_t=2.5, ΔPSI=P_0— P_t =1.7)

S'_c : Modulus of rupture (psi) for protland cement concrete, 578 psi (Refer: Design Guide page-I-5)

J : Load transfer coeeficient (assumed as 3.0 by Table 2.6)

C_d : Drainage coefficient (assumed as 1.0 by Table 2.5)

E_c : Modulus of Elasticity (PSI) for concrete=57,000× (compressive strength)$^{0.5}$ (P-II-16)

k : Modulus of subfrade reaction (PSI) assumed as 300 (P-II-39 Fig. 3.3)

It is difficult to calculate SN directly from the above formula. And "Nomogarm" is shown for the calculation, howeverthe usage of the nomogarm is not so practical, because it will make a eror on lining the line.

Therefore iteration calculation of design was conducted by using Goal Seek function of Excel program.

10.1.2 Performance Period (Design Period)

Table 10-1 Design Period by AASHTO

Road Conditions	Periods (Year)
Urban Rd. Big Traffic	30~50
Local Rd. Big Traffic	20~50
Pavement, small traffic	15~25
Gravel Rd. small traffic	10~20

Performance periods at design for the flexible pavement are variable from 10 years to 30 years according to the rank of the road. If the period is small, the initial cost becomes smaller, while maintenance cost and the frequency for the disturbance for traffic due to maintenance works will increase. Therefore longer period is preferable for the important trunk road. Design with long performance period has a weak point. It is difficult to appropriate structural design for the changing of the social conditions of the meantime and the increasing of traffic and heavy vehicles accompany to them.

The increasing value of SN for the design period of 20 years compared to 10 years is around 10~15%. Therefore stage construction methods are also popular. It means the initial design period is for 10 years and overlay work will be introduced after 5 years or so based on the observation results of the increasing of the actual traffic condition.

In this study, the design period is fixed to be 10 years.

10.1.3 ΔPSI : Difference between initial and terminal serviceability

One of the characteristic features of the AASHTO experiment is introduction of PSI (Performance Service Index), and the value shown in Table 10-2 are used in general.

Table 10-2 Value of PSI

	PSI	
Maximum	5.0	
Initial index, P_0	4.5 4.2	(Rigid Pavement) (Flexible Pavement)
Terminal index, P_t	3.0 2.5 2.0	It will be selected from three index shown in left as the execution time for overlay works (Refer to Table 3-3)
Minimum	0	

Formula 2-2-1 shows that AASHTO had conducted their experiment for the road with P_0=4.2 up to P_t=1.5. Acceptable values of terminal service index P_t are summarized as sown in Table 10-3.

Recently 4.5 is adopted as the value for P_0 according to the development of pavement technology, but 4.2 is used in this project based on the actual site and working conditions.

Generally acceptable P_t is summarized as follows;

Table 10-3 Terminal Performance Service Index - PSI

Terminal Performance Service Index P_t	Ratio of the user who recognized that the road is unacceptable to drive.
3.0	12%
2.5	55%
2.0	85%

P_t=2.5 will be used as the Terminal Performance Service Index in this study.

10.1.4 Design Traffic Volume

Accumulated number of 18kips trafafic volume is estimated as 4.4 million from the traffic survey data and assumed traffic growth rate, which shows the number of heavy veicles as 800 per day for the first year and 1,600 vehicles per day for the 10th year.

10.1.5 Evaluation of bearing strength of the subgrade (design CBR)

Design CBR is assumed to be 10 from the survey result of existing pavement structure.

10.1.6 Assumption of other conditions

Pavement design methods by AASHTO will request to decide the environmental, drain and reliability conditions of the design roads.

This is a suggestion for the design engineer to take into consideration the differences of the location and construction from experiment site by AASHTO. However the actual aplication methods are not suggested. The assumption in this study are shown in the following.

10.1.7 Environmental Conditions

Annual precipitation is 3,200 mm, and average temperature is 25~30 degrees and the weather conditions of the site shall be considered to be moderate.

10.1.8 Drain Conditions

The pavement deterioration will be accerarated if there is a drainage problem in the pavement structures and base course. AASHTO suugests to modify the layer coeeficient of base course by using layer drainage coefficient "m". In this project, the ratio of sorked conditions is assumed to be 25%, and drainage condtion is in good, then "m" becomes 1.00 and no special consiration will be required.

The troubles of the drain capacity of pavement structures, especially for base course, will affect the durability of the pavement. AASHTO suggests the drain coefficient "m" to adjust the layer coefficients of base course without stabilization as shown in Table

Table 10-5 Drain Coefficient

	Time ratio of Pavement in soaked conditions			
Drain	Less than 1%	1~5%	5~25%	25% or more
Very good	1.40~1.35	1.35~1.30	1.30~1.20	1.20
Good	1.35~1.25	1.25~1.15	1.15~1.00	1.00
Not good	1.25~1.15	1.15~1.05	1.00~0.80	0.80
Bad	1.15~1.05	1.05~0.80	0.80~0.60	0.60
Terrible	1.05~0.95	0.95~0.75	0.75~0.40	0.40

10.1.9 Z_R: Standard normal deviate

This is a probability of the retain of the pavement for the unexpected performance, load, damages including environmental or drain conditions during the design period. There is a relationship between reliability coefficient Z_R to be used in design formula and reliability probability R (%) as shown in Table

Table 10-6 Relation between reliability coefficient Z_R and reliability probability R (%)

R	99.9	99	98	97	96	95	93	90	85	80	70	60	50
ZR	-3.090	-2.327	-2.054	-1.881	-1.751	-1.645	-1.476	-1.282	-1.037	-0.841	-0.524	-0.253	-0.000

Recommended value of reliability probability R are shown in Table10-7

Table 10-7 Recommended reliability probability R

	Interstate・Highway	Trunk Road	Collector Road	Local Road
Urban area	85~99.9	80~99	80~95	50~80
Local area	80~99.9	75~95	75~95	50~80

In this study, R＝85% and ZR=-1.037 are used.

10.1.10 S_0: Overall Standard Deviation

Chance factor of each of above coefficients are assumed to be on regular distribution, and overall standard deviation is shown in Table

In this study, S_0=0.45 will be used for Asphalt Concrete pavement and 0.35 for Portland cement concrete pavement.

Table 10-8 Overall standard deviation, S_0

	Asphalt Pavement	Concrete Pavement
In consideration of future traffic disperse	0.44	0.34
No consideration of future traffic disperse	0.49	0.39
Including allowance for performance	0.45	0.35

10.1.11 Material characteristics

The most remaralkable characteristics of AASHTO pavement design method is in using resilient modulus of materials for subgrade, base course and surface. However there is a problem for applying similar one to elastic modulus to non elastic materials which are affected by loading time and temperature. It is important to adopt reasonable values for the calculations.

Resilient Coefficient, "M_R" and Layer Coefficient, "a" for subgrade, base course and surface are assumed as shown in Table 10-9.

Table 10-9 Resilient Coefficient and Layer Coefficient

	Subgrade	Subbase Course	Base Course	Surface (Asphalt)	Surface (Cement)
Specifications		Modified CBR=30% or more PI=6.0% or less	Modified CBR=80% or more PI=4.0% or less	Hot mixture	280 kg/cm^2 4,000 psi
Resilient Modules, M_R	CBR×1,500	1,500 (CBR×500)	2,8000 (CBR×350)	300,000	57,000×FC$^{0.5}$ 3.6×10^6
Layer coefficient		A_3 0.08	A_2 0.14	A_1 0.42	

10.1.12 Summary of Assumption for the design calculation

The following table shows the items to assume the value

Table 10-10 Assumed value for design calculation

	The value to be used	Clause
Design period	10 years	(2)
Current traffic Growth rate Predicted Volume	Around 4.4million for 10 years (5 to 7 million axle)	(3)
Existing Sugrade CBR	10	(4)
Miscellanious conditions	Z_R : Standard normal deviate =－1.037 So : Combined standard error＝0.45(As)・0.35(C) MR : Resilient Modulus of Subgrade＝15,000psi ΔPSI : Difference between Po and Pt＝1.7	(5) (6)
Matrial Characteristics Conversion ratio & Resilient Modulus	Subbase 0.08 21,000psi Granular base 0.14 28,000psi Stablized base 0.35 280,000psi Asphalt Concrete 0.42 400,000psi Cement Concrete 3,600,000 psi	(7)

10. 2 CALCULATION OF REQUIRED STRUCTURE

Calculations are shown bellow;

Table 10-11 Calculation of Required SN

Asphalt				
Input data	CBR	10	10	10
Design period	Year	10	10	10
To value	Axle No	5,000,000	6,000,000	7000000
	ZR	-1.037	-1.037	-1.037
	S0	0.45	0.45	0.45
	dPSI	1.7	1.7	1.7
Resilient Modulus	Mr	15,000	15,000	15,000
GoalSeekSet cell		5,000,000	6,000,000	7,000,000
Changing cell	SN	3.3	3.4	3.5
	D(inch)	7.9	8.2	8.4
	Converted Ta	20.2	20.8	21.3
Surface course		10.0	10.0	10.0
Base course		30.5	32.3	33.9

=10^(Zr×So+9.36×LOG(Sn+1)-0.2
+(LOG(dPSI/2.7)/(0.4+1094/(Sn+1)^5.19))+2.32×LOG(Mr)-8.07)

Table 10-12 Pavement Thichness Calculation **Concrete**

Input data	CBR	10	10	10		
Design period	Year	10	10	10		
To value	Axle No	5,000,000	6,000,000	7,000,000		
	ZR	-1.037	-1.037	-1.037		
	S0	0.35	0.35	0.35		
	dPSI	1.70	1.70	1.70		
Resilient Modulus	k	300	300	300		
	S'c	578	578	578		
	j	3.0	3.0	3.0		
	Cd	1.00	1.00	1.00	Comp Str	
	Ec	3,600,000	3,600,000	3,600,000	280	4060
GoalSeekSet cell		5,000,000	6,000,000	7,000,000	kg/cm2	psi
Changing cell	D	8.7	9.0	9.3		
	Converted to CM	22.2	22.9	23.5		

=10^(Zr×So+7.35×LOG(D+1)-0.06+(LOG(k/3)/(1+1.624/(D+1)^8.46))
+(4.22-0.32×2.5) ×LOG((S'c×Cd× (D^0.75-1.132))/(215.63×J× ((D^0.75)-(18.42/((Ec/k)^0.25)))))

10.3 THICKNESS OF BASE COUSE

Minimum requirement for the base course of asphalt pavement is defined to be 10cm, and it will be decided as the multiple layer on the subgrade considering the balance with surface asphalt pavement. Coral aggregate is recommended to be utilized in this project. The modified CBR is estimated to be similar to those of lime stabilization. 30 cm of base course was designed to place into 2 layers with 15 cm each.

In case of Cement Concrete pavement (Road B category), the base couse will be desgned to ensure the bearing coefficient of more than 20 kg/cm^2 by plate loading test. This will be determined from the ratio with the bearing coefficient with subgrade. However it is decided by CBR methods in this project, because of no actual test on the site. The Japanese desgn manual defines 15 cm thickness base couse of modified CBR is more than 80 on the subgrade of CBR 10.

10.4 CEMENT CONCRETE PAVEMENT

Concrete pavement is expected to place not separately with shoulders. In this case the construction width becomes 4.8 meters.

Longitudinal joint is dummy joint with tie-bar. The interval is every one meter. Cross-sectional expansion joint is expected to set every 300 to 350m usually. It means that one joint is necessary for each causeway. However it is omitted considering the small difference of temperature of the site. Cross-sectional shrinkage joint is dummy joint with slip bar and placed every 8 m interval. Wet formed joints shall be every 32 meters and other one shall be cutter joints. Necessary number of slip-bar is 12 within the 4.8 m width. Steel net and reinforcing bar shall be placed. Steel net weight of 6mm deformed bar is 3 kg/m2. It shall be set on the position of one third from the surface. Reinforcing bar is composed of 3 numbers of D13.

Cement concrete bending strength shall be 40 to 45kg/cm2, and the slump shall be 5 to 8 cm.

10.5 CONCLUSION

Pavement structures are decided as bellow considering the workability.

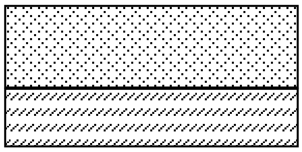
Table 10-13 Thickness of Pavement
(Design period 10years, CBR=10, 18kips, 5~7 million)

	Surface Course	Granular Base C.	Total Thickness
Alternative 1	As. 5 cm+5 cm	30 cm	40 cm
Alternative 2	Con. 25 cm	15 cm	40 cm

Asphalt Pave.

Subgrade CBR>10

Concrete Pave.



Subgrade CBR>10

10.6 ROAD CLASSIFICATION BY JAPANESE ROAD STANDARD

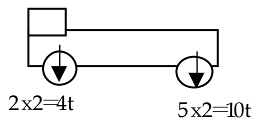
Japanese design manual suugests dividing the road categories to 5 ranks as shown in right by the estimated heavy vehicles numbers of 5 years later. The conversion from Japanese standard heavy vehicles (5 ton wheel load) to 18kips (=8.2 ton) axle load by AASHTO is shown as follows;

Table 10-4 Road Rank by Japanese Manuals

Rank	Heavy Vehicles(per day per one direction)
L	Bellow 100
A	100 to 250
B	250 to 1,000
C	1,000 to 3,000
D	More than 3,000

Assumption of the wheel load arrangement of the Japanese standard heavy vehicles are shown as right figure (Net weight is 14 tons).

Converted number of Japanese standard heavy vehicles to ESAL is (Front wheel $4/8.2)^4$+(Rear wheel $10/8.2)^4$=0.0566+2.2118=2.27 ---around 2.3



The number of heavy vehicles becomes 800/2.3 to 1600/2.23=400 to 700. Therefore, the road categories based on the Japanese standard becomes B rank.

11 MALAKAL ISLAND ROAD DRAINAGE D

The drainage design along Malakal Island Road was carried out based on the design runoff from the adjacent areas. Catchment areas were estimated as listed in Table 11-1, based on the topographic map of scale in 1:25,000, prepared by Department of the Interior, USA, 1983 (see a map right).

Table 11-1 Estimated Catchment Area

Catchment No.	Catchment Area (m^2)
A	15,700
B	55,100
C	42,000
D	72,400
E	3,800
F	75,800

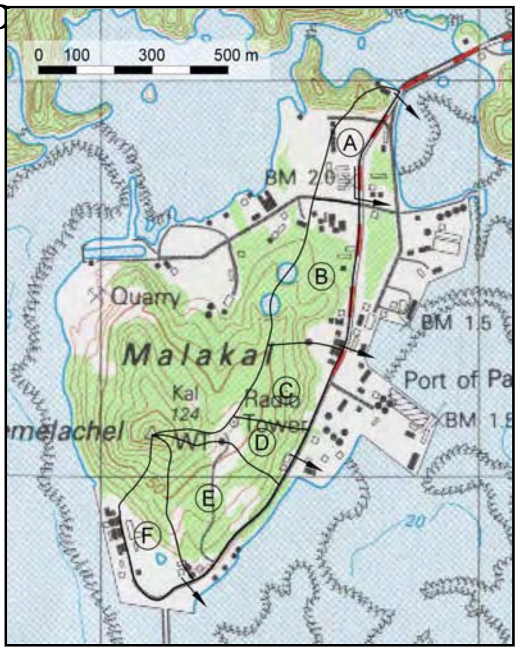


Fig. 11-1 Catchment Area of Malakal Island Road

The design runoff from the adjacent area were calculated by the following rational formula.

$$Q = \frac{1}{3.6 \times 10^6} \cdot C \cdot I \cdot a$$

where, Q : Peak runoff (design peak discharge: m^3/sec)
C : Runoff coefficient
I : Design rainfall intensity for a duration equal to the time of concentration (mm/h)
a : Catchment area (m^2)

Based on the site conditions, runoff coefficient (C) was determined as follows:

Table 11-2 Applied Runoff Coefficients (C)

Catchment No.	Land Use	Applied "C"
A	Suburban residential	0.7
B	Hill slope and residential area	0.5
C	Hill slope	0.7
D	Hill slope	0.7
E	Hill slope under construction	0.7
F	Hill slope, partially flat	0.7

Source: Guidelines for earthworks - drainage system, Japan Road Association

The time of concentration ("t" in minute) is the time required for the surface runoff from the remotest part of the catchment to reach the point being considered. The time of concentration (t) consists of the time (t_1) required for overland flow, and the time (t_2) required for stream flow. The time of concentration for overland flow (t_1) down to the point being considered was estimate by the following formula proposed by Kerby:

$$t_1 = \left[\frac{2}{3} \times 3.28 \cdot L \cdot \frac{n_d}{\sqrt{s}} \right]^{0.467}$$

where, t_1 : Time of concentration for overland flow (min)
L : Length of flow (m)
Nd : Retardance coefficient
S : Slope of the surface

Retardance coefficient (nd) was recommended for use as shown in Table11-3.

Table 11-3 Retardance Coefficient (nd) for Kerby's Formula

Ground Cover	Value of Nd
Asphalt, Concrete surface	0.013
Smooth bare packed soil, free of stones	0.10
Average grass	0.40
Deciduous timberland	0.60
Conifer timberland, dense grass	0.80

The time concentration for overland flow was estimated for each catchment as listed in Table 2-28 based on the topographic map prepared by Department of the Interior, USA, and assumed retardance coefficient of nd=0.6:

Table 11-4 Estimation of Time of Concentration for Overland Flow

Catchment No.	Length of Flow (m)	Elevation Difference (m)	Slope of the Surface (%)	t_1 (min)
A	100	53	53.0	11.3
B	400	43	10.8	31.4
C	250	58	23.2	21.0
D	380	119	31.3	23.9
E	180	29	16.1	19.7
F	400	122	30.5	24.6

The time of concentration for stream flow (t_2) was estimate by the following formula:

$$t_2 = \frac{L}{60 \cdot V}$$

where, t_2: Time of concentration for stream flow (min)
V: Average flow velocity (m/sec)
V was estimated by the following equation:

$$V = \frac{1}{n} \cdot R^{\frac{2}{3}} \cdot I^{\frac{1}{2}}$$

where, n: Mannings roughness coefficient
R: Hydraulic Radius (m) = Area/Wetted Perimeter
I: Stream Slope

Average stream flow velocity of 80% water depth in drainage channel (allowable flow capacity) was estimated as illustrated in Figure 11-2 assuming concrete U-ditch of 50 cm×50 cm, and concrete V shaped ditch of W1000 cm×H30 cm for estimation purpose.

Based on the Figure 11-3, the representative stream flow velocity for the Study was judged as 2.0 m/sec which corresponds to 1.0% to 2.0% of stream slope. When the average length of drainage channel in every catchment was assumed around 300 m, then the time of concentration for stream flow is estimated as 2.5 minutes.

Design rainfall intensity was estimated in accordance with Figure 11-3, the relationship of Rainfall Intensity- Duration-Frequency, which was provided for the design of Compact Road Project. Applied return period for this drainage design was 25 years.

Based on the above mentioned factors, the design runoff for drainage system was estimated for each catchment, as shown in Table 11-5.

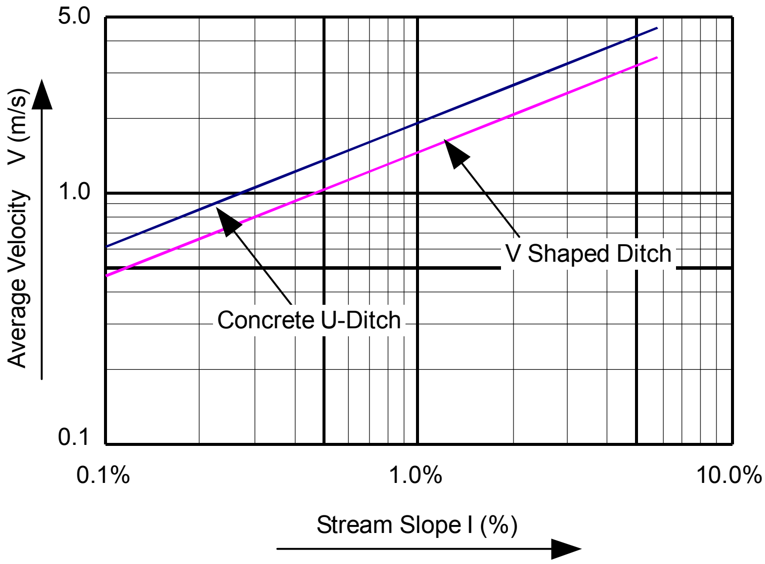


Fig. 11-2 Relationship between Average Velocity and Stream Slope

Table 11-5 Design Runoff of each Catchment

Catchment	A	B	C	D	E	F
Catchment Area (m^2)	15,700	55,100	42,000	72,400	3,800	75,800
Runoff Coefficient	0.7	0.5	0.7	0.7	0.7	0.7
Time of Concentration (min)	13.8	33.9	23.5	26.4	19.7	27.1
Design Rainfall Intensity (mm/h)	101.0	66.4	78.8	74.6	85.6	73.3
Design Runoff (m^3/sec)	0.308	0.508	0.644	1.050	0.063	1.086

Figure 11-3 shows the relation between allowable flow capacity of drainage channel and channel slope for concrete U-ditches, V shaped ditches, and concrete pipes. The drainage system for Malakal Island Road was prepared as shown in Table 11-6, based on the estimated design runoff of 25 years return period and relation of ditch type/size-allowable flow capacity-channel slope illustrated in Figure 11-4

Table11-6 Malakal Island Road Drainage System Schedule

Design Discharge		Hill Side	Cross Drainage	Sea Side V	Outlet Channel
	EP of Malakal Island Road				
0.308 m^3		U700x700 L=170 m *1		L=180 m	
	STA.1+450	CB 800x800	ϕ800, L=11m		U, L=40 m
0.508 m^3		U700x700 L=60 m *2		L=100 m	
	STA.1+350	CB 800x1000	ϕ1000, L=11m		U, L=40 m
		U700x700 L=210 m		L=202 m *3	
	STA.1+140				
0.644 m^3		U700x700 L=90 m		L=90 m	
	STA.1+050	CB 800x1000	ϕ1000, L=11m		U, L=40 m
		U700x700 L=220 m		L=200 m *4	
	STA.0+830				
1.050 m^3		U700x700 L=120 m		L=120 m	
	STA.0+710	CB 800x1000	ϕ1000, L=11m		U, L=40 m
		U700x700 L=130 m		L=60 m	
				STA.0+650	
	STA.0+580				
		U400x400 L=180 m			
	STA.0+400	CB 600x600	ϕ500, L=11m		U, L=40 m
0.063 m^3		U400x400 L=80 m			
	STA.0+320				
1.086 m^3		U400x400 L=140 m		STA.0+270	
	STA.0+180				
		U700x700 L=100 m		L=190 m	
	STA.0+080	CB 800x1000	ϕ1000, L=11m		U, L=40 m
		U700x700 L=80 m		L=80 m	
	STA.0+000				

Remarks,
1. "U", "V" and ""CB" represent U-Ditch, V-Ditch and Catch Basin respectively.
2. The Basic Design Study proposed to install V-Ditch of W1000×H300 at the sea side to enable vehicle override. It is required to adjust the dimensions and size variations to follow Palau standards was planned to install.

*1 Length of 10 m for T-Junction (STA. 1+525~535) is excluded.
*2 Length of 40 m for T-Junction (STA. 1_300~340) is excluded.
*3 Length of 8 m for T-Junction (STA. 1+225~233) is excluded.
*4 Length of 20 m for T-Junction (STA. 0+860~880) is excluded.

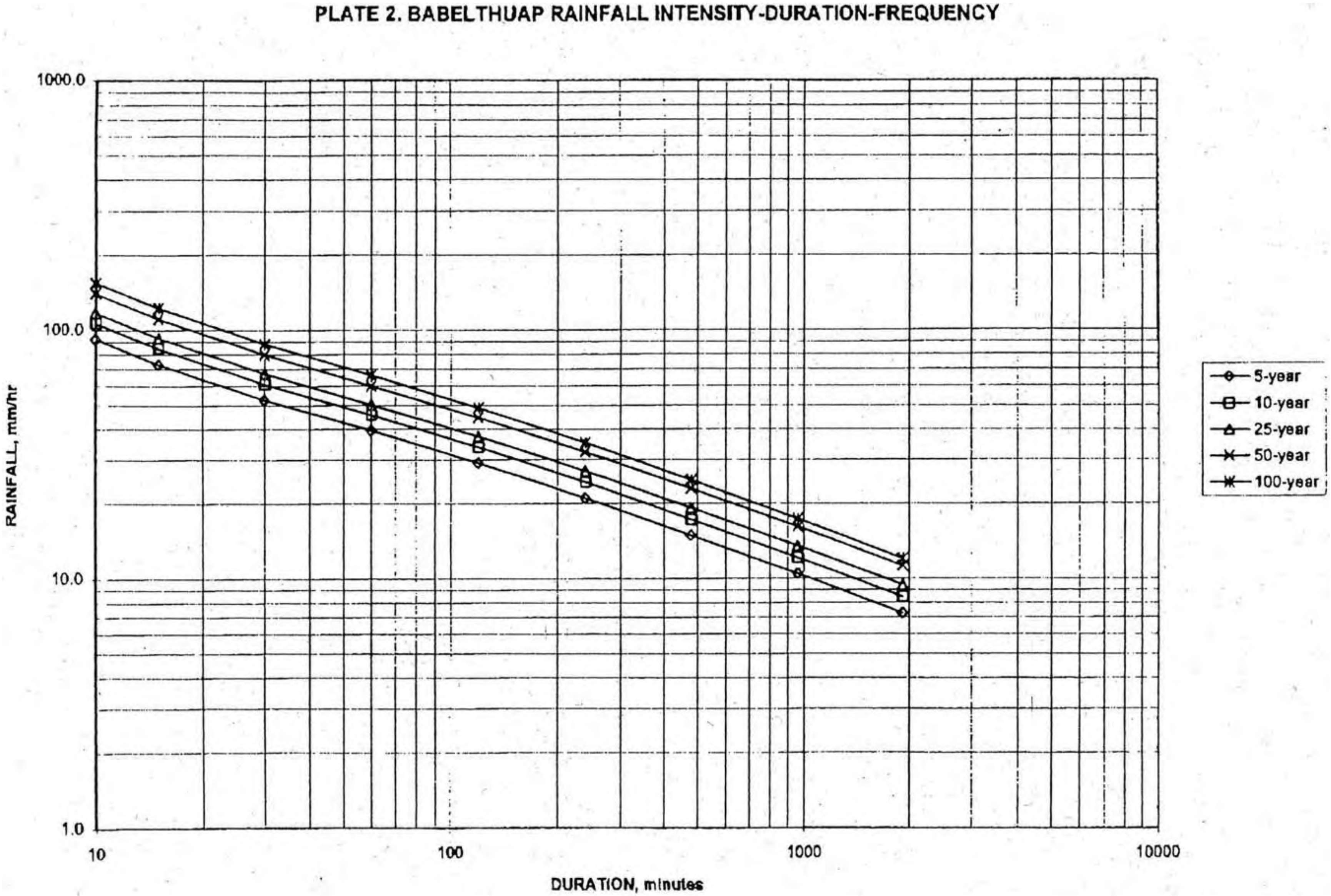


Fig.11-3 Relation between Rainfall Intensity-Duration-Frequency

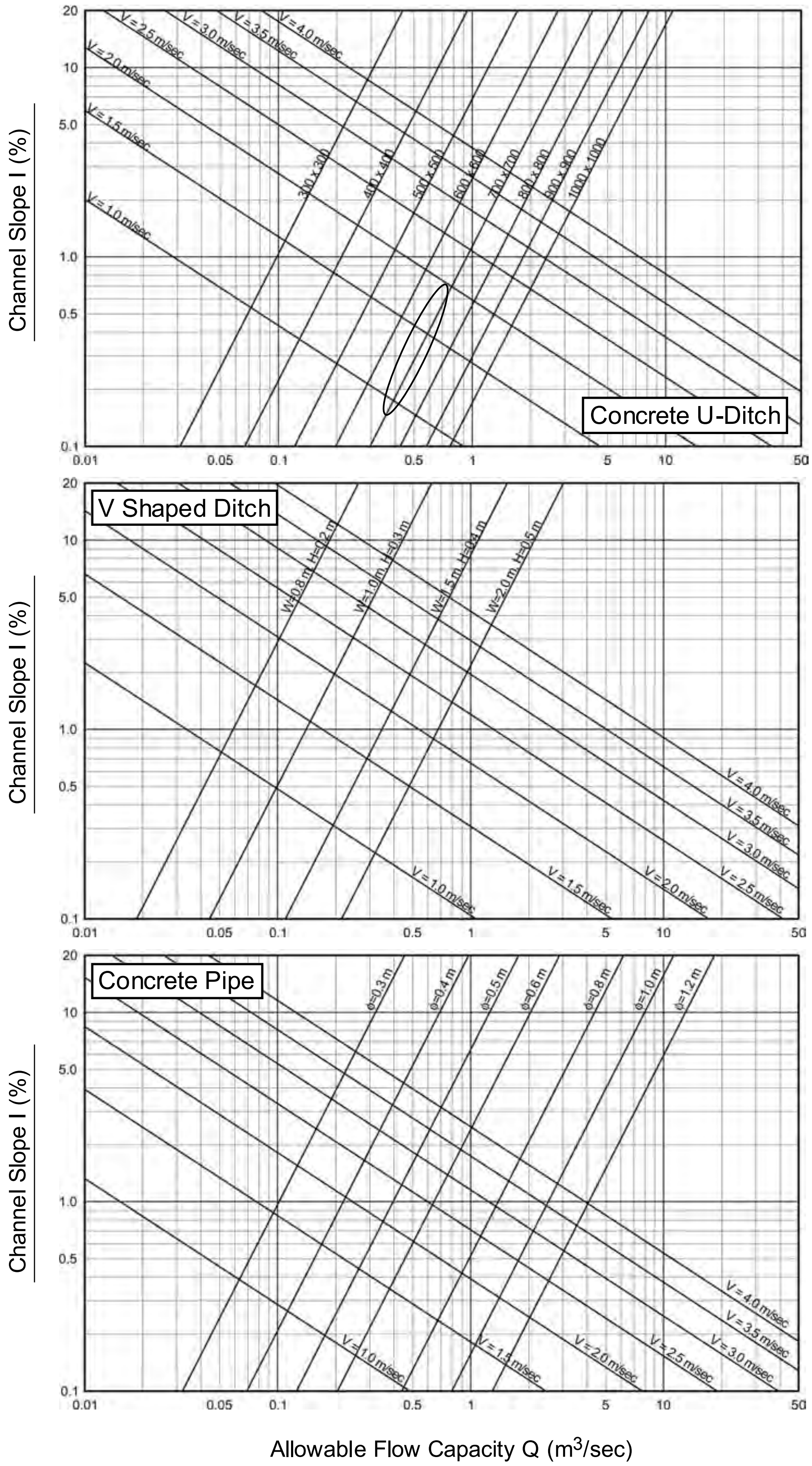


Fig.11-4 Relationship between Allowable Discharge (80% Flow Capacity) and Channel Slope

Table 11-7 Quantities of Drainage System

Unit: m

U700	U400	Junction	V1000	ϕ1000	ϕ800	ϕ500	U700
170	+	10	=180		11		
60	+	40	=100	11			40
210	=	8	+202				
90			90	11			40
220	=	20	+200				40
120			120				
130	180		60	11			40
	80		0				
100	140		190			11	40
80			80	11			40
1,180	400	78	1,222	55	11	11	240