ANNEXES

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ANNEX I. TRAFFIC ASSESSMENT

Traffic Surveys

The guidance in this Annex is derived principally from Overseas Road Note 20 (TRL 2003) and Overseas Road Note 40 (TRL, 2004) amended for LVRR application on the basis of recommendations in the ReCAP LVRR Design Manual for Ethiopia (2016) and Tanzania (2016).

In order to prepare geometric and pavement designs, it is necessary to estimate a both equivalent daily traffic volume or Annual Average Daily Traffic (AADT) and Passenger Car Units (PCUs) for the design period for the road. Whereas a 'total vehicle count' does not differentiate between small and large vehicles, PCU count takes vehicle type into account and so addresses the extra congestion caused by large vehicles or IMTs. Traffic surveys are carried out to measure existing traffic flows and characteristics on a route.

Traffic survey methodology

The following advice for traffic counts is recommended by ORN 40.

On an existing route for higher traffic LVRs (estimated at combined directions flow of >100vpd), count the traffic in each direction for 7 days and for 24 hours each day. If security for night surveys is a problem, arrange for security protection personnel, or carry out verbal investigations to determine the nature and estimated quantities of night traffic.

On lower traffic roads, count traffic in both directions from dawn until dusk for 2 days. If traffic is known to pass at night, make investigations regarding the nature and flows, or multiply by 1.2 to estimate the 24 hour count; if no traffic passes at night, the 24 hour count equals the day count. Avoid market days and intense periods of harvest activity if possible. Single-day counts are too variable and should not be used.

The counts should be carried out during the dry season. Wet season counts may be very low due to poor road conditions. A standard traffic count field data sheet is included as Figure I.3.

Calculate the average daily traffic for each vehicle type and then convert the average daily traffic to an equivalent daily traffic using the factors in Table I.1 for the different vehicle types and then sum to give the total equivalent daily traffic for pavement design.

For geometric design purposes it is necessary to convert the average daily traffic to equivalent Passenger Car Units (PCUs) using the factors in Table I.2 for the different vehicle types. These data are then summed give the total PCUs as an input to geometric design.

Table I.1 Typical Equivalent Traffic Conversion for AADTs (TRL,2003, ORN 20)

Vehicle Type	Equivalent traffic factor
Truck and bus	5
Tractor	4
Small bus	2
Pick up	1
Car	0.8
Animal	0.2
Motorcycle	0.1
Bicycle	0.05
Pedestrian	0.02

Table I.2 PCU values (See Chapter 6)

Ref.	Vehicle Type	PCU
1	Car, Light Van, jeeps and Pick Up	1.0
2	Light Truck up to 2.5 tonnes gross	1.5
3	> 2 axle Truck up to 10 tonnes gross	3.0
4	Truck up to 15 tonnes gross	4.0
5	2 axle tractor towed trailers -standard	3.0
6	Single axle tractor towed trailers -standard	1.5
7	Bus up to 40 passengers	3.0
8	Bus over 40 passengers	4.0
9	Motorcycle or scooter	0.5
10	Bicycle	0.5
11	Rickshaw and Tricycle carrying goods	1.0
12	Auto Rickshaw	0.75
13	Hand Cart	2.0
14	Bullock Cart with Tyre	6.0
15	Bullock Cart with Wooden Wheel	8.0
16	Horse-drawn carts	6.0
17	Pedestrian	0.2

If there is established traffic, but it is impossible to organise a traffic survey, traffic can be estimated from a moving observer count. The 12 hour count, Q, is estimated as follows:

$$Q = (x + y) \text{ X } 720/t$$

Where x = the number of vehicles passing in the opposite direction.

y = the number of vehicles which overtake minus those which are overtaken

t = the time taken to drive the road minutes.

This calculation should be made for each vehicle type and then night factors and equivalent traffic factors multiplied as appropriate. Since a moving observer count can be inaccurate, particularly when traffic volumes are low, it is recommended that a minimum of one count in each direction, and preferably two in each direction, are made and then averaged.

If the route is impassable or is not yet developed, then traffic estimates will have to be developed from consideration of the economic potential of the road 'catchment area' and socio-economic needs. This requires very careful consideration of a range of factors, such as local economic development potential, alternative routes, network connectivity and agricultural development. In such circumstances the road design strategy should be flexible to allow for ease of upgrading should the future traffic flows substantially exceed the 'desk' assessment of likely traffic.

Axle load surveys

Axle loading data is a vital input to the process of designing the road pavement, particularly for sealed road options and for bridge design. Axle load data is not generally required for unsealed earth of gravel road design.

Overloading in parts of Myanmar is reported to be widespread, therefore, axle load surveys or at least truck loading surveys are strongly recommended. Due to the exponential nature of pavement damage due to overloaded vehicles, it is important to pragmatically assess the potential axle loading scenarios as part of the road environment input into design.

The aim of an axle load survey is to estimate the number of ‘equivalent standard axles’ currently using the existing road. To do this, a survey is undertaken to determine an average equivalency factor for each vehicle type Axle load surveys can be undertaken using fixed or portable weighbridges, as well as weigh-in-motion equipment. See Figure I.4 for use in axle load surveys.

Evaluating traffic flow volumes for pavement design

Depending on the paving option, the basis for pavement design is either Annual Average Daily Traffic (AADT) or Equivalent Standard Axles (esa).

Current traffic, as derived from the surveys, will be used as the basis for developing predictions of traffic for the design period for the road. For impassable or undeveloped routes future traffic should be estimated on the basis of similar roads in similar environments.

Future traffic growth prediction is developed from.

- **Normal traffic:** Traffic which would pass along the existing road or track even if no new pavement were provided.
- **Generated traffic**: Additional traffic which occurs in response to the provision or improvement of the road. This will be the principal component of new roads and likely to be based the results of origin and destination surveys (TRL, 1993)
- **Diverted traffic:** Traffic that changes from another route (or mode of transport) to the project road because of the improved pavement, but still travels between the same origin and destination.

Normal traffic

The most common method of forecasting the growth of normal traffic is to extrapolate time series data on traffic levels and assume that growth will either remain constant in absolute terms (a linear extrapolation) or constant in relative terms (a constant elasticity extrapolation), i.e. traffic growth will be a fixed number of vehicles per year or a fixed percentage increase. Data on national or regional fuel sales can often be used as a guide to country-wide or regional growth in traffic levels although improvements in fuel economy over time should be considered.

As an alternative to time, growth can be related linearly to GDP (or gross domestic income). This is normally preferable, since it explicitly takes into account changes in overall Myanmar national economic activity.

It is likely, however, that LVRRs will be substantially affected by local factors and that the rate of economic development in Myanmar will vary considerably in localities depending on local projects and programmes in a range of sectors.

If it is thought that a particular component of the traffic will grow at a different rate to the rest, then it should be specifically identified and dealt with separately. The opening-up, or even temporary use, of quarries can significantly impact on the vehicle usage.

Construction traffic can also be a significant proportion of total traffic on LVRRs (sometimes 20 – 40 % of total traffic) and should be taken into account considered in the design of the pavement.

For very low volume roads (traffic <25 vpd), a detailed traffic analysis is seldom warranted because environmental rather than traffic loading factors generally determine the performance of roads.

Diverted traffic

Where parallel routes exist, traffic will usually travel on the quickest and most economical route, although this may not necessarily be the shortest. Thus, surfacing an existing road may divert traffic from a parallel and shorter route because higher speeds are possible on the surfaced road. Origin and destination surveys can be carried out to provide data which can be used to estimate likely traffic diversions. Assignment of diverted traffic is normally done by an 'all-or-nothing' method in which it is assumed that all vehicles that will save time or money by diverting would do so, and that vehicles that would lose time or increase costs will not transfer. With such a method, it is important that all perceived costs are included. Diverted traffic is normally forecast to grow at the same rate as traffic on the road or mode from which it diverted.

Generated Traffic

Generated traffic arises either because a journey becomes more attractive because of a cost or time reduction, or because of the increased development that is brought about by a road investment. It is difficult to forecast accurately and can be easily overestimated. It is only likely to be significant in those cases where the road investment brings about large reductions in transport costs. For example, in the case of a small improvement within an already developed highway system, generated traffic will be small and can normally be ignored. Similarly, for projects involving the improvement of short lengths of rural roads and tracks, there will usually be little generated traffic. However, in the case of a new road allowing access to a hitherto undeveloped area, there could be large reductions in transport costs as a result of changing mode from head loading to motor vehicle transport and, in this case, generated traffic could be the main component of future traffic flow.

The recommended approach to forecasting generated traffic is to use demand relationships (TRL 1993). The price elasticity of demand for transport measures the responsiveness of traffic to a change in transport costs following a road investment). On inter-urban roads, a distinction is normally drawn between passenger and freight traffic and, on roads providing access to rural areas, a further distinction is usually made between agricultural and non-agricultural freight traffic.

Traffic Lane Distribution

The actual traffic loading impacting the pavement needs to take into account the distribution of wheel loads across the pavement width, Table I.3.

Table I.3 Factors for Distribution of Pavement Traffic Loading TRL, ORN 31

Cross Section	Paved width	Correct design traffic loading (esa)	Explanatory notes
Single Carriageway	< 3.5m	Double the sum of the esas in both directions.	The driving pattern on this cross-section is very channelized.
	Min. 3.5m but less than 4.5m	The sum of esas in both directions.	Traffic in both directions uses the same lane.
	Min, 4.5m but less than 6m	80% of the esas in both directions.	To allow for the overlap in the centre section of the road.
	6m or wider	Total esas in the heaviest loaded direction.	Minimal traffic overlaps in the centre section of the road.
More than one lane		90% of the total esas in the studied direction.	The majority of vehicles use one lane in each direction.

Traffic Analysis Example

The following example of traffic analysis for pavement design is based on a traffic survey undertaken as part of the design for the DRRD pavement trials in Taungyii (KfW, 2014). The relevant 3-day traffic count is presented in Table I.4 along with corrected figure for ADT, PCU and esa.

Table I.4 Adjustment of Daily Traffic Counts

		1	2	3	4	5	6	7
		Ave. per day	ADT Factor	ADT	PCU Factor	PCU	Axle Factor	esa
A	Minibuses	10	2.00	20.0	3	30.0	0.30	3
B	Light Trucks <5 t	10	2.00	20.0	1.5	15.0	1.50	15
C	Heavy Trucks >11t	5	4.00	20.0	4	20.0	4.00	20
D	Very Heavy Trucks (13t)	1	4.00	4.0	4	4.0	8.00	8
E	Pickups	3	1.00	3.0	1	3.0	0.02	0.06
F	SUV/4WD	9	1.00	9.0	1	9.0	0.02	0.18
G	Cars/Saloon	4	0.80	3.2	1	4.0	0.01	0.04
H	Motorcycle trailers	10	0.50	5.0	1	10.0	0.01	0.1
J	Power tillers	50	0.50	25.0	3	150.0	0.01	0.5
K	Agriculture tractor	1	0.50	0.5	1.5	1.5	0.01	0.01
L	Motorcycles	150	0.10	15.0	0.5	75.0	0.01	1.5
M	**Total Motorized Vehicles**	**253**		**124.7**		**321.5**		
N	Bicycles	25	0.05	1.3	0.5	12.5		
O	Ox carts	5	0.10	0.5	8	40.0		
P	**Total Non-motorized Vehicles**	**30**		**1.8**		**52.5**		
Q	**TOTAL**	**283**						
R	Pedestrian	120	0.02	2.4	0.2	24.0		
S			**Total**	**129**	**Total**	**398**	**Total**	**48**

The adjustment factor for ADT and PCU were obtained from Tables I.2 and I.3. The esa equivalence factor adjustment factors are estimated (TRL, 1993 ORN 31) but should ideally be based on local axle load surveys or local knowledge of the vehicle fleet using the road in question.

From Table I.4 we can get the following data:

- Cumulative motorised AADT year 1 — 125 (cell M3)
- PCU (Motorised Traffic) — 322 (cell M5)
- PCU (Non-Motorised Traffic) — 53 (cell P5)
- Daily esa (Desa) — 48 (cell S7)

For pavement design the figure Desa has to be adjusted to take into account:

- Diverted traffic: this was assumed to be negligible
- Generated traffic; as an existing road in fair condition, this was also assumed to be negligible
- Design life ; this was taken as being 12 years (N)
- Traffic growth for this exercise was estimated as being 6%/year (R=0.06)
- Wheel path adjustment (Table I.3) for a 4.5m wide carriageway is 80% of esa

- Design CBR% with pavement on imported fill is taken as 6%

Cumulative esa (Cesa) = Desa x 0.8 x 365 x[(1+R)^N – 1]/R Equation 1

Or Using Figure I.1 (ReCAP, 2017) with calculated year multipliers, equation 1 becomes

= Desa x 0.8 x 365 x year multiplier

= 48 x 0.8 x 365 x 16.5

= 238,272.or traffic range LV3 (0.1-0.3 Mesa)

This LV3 figure can then be taken to the design charts in chapter 7 and, for example, for a DBST sealed road on compacted fill of CBR 6%, the thickness design would be as shown in Table I.7

Figure I.1 Multiplier for the first year Cesa to calculate the Cesa after a number of years (ReCAP, 2016a**)**

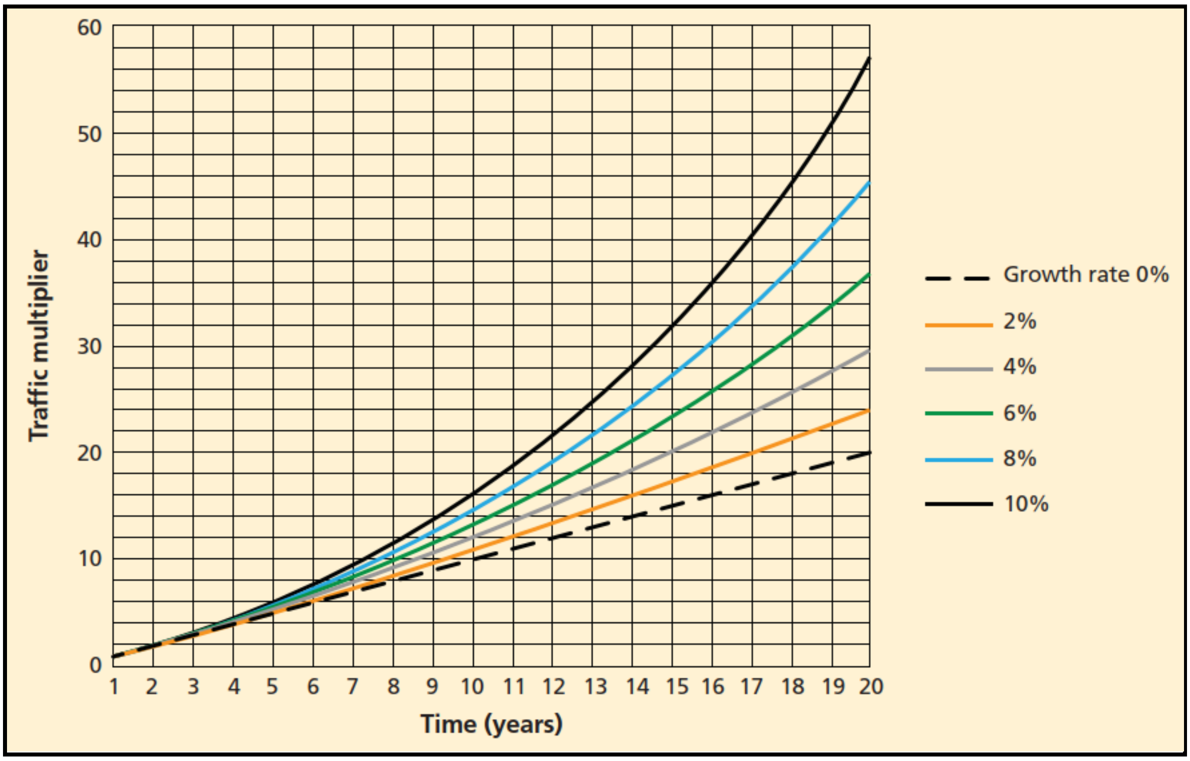


Table I.5 Example of Thickness Selection

Traffic range (mesas)	Layer	LV1	LV2	LV3	LV4	LV5
Subgrade class (CBR)		**< 0.01**	**0.01 – 0.1**	**0.1 – 0.3**	**0.3 – 0.5**	**0.5 – 1.0**
S2 (3-4%)	Base	150 G65	150 G65	150 G65	175 G80	200 G80
	Sub-Base	150 G15	125 G30	150 G30	175 G30	175 G30
	Subgrade		130 G15	175 G15	175 G15	200 G15
S3 (5-7%)	Base	125 G65	150 G65	150 G65	175 G65	200 G80
	Sub-Base	150 G15	100 G30	150 G30	150 G30	150 G30
	Subgrade		100 G15	150 G15	150 G15	150 G15
S4 (8-14%)	Base	175 G45	150 G65	150 G65	175 G65	200 G80
	Sub-Base		120 G30	200 G30	200 G30	200 G30
S5 (15-29%)	Base	175 G45	125 G65	175 G65	175 G65	175 G80
	Sub-Base		125 G30	150 G30	150 G30	150 G30
S6 (>30%)	Base	150 G45	150 G65	175 G65	175 G65	200 G80

If, as in this, case the calculated esa is close to a Traffic Group border, it is usual to keep it in the higher groups. Note that for unsealed roads a similar process can be gone through with AADT using Figure 7.7 in Chapter 7. In the above example the level of traffic indicates it unsuitable for an unsealed option. If however the road carried only light traffic (no trucks or busses) then around AADT 130 would result. With a design life of 8 years this would have indicated a GWC of 175mm over sub-base of 150mm G25 material, using the multipliers in Figure I.2.

Figure I.2 Multipliers for Obtaining Future AADTs for Different Growth Rates (ReCAP, 2016a)

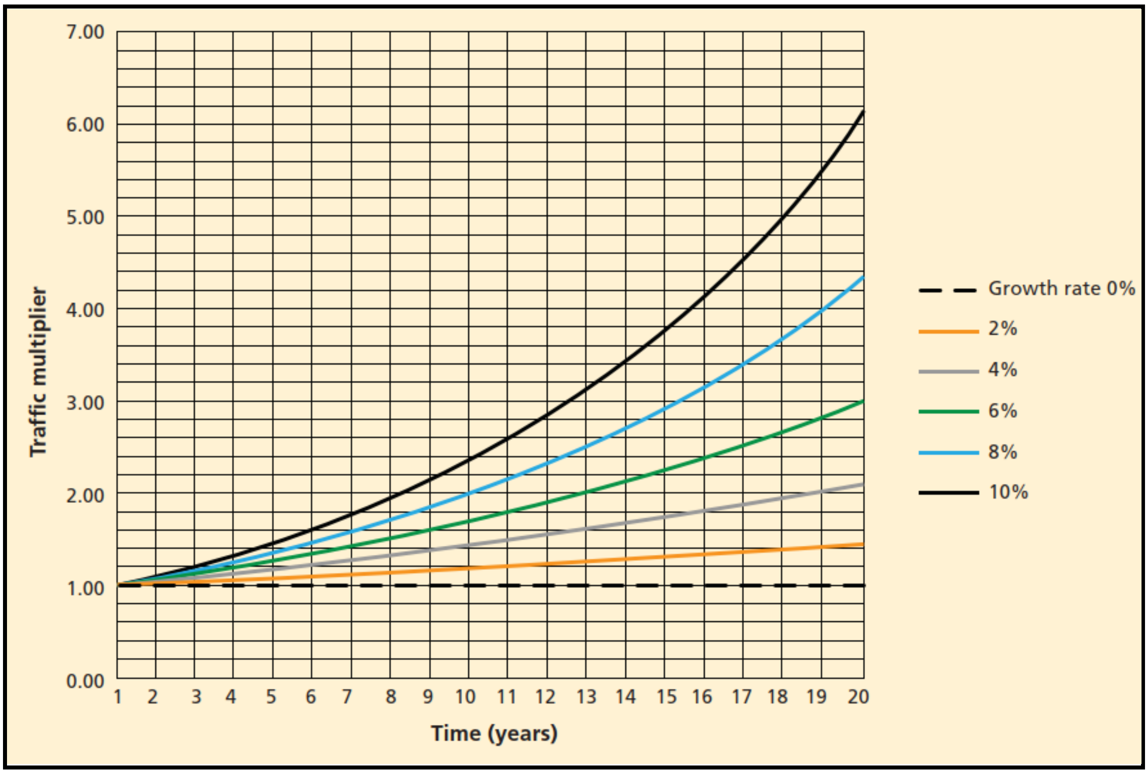


Figure I.3 Typical Traffic Count Form for Low Volume Rural Traffic (Intech-TRL, 2006)

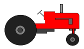
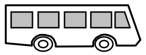
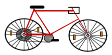
Province					**SURVEYOR**	
District					**LOCATION**	
Daily 12 hour counts *DATE*						
Traffic Class	**0600-0900hrs**	**0900-1200hrs**	**1200-1500hrs**	**1500-1800hrs**	**Option for additional Hours**	**Daily Average**
MOTORCYCLE 						
CAR, 4WD, PICKUP 						
Tractor 						
LIGHT TRUCK =< 5 TONS GVW 						
TRUCK > 5 T (2 axle) GVW 						
TRUCK > 5 T (3 axle +) GVW 						
Mini-bus/Bus 						
PEDESTRIAN, WALKER						
ANIMAL/HAND CART 						
BICYCLE 						
TOTALS						
Rain This Period?						
Daily Survey Period:	6.00 hours to 18.00 hours		*GVW = Gross Vehicle Weight*			

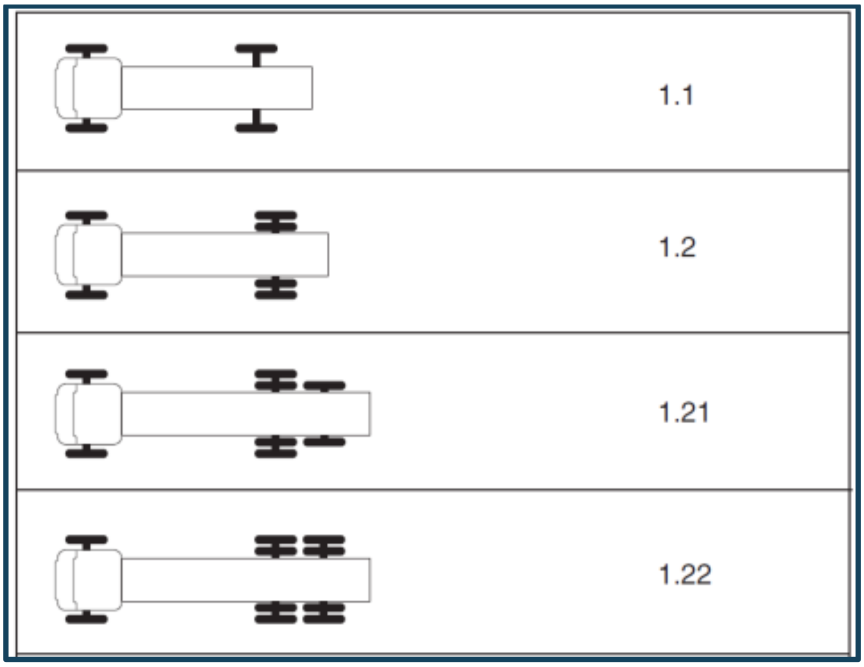
Figure I.4 Typical Axle Load Form

AXLE LOAD SURVEY

Road Name/Number:		**Survey Date:**	
Province:		**Time of survey:**	
		Surveyor:	

No	AXIAL CONFIG	AXIAL LOADS (TONNES)				Comment on vehicle type	Type of loading
		1	2	3	4		
1							
2							
3							
4							
5							
6							
7							
8							
9							
10							
11							
12							
13							
14							
15							
16							
17							
18							
19							
20							
21							

Figure I.5 Axle configurations to be used in conjunction with Figure I.4



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ANNEX II. LABORATORY SOIL AND ROCK TESTING

Objectives

An assessment of the geotechnical properties of the soils and rocks associated with a road project is a vitally important aspect of its cost-effective design and construction. This assessment may be concerned with the condition and performance of the soil-rock masses along the route or with the suitability of various soils and rocks as construction materials. In either case laboratory testing is likely to form an integral part of this assessment within the overall framework of the geotechnical and materials investigations (Chapters 5 and 8).

To be effective, laboratory testing programmes should take into account not only the selection of appropriate tests, but also account for the capacity of the laboratory and staff to undertake the tests and quality manage the data produced.

This Annex outlines the key issues in the design and undertaking of laboratory test programmes. Particular emphasis should be placed on the selection of appropriate tests and the need for effective quality management throughout the whole testing and reporting process. The detail in this Annex is based largely on the references in Table II.1.

Table II.1 Key References for LVRR Laboratory Testing

Reference	Relevance
Cook J R, Gourley C S and Elsworth N E, 2001. Guidelines on the selection and use of road construction materials in Developing Countries.	Chapter 6, in particular provides a guidance on selection of tests in the context of LVRRs
Roughton International Ltd 2000. Guidelines on Materials and Borrow Pit Management for Low Cost Roads.	A companion publication to Cook et al 2001, it provides guidance on the selection and analysis of LVRR materials tests with illustrations and examples.
Head K.H., (1992). Manual of Soil Laboratory Testing. Vol 1, Soil Classification and Compaction Tests	Provides guidance on the soil classification and compaction testing as well on the management and QA of laboratories and their data. Based on British Standard test methods
Head K.H., (1994). Manual of Soil Laboratory Testing. Vol 2 Permeability, Shear Strength and Compressibility Tests	Provides guidance on the simple strength testing required for LVRR projects. Based on British Standard test methods
Sabatini P.J, et al .2002. Evaluation of Soil and Rock Properties. Report No.FHWA-IF-02-034.	Provides guidance on the sampling, testing and analysis of soil and rock material based on AASHTO and ASTM procedures.

Testing Programmes

Laboratory testing programmes vary greatly in size and scope depending on the nature of the road project and associated works. Testing should not be commissioned on an arbitrary or ad hoc basis but should be part of a rationally designed programme. Clear objectives should be identified and test programmes need to be designed with these in mind. The relationships between in situ conditions and those experienced by the sampled and tested material need to be taken into account when developing test programmes.

Within an overall aim of assuring that selected materials and designs are capable of carrying out their function, testing is undertaken for a number of reasons.

- Characterisation of soil rock masses and materials along the route;
- Assessment of geotechnical properties influencing earthwork cuts and fills;
- Assessment of geotechnical properties of natural hazards;
- Identification of potential material resources;
- Proving quality and quantity of material reserves or processed materials;

- Construction quality assurance;
- In service monitoring;

It is useful to divide materials test procedures into a number of general categories that reflect the nature of the test. These general are:

1. Physical: Index Tests associated with defining inherent physical properties or conditions;
2. Simulation: Tests associated with portraying some form of geotechnical or engineering character either directly or by implication;
3. Chemical: Tests aimed at identifying the occurrence of key chemical compounds;
4. Petrographic: Those tests or assessments associated with analysing or describing fabric or mineralogy.

Tables II.2 to II.4 list common soil and rock material tests under the above headings, taken largely from Cook et al, 2001

An understanding of the properties being measured by the individual tests is important in the selection of appropriate procedures. Simulation testing, in particular, may be based largely on empirical testing procedures rather than modelling expected service behaviours.

In the majority of cases no single test procedure will satisfy specification requirements and a battery of test procedures will be needed. An appropriate test programme specification will include a logical selection and sequence of procedures that is function of material quality, the environment and the road design.

Final as-built road quality is dependent on the processes of selection, winning, hauling, spreading and compaction, and attempts need to be made to replicate and to predict their impacts through pre-treatment programmes prior to testing. For example, by subjecting samples due for particle size analysis, to a compaction cycle prior to sieving. Aggregate impact testing (e.g. AIV), abrasion, soaking, drying or slake durability pre-treatments could also be used in appropriate circumstance on samples prior to a main test.

Table II.5 presents recommendations on sample sizes required for testing.

Application of Testing Standards

The majority of laboratory tests in developing countries are governed by strict procedures that, in the main, have been originally derived from British (BS), American (AASHTO, ASTM) or French (AFNOR) Standards. In most cases they have been incorporated into national standards, sometimes, however, with local amendments. It is understood that AASHTO-ASTM standards are adopted in Myanmar, with additional supplementary testing to BS where no AASHTO-ASTM equivalent exists.

Standards such as AASHTO-ASTM lay down standards of good practice that are in the main based on "normal" experience with temperate zone sedimentary soils. When dealing with tropical residually weathered materials special procedures are often necessary to obtain reliable, relevant and consistent results. This applies particularly to the handling and treatment of samples before testing (Head 1992).

The approach to the laboratory investigation of tropical materials in terms of the range of tests employed, their detailed procedures and their interpretation should derive principally from the following:

- Chemically bonded materials (e.g. affects assessment of strength);
- Mineralogical complexity. (e.g. influences volume change);
- Fragile relict fabric and texture. (e.g. leads to particle break-down);
- Moisture susceptibility.

Table II.2 Laboratory Physical Condition Index Tests

Physical Condition Tests	Standard Procedures		Comment on Test	Disadvantages and Factors to be Aware of.	Alternative/Modified Tests
	AASHTO	ASTM			
Moisture Content	T265	D2216	Simple and widely accepted test.	Misleadingly high moisture contents in halloysitic and allophane rich soils.	Drying at differing temperatures to establish "working" moisture content. Sand bath option available option for granular materials.
Water Absorption	T84	C127 & C128	Simple test with correlations established with bitumen-bound material design.	Variability in multi-clast type deposits.	
Liquid Limit (WL)	T89	D4318	Well established soil index and classification test.	Influence of >425μm particles; moisture condition and mixing time. Correlations between AASHTO/ASTM and BSS procedures require caution.	Undertake at differing moisture states. Drying at differing temperatures. ASTM D421 is an air dry option.)
Plastic Limit (Wp)	T90	D4318	Well established soil index test. Plasticity index (Ip = W_L-Wp) used as a key defining parameter in many specifications.	Influence of >425μm particles; moisture condition and mixing time. Poor reproducibility and repeatability. Correlations between.	Undertake at differing moisture states. Drying at differing temperatures. Potential differences in plasticity results between ASTM and BS procedures.
Shrinkage Limit (Ws)	T92	D427 & D4943	Yields index information on volume change potential, not common,y undertaken for LVRR projects.	Initially intended for undisturbed samples although remoulded material can be used.	D427: Shrinkage factor. D4943 Wax method.
Linear Shrinkage (Ls)	T216	(BS1377)	Can give an estimate of Ip for soils where W_L and Ws are difficult to obtain Better repeatability and reproducibility than Ip	Established relationships between Ls . and Ip may not hold true for some tropical soils.	Drying at differing temperatures.

Table II.3 Laboratory Physical Condition Index Tests (continued)

Physical Condition Tests	Standard Procedures		Comment on Test	Disadvantages and Factors to be Aware of.	Alternative/Modified Tests
	AASHTO	ASTM			
Particle Size Distribution	T88	D422	Simple and widely accepted test incorporating both sieving and sedimentation.	Interpretation problems with aggregated particles or weak clasts. Requires particle density values.	Alternative dispersion agents; trisodium phosphate; tetrasodium phosphate better for some tropical soils. Cohesive soils with gravel/cobble; special attention.
Sand Equivalent Value	T176	D2419	A rapid site/lab means of determining relative fines content.	Dispersion problem in agglomerated minerals. Relative proportions only.	n/a
Aggregate Grading	T311	C136 & C117	Accepted test for aggregate size distribution.	Wet sieve unless little or no fines.	Dry sieving in materials free from agglomerated particles only.
Flakiness Index (If) Elongation Index (Ie)	T27	D4791	Standard gauge methods of ascertaining particle shape. Parameters incorporated into coarse aggregate specifications.	Use restricted to coarse aggregate only.	Additional shape test: Average Least Dimension (ALD): NTRR, 1986. D4791 produces estimates of flat, elongated or flat and elongated particles only.
Angularity Number	T304	(BS 812:105)	Rapid indirect method of estimating roundness based on relative voids.	Can only be valid for strong aggregate particles.	Roundness also by observational methods.
Soil Particle Density	T100	D854	Required for use in analysis of other parameters	Some soils influenced by drying temperature. Care required in testing of materials with clasts of variable mineralogy	Undertake at natural moisture content. Drying at differing temperatures.
Aggregate Particle Density (Bulk particle or Relative Density)	T85	C127/128	Required in bitumen-bound granular material design calculations.	In aggregate the procedure will give an "apparent" rather than an "absolute" value.	Can be measured for a number of states: saturated surface dried (SSD); wet surface dried (WSD) or oven dried (OD).
Bulk Density	T19M/T19	C29 & C29M	Variety of density definitions.	Need to have clarity on density definition.	Variety of alternative methods. ASTM test for aggregate <150mm.

Table II.4 Laboratory Simulation Tests (continued)

Simulation Tests	Standard Procedure		Comment on Test	Disadvantages and Factors to be Aware of.	Alternative/Modified Tests
	AASHTO	ASTM			
Swell Pressure	T258	D4546	Undertaken on undisturbed or recompacted material to determine pressure to minimise swell.	Only measures swelling pressure. Soil or fine aggregate only. To measure swell amount use BS 1377:5, 4.4.	Swell amount test; ASTM D4546. ASTM: D4829 - use of swell index EI. Unconfined swell; ISRM (1981)
Collapse	T258	D4546	Can give good indication of potential for fabric collapse.	Disturbance problems in sensitive fabric materials.	Alternatively Collapse Potential Index (CPI) load at 200kPa; Jennings and Knight (1975).
Consolidation (oedometer)	T216	D2435	Consolidation characteristics of as-compacted soil-fill or on undisturbed samples.	Disturbance problems in sensitive materials. Allows vertical drainage only, unrealistic in structured materials.	For radial drainage consolidation and for undisturbed materials the use of the Rowe cell procedures is recommended
Pinhole Test		D4647	Laboratory assessment of soil dispersion.	Based on empirical evaluation of material performance, mainly in temperate materials.	D4221 – Double hydrometer test based on comparison of gradings before and after artificial dispersion. Needs PI>4.
Crumb Test		D6572	Laboratory assessment of soil dispersion.	Based on empirical evaluation of material performance, mainly in temperate materials	Physical observational tests associated with chemical tests for sodium cations in pore water.
Compaction	T99 & T180	D698 & D1557	Simple test. Basis of control on site compaction of fill and pavement materials.	Zero air voids a function of particle density- highly variable in tropical soils. Be aware of relationships between "laboratory" and 'engineering' moisture.	Avoid drying of samples as much as possible and use fresh sample for each moisture point

Table II.5 Laboratory Simulation Tests (continued)

Simulation Tests	Standard Procedure		Comment on Test	Disadvantages and Factors to be Aware of.	Alternative/Modified Tests
	AASHTO	ASTM			
CBR	T193	D1883	Quick and simple to perform. A convenient and widely established test for defining material suitability for road construction and subsequent quality control.	An empirical test only. Correlations with other parameters may be material-specific. Dependant on soil moisture-density-void ratio conditions. Material >19mm excluded.	A range of conditions and procedures. ASTM allows for testing at a range of compactive efforts.
Triaxial: UU (soil)	T296	D2850	Unconsolidated Undrained Short term fill analysis and cut-slope during construction.	Unconsolidated Undrained. . Not strictly applicable for non-saturated conditions or for non-cohesive materials.	Single point tests at range of moisture contents for trafficability.
Triaxial: CU (soil)	T297	D4767	Consolidated Undrained (with pore pressure measurement). Long term effective stress analysis of cut slopes	Sophisticated test requiring careful supervision of experienced staff. Not commonly used in LVRR projects	May need increased backpressures in some residual soils.
Triaxial: CD (soil)		D7181-11	Consolidated Drained. Enables long-term effective stress analysis of fill slopes.	Time consuming test requiring careful supervision of experienced staff. Not commonly used in LVRR projects	May need increased backpressures in some residual soils.
Unconfined Compressive Strength (Soil)	T208	D2166	Quick and straightforward method of obtaining undrained shear strength.	Material needs to be intact, cohesive and at least stiff in consistency.	
Vane Shear (Lab)		D4648	Rapid test for undrained shear strength.	Only of use for soft saturated clays with no coarse particles.	Can be used on materials in compaction or CBR moulds.

Table II.6 Laboratory Simulation Tests (continued)

Simulation Tests	Standard Procedure		Comment on Test	Disadvantages and Factors to be Aware of.	Alternative/Modified Tests
	AASHTO	ASTM			
Point Load Strength	[ISRM]	D5731	Simple test with portable equipment. Correlates with UCS.	Sensitive to changes in moisture condition and surface crushing. Requires identical samples (10 min). Correlations with compressive strength vary with material types.	Tests at soaked, natural and dry moisture conditions.
Unconfined Compressive Strength (rock)	[ISRM]	D2938	Straightforward test for measure strength of intact rock samples.	Requires regular (core) shaped samples. Sensitive to changes in sample moisture condition, orientation and end-face preparation.	Tests at soaked, natural and dry moisture conditions.
Schmidt Hammer	[ISRM]		Very simple portable field test modified from concrete test. Derived for non-destructive concrete testing. Can be used on intact rock.	Correlations with strength require confirmation per rock type.	Laboratory procedures specify "L" type hammer; alternative "N" hammer requires separate correlation.
Aggregate Impact Value (AIV)	[812: 112]		Simple test with inexpensive portable equipment giving a basic index parameter for aggregates.	Flakiness, elongation can influence results as well as base-floor condition. Tests limited grading. For limited grading only	Soaked/unsoaked tests. AIV(R) value measures breakdown from 10-2.36mm (M)AIV limits blows for weaker materials. Ethylene glycol soaking may be appropriate for some materials.
Aggregate Crushing Value (ACV)	[812:110]		Gives basic index parameter for aggregates commonly used in specifications.	As for AIV.	Soaked/unsoaked tests. ACV(R) value measures breakdown from 10-2.36mm.
10% Fines Aggregate Crushing Tests	[812: 111]		Modification of ACV test, particularly for weaker materials.	As for AIV	Soaked/unsoaked tests. Ethylene glycol soaking may be appropriate for some materials such as basic igneous rocks.

Table II.7 Laboratory Simulation Tests (continued)

Simulation Tests	Standard Procedure		Comment on Test	Disadvantages and Factors to be Aware of.	Alternative/Modified Tests
		ASTM			
Sulphate Soundness	T104	C88	Assesses aggregate durability as a response to repeated crystallisation and rehydration stresses. Incorporated in many specifications.	Time consuming. Poor repeatability and reproducibility unless great care taken over procedures.	Magnesium sulphate may be preferred to sodium sulphate because of greater penetrating power of the saturated solution.
Slake Durability	T210 & [ISRM]	D4644	Simple assessment of durability of rock-like material.	Fragile materials require careful handling.	Use with plasticity index for argillaceous materials. D3744: Durability Index-separate fine and coarse tests.
Los Angeles Abrasion (LAA)	T96	C131/535	Standard combined impact and rolling abrasion test. Commonly used as a specification parameter.	For aggregate <37.5mm. Tests a specified grading only. Measures breakdown in terms of material passing 1.68 mm sieve only.	ASTMC535 for aggregate >19mm.
Micro-Deval		[NF P 18-572]	Similar to LAA test but is also used to define surface aggregate suitability.	Measures breakdown in terms of material passing 1.6 mm sieve only. Smaller equipment than LAA.	Can be reported "dry" : MD:S or "wet" MD:E.
Accelerating Polishing Test	T279	D3319 &E303	Means of assessing the tendency for aggregate to polish. Polished Stone Value (PSV) commonly incorporated into surfacing aggregate specifications.	Difficult and time-consuming test not normally carried out in standard laboratories. Selected aggregate pieces only.	D3319. PSV based on accelerated polishing machine. E660 based on small wheel circular track polishing machine.
Aggregate –Bitumen Adhesion	T182	D1664	Tests for assessing adhesion of bitumen to aggregate in water.	Observational test only. Takes account of stripping only and not prior coating difficulties.	D2849. Degree of particle coating. Adhesion may be indirectly assessed by mineralogical examination.

Table II.8 Soil and Aggregate Chemical Tests

Chemical Tests	Standard Procedure		Advantages of Using Test	Disadvantages and Factors to be Aware of.	Alternative/Modified Tests
	BS	ASTM			
pH	1377:3, 9	E70	BS Electrometric: Standard method, accurate to 0.1pH.	Requires regular cross-checking against buffer solutions.	Use of Indicator papers - simple and quick, approximate values only Colourmetric method requires comparison with standard charts.
Sulphate Content	1377:3, 5.2-5.5		Total sulphate in soils, including water- soluble calcium sulphate. Accurate if performed with care.	If measured sulphate content is >0.5% the water soluble sulphates should also be measured.	Water soluble sulphate in soil and sulphates in water also by gravimetric (1377:3, 5.6) and ion exchange(1377:3, 5.5) methods.
Organic Content	1377:3, 3	C40	BS dichromate oxidation method. Accurate and suitable for all soils. Fairly rapid test.	Presence of chlorides influences results, a correction can be applied.	Peroxide oxidation - used to eliminate organic matter for PSD testing.
Carbonate Content	1377:3, 6.3	D4373	BS Rapid titration for carbonate content greater than 10%, has 1% accuracy.	Not suitable for carbonate content <10%. ASTM utilises gas pressure method.	Gravimetric 1377 3:6.4. Used for hardened concrete. D4373 solubility in HCL. Calcimeter: simple, quick - approximate but adequate for most engineering purposes.
Chloride Content	1377:3, 7.2-7.3	D1411	BS Silver nitrate method. Designed for concrete aggregate testing purposes.	Titration process requires proper chemical facilities.	Water Soluble: 1377 3:7.2; 812.117 BS Acid Soluble: 3:7.3. D1411, Calcium and magnesium chloride in graded aggregate.
Loss on Ignition (LOI)	1377:3, 4		Destroys all organic matter. Applicable for sandy soils containing little or no clay.	High temperature may break down water of crystallisation in some minerals and give misleading results.	

Table II.9 Petrographic Assessment Procedures

Petrographic Procedure	Procedure Description	Procedure Application
Aggregate: Qualitative Visual Examination	Record general character of aggregate sample including grading, texture, shape and rock type.	A quick and rapid assessment
Aggregate: Quantitative Visual Examination	Sieve into separate size fractions and examine each fraction in terms of grading, texture, shape, rock type and mineralogy. Utilise additional procedures set out below as appropriate.	Detailed petrographic procedure for identification of weak and/or unsuitable materials and recognition of potentially deleterious minerals.
Methylene Blue Value	Based on absorption of methylene blue by clay minerals. Powdered rock or fine soil sample suspended in solution and then titrated with methylene blue.	Rapid method of indicating the presence of deleterious clay minerals Does not give any indication of mineral type. May need additional fabric assessment work for more reliable results.
Binocular Microscopy	The use of plane light binocular microscope requires little sample preparation. Small hand-held microscopes can be used in the field.	A quick and straightforward method for the examination of soil fabric and texture of hand specimens. Photographs can be easily taken to support descriptions
Thin Section Microscopy	The traditional geological method of examination of mineralogy and fabric of thin sub-samples of hand specimens under both plane and polarised light.	May be used for the examination of fabric and as a means of establishing mineral composition by point-count techniques. Difficult to make sections in friable materials. Possible to take photographs.

Table II.10 Standard Materials Tests and Required Sample Sizes

Test Procedure	ASTM	Minimum Sample Required Fine	Medium	Coarse
Moisture Content	D2216	0.05kg	0.35kg	4.00kg
Liquid Limit (Cone /Casagrande))	D4318	0.50kg	1.00kg	2.00kg
Liquid Limit (one point Cone)	D4318	0.10kg	0.20kg	0.40kg
Plastic Limit	D4318	0.05kg	0.10kg	0.20kg
Shrinkage Limit	D427	0.50kg	1.00kg	2.00kg
Linear Shrinkage	(BS1377)	0.50kg	0.80kg	1.50kg
Particle Size (Sieve)	C136 -117	0.15kg	2.50kg	17.00kg
Particle Size (Hydrometer)		0.25kg		
Particle Density	D854	0.30kg	0.60kg	0.60kg
Compaction – CBR (Modified)	D1883		80.0 kg	
Mg/Na Soundness	C88	150g	600g	850g
Chemical Tests (Organic, Chloride, carbonate etc.)	C40, D1411, D4373	150g	600g	350g
Point Load Test	(ISRM)		Ten identical samples	
Los Angeles Abrasion (LAA)	C131		5.00-10.00kg	

Derived Indices

A number of common soil indices are derived from relationships between, Atterberg limits and particle size and are used to characterise unbound granular materials and soils. These can be useful for characterising general engineering and geotechnical behaviour. Commonly used grading indices are defined below:

Fines Ratio (FR) = P 0.075/P 0.425

Grading Coefficient (GC) = (P 26.5 – P 2.00) x P 4.75/100

Grading Modulus (GM) = [300 - (P 2.00 + P 0.425 + P 0.075)]/100

Coarseness Index (IC) = (100 – P 2.36)

Fineness Index (IF) = P 0.075

(Where P 0.425 = percentage of material passing the 0.425mm sieve etc., and P is the percentage passing the sieve size given)

Parameters defined to evaluate the relationship between plasticity and fines content include:

Plasticity Modulus = Plasticity Index x % passing 0.425 mm sieve

Plasticity Product = Plasticity Index x % passing 0.075 mm sieve

Shrinkage Product = Linear Shrinkage x % passing 0.425 mm sieve

References

BSS 1980. BS 1377: Methods of Test for Soil for Civil Engineering. British Standards Institution

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ANNEX III. MARGINAL CONSTRUCTION MATERIALS

Introduction

For the purposes of this Manual the discussion on marginal materials is largely focussed occurring road construction materials that do not comply with accepted specifications but which can perform adequately in service for pavements within identifiable limits.

In Chapter 8 it was noted non-standard materials might be used successfully in LVRRs pavements where traffic is low and the road environment understood and, provided quality control is adequate.

Specifications drawn-up for specific project environments need not be as conservative as overall international or national specification and hence may allow the use of previously non-conforming or marginal materials. In effect this means selecting materials on an “appropriateness-for-use” basis.

Details in the Annex are drawn primarily from two documents:

Austroads, 2018.Appropriate Use of Marginal and Non-standard Materials in Road Construction and Maintenance. Technical Report AP-T335-18.

Cook J R, Bishop E C, Gourley C S and Elsworth N E. 2002. Promoting the use of marginal materials TRL Ltd DFID KaR Project PR/INT/205/2001 R6887.

General characteristics of marginal materials

Marginal materials that could be considered for use in Myanmar in pavement construction can effectively be grouped within a five tier system as shown in Table III.1.

Table III.1 Marginal Material Groups

Group	Sub-Group	Examples
I Strong Rocks		Foliated Metamorphic Rocks Crystalline Basic Igneous & Metamorphic Rocks
II Weak Rocks	II Inherently Weak or Poorly Consolidated Rocks	Weak Conglomerates &Sandstones Shale, Siltstone and Mudstone Deposits
	II b Weathered and/or Highly Fractured Rocks	Weak Volcanic Agglomerates and Breccias Other partially Weathered Rocks
III Natural Granular Deposits	III a Transported Soils and Gravels	Alluvial Sand & Gravel Deposits Alluvial and Aeolian Sand Deposits Clayey Sand Deposits Colluvial Deposits
	III b. Residual Soils and Gravels	Quartz Gravels Weathered Granite / Gneiss Other Residual Gravelly Soils Clayey Sand Deposits
IV Duricrust Gravels		Laterites
V Manufactured Materials		Fired Clay Bricks Demolition Waste (Concrete and Brick) Industrial By-products (Plastic) & Waste Material Products

Tables III-2 to III-8 provide a summary review of typical aspects of each group. These tables describe the typical international and regional examples and identify the properties of the materials which force their consideration as marginal materials. Within each group consideration is given to the potential use of the materials in road construction and further considers actions that can be taken to improve the material standard.

Table III.2 Review of Marginal Materials: Group I hard rocks

Material Types	Material Description	Typical Defects	Potential Pavement Construction & Performance
FOLIATED METAMORPHIC ROCKS : Common Types: • Slate • Phyllite • Schist • Gneiss • Amphibolite	Strong massive to closely jointed STRONG ROCKS, which may produce poorly graded materials on crushing comprising a significant proportion of flaky and elongate particles.	POOR PARTICLE SHAPE. High proportion of flaky particles (If > 40%) in roadbase materials will lead to poor particle interlock, compaction difficulties and relatively low in situ dry densities. HIGH MICA CONTENT. High content of micaceous minerals can lead to difficulties with compaction in the laboratory and on site. May also affect liquid limit determination and unrealistically high PI's that bear little relationship to field performance.	Materials with poor particle shape tend not to satisfy laboratory CBR required for "standard" roadbase materials. May be satisfactory for lower standard roadbase design such as CBR 50 or CBR 40% for low volume sealed roads (less than 0.5 – 1.0 Mesa). Can be improved by mechanical stabilisation – blending with well-shaped angular materials designed to improve particle interlock, reduce voids and produce a smooth curve within the desired grading envelope. De-densification of compacted layers can occur due to presence of excess mica, particularly when using vibratory compaction equipment.
CRYSTALLINE IGNEOUS ROCKS 1 Special Group: (fine to medium grained) Basic Igneous Rocks i.e.: • Basalt • Dolerite • Gabbro	Strong massive to closely jointed STRONG ROCKS which can typically be processed by crushing and screening to produce desirable grading.	DECAY IN-SERVICE DUE TO MINERAL ALERTATION. Apparently sound, but slightly weathered, strong rock aggregate may deteriorate (decompose) rapidly after processing and in the road pavement to produce plastic fines.	Provided that secondary mineralisation is not significantly developed then these hard rocks will produce good quality crushed roadbase, sub-base and sealing aggregate. Susceptible materials can however deteriorate during pavement design-life and even while stockpiling.

Table III.3 Review of Marginal Materials: Group IIa Weak or Poorly Consolidated Rocks

Material Types	Material Description	Typical Defect	Potential Pavement Construction & Performance
WEAK VOLCANIC AGGLOMERATES AND BRECCIAS	May comprise poorly consolidated (rippable) deposits that when excavated produce variably graded silty sandy angular to sub angular GRAVEL and COBBLES with some boulders.	• POOR "AS DUG" GRADING. Frequently gap graded with a high proportion of oversize material. • HIGH VARIABILITY WITHIN OUTCROP. Often interbedded with finer ash deposits , which may have high PI. Near surface deposits may be weathered but with well cemented HARD ROCK appearing at depth. • UNSOUND STONE CONTENT. Rippable materials may have undergone significant weathering	Rarely suitable for use in pavement construction without some processing to reduce oversize content and improve grading. Cobble and boulder size fragments are typically strong and may be difficult to treat with a grid roller or mobile hammer mill. Crushing and processing is likely to be required.
WEAK CONGLOMERATES	Weakly cemented rock comprising sand and pebbles that typically produces moderately to well graded silty SAND and rounded to subangular GRAVEL with a variable proportion of cobbles	• POOR PARTICLE SHAPE. Rounded particles have poor interlocking properties, hence "as dug" conglomerate deposits will tend to be difficult to compact and produce low dry densities. • VARIABLE UNSOUND STONE CONTENT. Conglomerate gravels can comprise a mix of rock types and may contain a significant proportion of weak or weathered particles. • HIGH PLASTICITY FINES. Some conglomerates may have a fine matrix producing high plasticity fines.	Conglomerate gravels will typically require crushing and screening in order to satisfy "standard" roadbase specification requirements. Roadbase materials may be supplied from well graded or simply screened Crushed gravels for use in bituminous surfacing should be investigated to determine their unsound (weathered and inherently weak) stone content and adhesion characteristics.
WEAK SANDSTONES	Weakly cemented rock predominantly comprising sand size particles usually dominated by quartz although feldspar material (arkose) also encountered.	• LOW PARTICLE STRENGTH. • POOR AGGREGATE DURABILITY. Particularly associated with argillaceous (clayey) sandstones. • POOR "AS DUG"GRADING • HIGH PERMEABILITY loss of strength on saturation. • HIGH PI in arkose material when feldspars decay .	Selected deposits may supply roadbase. materials for low volume sealed roads in low rainfall areas e.g. those exhibiting high un-soaked CBR values but poor soaked CBRs.

Table III.4 Review of Marginal Materials: Group IId Partially Weathered or Highly fractured rocks

Material Types	Material Description	Typical Defect	Potential Pavement Construction & Performance
FRACTURED/ WEATHERED (RIPPABLE) LIMESTONES	Fractured and weathered rock forming clayey slightly sandy angular GRAVEL and cobbles.	• HIGH PI CARBONATE FINES. Typically associated with weathering along joints and fractures. • POOR "AS DUG" GRADING with variable proportion of oversize. • DIFFICULT TO CRUSH with traditional equipment due to clogging. Grid roller or mobile hammer mill may be appropriate.	Well graded (suitably processed) clayey materials typically provide high soaked CBR strengths of 60 – 80%. Can supply roadbase aggregates for low volume sealed roads.
ARGILLACEOUS MATERIALS Shale Siltstone Mudstone	Fine grained weak rocks that may be fissile. Typically produce silty to clayey weak angular or platy GRAVEL.	• LOW PARTICLE STRENGTH. Inherently weak rock types. • AGGREGATE DETERIORATION. Will tend to "slake" after extraction and in the road to produce plastic fines. • POOR GRADING • POOR SHAPE	Some materials may be suitable for use as sub-base in roads up to medium traffic in well drained dry conditions. Will tend to soften rapidly in wet conditions.
WEATHERED ROCKS	Many partially weathered rock types (whether sedimentary, igneous or metamorphic) may produce sandy GRAVEL materials. Fracture spacing and or bedding planes facilitate extraction of well graded materials by dozer ripping.	• VARIABILITY WITHIN OUTCROP. Expect considerable and sometimes unpredictable lateral and horizontal variation in aggregate quality.. • PRESENCE OF DELETERIOUS SECONDARY MINERALS • LOW PARTICLE STRENGTH. • POOR "AS DUG" GRADING • HIGH PLASTICITY FINES	Some rippable partially weathered and fractured rock types can supply roadbase material for low volume sealed roads. Aggregate quality will vary according to degree of alteration (i.e. depth below ground). Selection and mixing during extraction may be critical to obtaining a satisfactory material. A wider range of weathered rock types will be suitable for supply of sub-base and selected subgrade aggregates.

Table III.5 Review of Marginal Materials: Group IIIa: Transported Soils and Gravels

Material Types	Material Description	Typical Defect	Potential Pavement Construction & Performance
ALLUVIAL SAND DEPOSITS	Typically silty non plastic to low plasticity SAND deposits.	• UNIFORMITY OF PARTICLE SIZE. Poor performance in pavement layers is associated with sand deposits comprising a high proportion of single size particles. • POOR PARTICLE SHAPE.	Well graded unstabilised materials may be suitable for sub-base construction (soaked CBR 20-30%). Cement or bitumen treated materials can form roadbase, but can exhibit shrinkage cracking with former. .
ALLUVIAL CLAYEY SAND DEPOSITS	Clayey (low to moderate PI) silty SAND.	• POOR GRADING. By definition these deposits lack gravel size fraction. Materials with good engineering properties will usually have a wide range of fine grained particle sizes. • POOR PARTICLE SHAPE. Angular particles provide good interlock and improved engineering properties. • MODERATE PI FINES. Performance is related to the PIs but more significantly related to the volumetric stability	Un-stabilised materials have been used for roadbase construction for very low volume sealed roads in low rainfall areas (< 500 mm/year). If cement stabilisation is considered for more highly trafficked roads these are prone to cracking and preferred use is in sub-base beneath an un-stabilised roadbase.
COLLUVIAL DEPOSITS	Typically coarse angular SAND and GRAVEL deposits with a variable cobble and boulder content in a matrix of silty sand or sandy clay.	• POOR GRADING. Usually gap graded with a high proportion of oversize material. • VARIABILITY WITHIN THE DEPOSIT. Colluvial deposits frequently comprise a variable mix of rock types. • HIGH PI FINES.	The character of these deposits is dependent on the nature of the parent rocks and terrain.
ALLUVIAL GRAVEL DEPOSITS	Typically moderately to well graded silty SAND and rounded to subangular GRAVEL with a variable proportion of cobbles and boulders.	• POOR PARTICLE SHAPE. Rounded particles have poor interlocking properties, and difficult to compact . • VARIABLE UNSOUND STONE CONTENT. Alluvial deposits comprise a mix of rock types that reflect the geology of the drainage catchment. • HIGH PLASTICITY FINES. Some alluvial deposits, particularly terrace deposits, may contain an excess of plastic fines. s.	Alluvial gravels typically require crushing and screening in order to satisfy “standard” roadbase specification requirements. Roadbase materials for low volume sealed roads may be supplied from well graded or simply screened (i.e. grizzly) subrounded to subangular deposits.

Table III.6 Review of Marginal Materials: Group IIIb: Residual Soils and Gravels

Material Types	Material Description	Typical Defect	Potential Pavement Construction & Performance
RESIDUAL CLAYEY SAND DEPOSITS	Clayey (low to mod PI) silty SAND.	• POOR GRADING. By definition these deposits lack gravel size fraction. Materials with good engineering properties will usually have a wide range of fine grained particle sizes. • POOR PARTICLE SHAPE.	Un-stabilised materials have been used as roadbase for low volume and very low volume sealed roads. Problems have been encountered with cement improved/ stabilised lateritic clayey sands. Careful evaluation is required if stabilisation is considered.
RESIDUAL GRAVEL DEPOSITS	Variably graded typically clayey sandy angular to subangular GRAVEL.	• POOR GRADING. These deposits tend to be variably graded within the exploitable horizon and are frequently gap graded. • HIGH PLASTICITY FINES. In situ weathering can lead to mineralogical decay that produces plastic fines. • HIGH UNSOUND STONE CONTENT. High proportion of partially weathered particles can be present	"As dug" deposits will rarely be suitable for standard roadbase construction, due to inherent variability in terms of grading, particle strength and plasticity. However, this group of deposits has been widely used as a source of aggregate for lime or cement improved/stabilised roadbase material. Also used as roadbase and sub-base in low volume sealed roads in arid, semi-arid and seasonally wet climatic areas.

Table III.7 Review of marginal materials: Group IV: Duricrust or Pedogenic Gravels

Material Types	Material Description	Typical Defect	Potential Pavement Construction & Performance
Laterite Deposits	In situ varies from moderately strong rock (curasse) to weakly cemented or dense clayey gravel. "As dug" materials highly variable but typically clayey to silty slightly sandy subangular relatively weak GRAVEL.	• LOW PARTICLE STRENGTH. Particle strength is highly variable but rarely complies with "standard" pavement materials requirements. • HIGH PLASTICITY FINES. Low plasticity deposits occur and perform well in road pavements, however many laterite aggregates contain a high proportion of plastic fines that exceeding standard recommendations.	When well compacted these deposits form a dense relatively impervious pavement with good load bearing characteristics. Higher plasticity materials can be subject to significant loss of strength on saturation. Careful laboratory testing is needed in addition to case study experience from the region to be confident in using the material as roadbase.

Table III.8 Review of Marginal Materials: Group V: Manufactured Materials

Material Types	Material Description	Typical Defect	Potential Pavement Construction & Performance
Bricks	Bricks with a variety of strength characteristics may be produced depending on the available raw material source and quality manufacturing procedures. Bricks may be used whole or crushed.	• POOR GRADING. Hand knapping of bricks tends to produce poorly graded aggregates. • LOW PARTICLE STRENGTH. Particle strength is variable but relatively high strength aggregates can be produced from crushed high quality brick.	Whole brick (block) pavements with bitumen surfacing have been successfully constructed in rural and urban situations. If adequately supported by underlying layers then such pavements should carry relatively heavy traffic loads. Strong (over burnt bricks) can be crushed to produce roadbase material for light to medium trafficked roads (up to 3 M esa). 10% to 20% sand may be needed to improve grading and stability of some crushed brick products. Crushed brick mixed with sand (20%–50%) has been found to be suitable for use as sub-base material in medium trafficked roads.
Demolition Waste (concrete & brick)	Recycled brick and concrete waste can be processed to supply various qualities of graded granular material.	• VARIABLE PARTICLE STRENGTH. Particle strength will vary in accordance with the variability of the materials being recycled.	Guidelines for use of crushed brick aggregates will generally be applicable (see above).
Industrial By-products & Waste Materials Products • Blast Furnace Slag • Fly Ash • Mine Tailings • Demolished asphalt Pavement	Variety of granular materials including ash and clinker type waste products from coal burning power stations and steelworks.	• LOW PARTICLE STRENGTH may be associated with ash and slag materials. • POOR GRADING. Most waste products tend to be relatively uniformly graded.	Sometimes used in lower pavement layers. Pozzolanic materials may be suitable for use as stabilising additives in upper pavement layers

Problem Definition and Analysis

Key steps in the decision-making process in relation the use of marginal or non-standard materials may be summarised as follows

- Defining the material characteristics;
- Evaluation of the engineering environment;
- Assessment of engineering risk in the light of above.

In addition, in order to define the limits of use it is necessary to clearly identify the "non-standard" characteristics of the materials and furthermore to identify and understand the engineering limitations of the material. This will involve one or more of the following activities:

- Identification of the standard laboratory-based properties (usually those laid out in the specification);
- Detailed examination using special laboratory tests or procedures;
- Assessment of changes in engineering character during construction operations;
- Evaluation of in-service performance.

A clear understanding of the above will enable a more confident judgment of the appropriate use of the material.

Options for Use or Improvement

To compensate for using lower strength materials greater thicknesses of material may be needed in some circumstances to protect the road from sub-grade deformation. The use of higher compaction standards for some marginal materials may not possible or appropriate (e.g., foliated materials of Group 1, weak materials in Groups II, IIIa and IV).

Achieving even higher levels of compaction than those normally specified for sub-base and base could be a relatively cheap method of increasing the stiffness of the pavement and increasing performance of harder materials such as:

- Crystalline materials in Group 1;
- Some weathered materials (Group IIb);
- Alluvial or colluvial materials (Group IIIa);
- Residual gravels (Group IIIb);
- Highly indurated duricrusts (Group IV).

Groups may be defined according to the non-standard or defective property that will be identified during laboratory investigations. The "defect groups" considered are:

- High Plasticity Materials;
- Poorly Graded Materials;
- Poorly Shaped Materials;
- Low Particle Strength Materials;
- Low Durability Materials.

Tables III-9.to III-13 review each of these "defect groups" and present a summary of geological material types associated with the non-standard property.

Table III.9 High Plasticity Materials

Types of Marginal Material	Problems	Test Methods & Analysis to Quantify and Limit the Problem	Pavement Design To Accommodate High Plasticity Fines	Options for Improving Material Quality / Performance
Laterite Gravels Quartz Gravels River Terrace Deposits Colluvial deposits Clayey Sands Other Rippable Weathered Rocks	**Engineering Properties** Poor soaked CBR results (i.e. poor load bearing capacity) Compaction problems Susceptibility to loss of strength on wetting **Pavement Defects** Potholes Rutting Cracking	**Standard Tests**: Liquid Limit (LL), Plastic Limit (PL), Linear Shrinkage (LS), Activity Hydrometer Grading Compaction and CBR Grading Modulus Plasticity Modulus Plasticity Product Shrinkage Modulus **Special Tests** Mineralogy Chemical analysis, Volume change	Restrict use according to climatic or road environment factors. (take note of potential flood or drought risks). Restrict use according to traffic type & loading, e.g. very low-volume roads only. Ensure protection from pavement saturation: Good bituminous surface seal Sealed Shoulders prevent upward migration of moisture (i.e. from underlying layers). Maintenance of waterproof seals	Mechanical Stabilisation Blend with low plasticity material Lime Treatment: typically suitable for base when: Passing 0.425mm min 15% Passing 0.075 mm 5-35% PI 10 – 25% Soaked CBR min 20% Lime Treatment: typically suitable for sub-base when: Passing 0.425mm min 15% Passing 0.075 mm max 40% PI 10 – 30 % Cement Treatment: typically suitable for base when: PI max 25% Passing 0.075 mm 5-35% Soaked CBR min 20% Cement treatment: typically suitable for sub-base when: Passing 0.075 mm max 40% PI max 30% Bitumen Treatment: typically suitable for sand base when: Passing 0.075 mm 10-30% LL max 40% PI max 15%

Table III.10 Poorly Graded Materials

Types of Marginal Material	Problems	Test Methods & Analysis to Quantify and Limit the Problem	Pavement Design To Accommodate Poor Grading	Options for Improving Material Quality / Performance
Any natural granular deposit (incL. alluvial, colluvial, residual soils, duricrust deposits) Weak or poorly cemented materials (e.g. laterite curasse, weak conglomerate) Highly fractured competent rocks	**Coarse Gap Graded** Compaction Problems High % of voids, will result in high point loads; break-down of weaker particles and high permeability. Potential for collapse. Poor load bearing capacity (CBR) associated with poor particle interlock. **Excess Fines Content** Compaction Problems Poor internal friction characteristics with poor interlock between larger particles (ie they "float") resulting in low CBR. If fines are plastic the material will prone to weakening on saturation (low soaked CBR). **Uniformly graded** Poor compaction, low density & high permeability.	**Standard Tests**: ~Particle Size Distribution, ~Sand Equivalence Testing **All Materials** Grading Modulus and Uniformity Coefficients Curvature Coefficients Reject Index (% retained on 37.5mm sieve) Coarseness Index Fineness Index (% passing 0.075mm sieve) **b) Fine Materials** Void ratio Permeability Level of compaction and air voids Relationship between Compaction, Moisture Content and CBR	Restrict use according to climatic factors and road environment Restrict use according to traffic type & loading. Select aggregate grading specification that allows optimum use of available material. For example, consider: Water bound macadam Dry bound Macadam Telford base Ensure protection from pavement saturation if excess plastic fines.	Mechanical Stabilisation ~ Blend with materials that will improve grading characteristics Screen Removal of oversize usually feasible, but removal of sticky excess fines may be difficult when materials damp. Crush and screen To create desirable grading, using one or more material sources. Lime or cement treatment: typically suitable for improving materials with excess fines.

Table III.11 Review of Low Particle Strength Marginal Materials

Types of Marginal Material	Problems	Test Methods & Analysis to Quantify and Limit the Problem	Pavement Design To Accommodate Poor Particle Shape	Options for Improving Material Quality / Performance
Foliated Metamorphic rocks (flaky and elongated). Alluvial Gravels and Sands (rounded to subrounded). Conglomerates (rounded to subrounded)	Compaction Problems. High % of voids, will result in high point loads that will cause break-down of weaker particles and high permeability. May give poor CBR results (ie poor load bearing capacity) associated with poor particle interlock and internal friction.	• **Standard Tests**: Flakiness Index, Elongation Index, Particle Size Distribution, % Crushed Particles Crushing Coefficient Crushing Ratio Average Least Dimension **Visual inspection** Flakiness Elongation ALD Value Grading Modulus Well graded materials are better able to tolerate poor shaped particles due to reduced point load contacts, % voids and permeability. Level of Compaction	Restrict use according to traffic type and loading. Restrict use according to climatic and road environment factors.	Mechanical Stabilisation Blend with suitably graded materials that have good (cubical) particle shape. Crush Rounded materials will be improved by crushing. Improve crushing procedures (flaky materials) The type of crushing apparatus (ie whether toggle jaw crusher or cone crusher etc) may significantly influence the proportion of flaky particles produced during aggregate processing. Select compaction plant that will limit break-down of (carefully processed) aggregate during pavement laying.
Inherently weak rocks Marls & Limestones, Mudstone & Siltstones, Weak Sandstones, Weak Tuffs Partially Weathered Rocks (all types) Weak Natural Gravels Some calcretes	Change in grading characteristics during compaction. Including generation of excess fines. Difficulty in identifying MDD and OMC. Compaction Problems. Difficulty in achieving required field density. Low density will be linked to low CBR strength.	**Standard Tests:** Aggregate Crushing Value (ACV) Los Angeles Abrasion (LAA) Value Aggregate Impact Value (AIV) 10 % FACT Water Absorption Test	Restrict use according to traffic type and loading. Restrict use according to climatic factors – do not use in environments that will induce aggregate deterioration.	Mechanical Stabilisation Blend with stronger materials that will improve grading characteristics. Crushing and Screening Removal of weaker particles in a mixed strength material. Lime or cement treatment may significantly improve material

Some laterites Some silcretes Most volcanic scoria cinders) Volcanic ash & pumice Weak Manufactured Materials Weak Bricks Weak Demolition Waste Weak Industrial Wastes		**Recommended Test for Low Strength Aggregates** 10% FACT Wet and Dry Modified AIV procedures		performance. Match construction plant and construction procedures with material characteristics.

Table III.12 Review of Low durability marginal materials

Types of Marginal Material	Problems	Test Methods & Analysis to Quantify and Limit the Problem	Pavement Design To Accommodate Poor Durability	Options for Improving Material Quality / Performance
Argillaceous Rock types: Marl Limestone Mudstone, shale Argillaceous sandstone Poorly Cemented Rock Types: Weak Tuffs Weak Sandstones Partially Weathered Rocks (all types) Some basic to intermediate igneous rock Basalt Dolerite Gabbro Andesite etc.	Apparently strong pavement aggregates decompose in-service or during construction/stockpiling procedures (climatic influences important). Decomposition may generate plastic fines that are susceptible to softening and volumetric change on wetting or drying.	**Standard Tests**: Sodium Sulphate & Magnesium Sulphate Soundness Tests LAA Slake durability **Mineralogical Analysis**	Restrict use according to climatic or road environments factors – do not use in environments that will induce aggregate deterioration. Ensure protection from pavement saturation. Good bituminous surface seal. Sealed Shoulders. Prevent upward migration of moisture (i.e. from underlying layers). Maintenance of waterproof seals.	Mechanical Stabilisation Blend with materials that will diminish overall degradation Lime or cement treatment may inhibit durability problems but will require detailed investigation and possibly long term field trails.

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ANNEX IV. GROUND INVESTIGATION TECHNIQUES

Introduction

The following sections provide in some detail the principal Ground Investigation measures that are likely to be used for the assessment of ground conditions and materials for LVRR design in Myanmar. Form IV.1 is a typical walkover survey sheet that incorporates aspects of climate vulnerability.

Dynamic Cone Penetration (DCP) Tests

The DCP test is an effective method for obtaining sub-surface information for LVRR design to a depth of approximately 0.8m (1.40-1.60m with extension rod). The use of the DCP helps to delineate homogenous subgrade sections along the road and to identify soft spots of the subgrade for further investigation using pits and trenches. Using this test, strength characteristics of the subsurface soils at field moisture and density conditions can be obtained directly. The equipment is light and portable and is also useful for investigating the characteristics of pavement layers of existing roads for rehabilitation projects. In addition, DCP tests can also be used for quality control during construction and near surface estimations of strength and bearing capacity for shallow slope failure assessment and small structure foundation design.

Annex V DCP Options presents in more detail the procedures for incorporating DCP surveys into pavement design.

The In Situ Field Vane Shear Test

The vane shear test involves the use of a simple rotated blade to evaluate the undrained and remoulded shear strength in soft to stiff clays and silts. The use of the vane shear test should be limited to soils in which slow (6^{o} / min) rotation of the blade will lead to undrained shearing. Vane sizes range from a diameter of 38 to 92 mm, a height of 76 to 184 mm, a blade thickness between 1.6 and 3.2 mm, and are attached to a 12.7-mm diameter rod.

Vane size selection is a function of the anticipated strength of the soil and accuracy of the torque wrench. Larger vanes are typically used in soft soils and smaller vanes used in stiffer soils. While a large vane will provide better resolution than a smaller vane, it may cause more disturbance during insertion, be more difficult to rotate and thus lead to additional disturbance, or result in loads that overstress the capacity of the torque wrench.

Three parameters can be obtained from the vane shear test:

- Undrained shear strength;
- Remoulded undrained shear strength;
- Sensitivity (the ratio of 1 to 2 above).

For detailed information on in situ vane shear equipment and its use see ASTM D2573 and for analysis see Sabatini P.J, et al, 2002, Evaluation of Soil and Rock Properties.

Cone Penetration Testing

The cone penetration test (CPT) involves the hydraulic or mechanical push of an instrumented steel probe at constant rate to obtain continuous vertical profiles of stress, pressures, and/or other measurements (ASTM D5778). No borehole, cuttings, or spoil are produced by this test. Cone resistance and side friction are measured as the standard parameters for correlation with the geotechnical properties of soil. Empirical correlations are widely used to obtain estimates of relative density, effective angle of shearing resistance (ϕ'), and stiffness. It should be borne in mind that empirical correlations are soil-type dependent, and therefore may be of limited accuracy unless backed with more direct information from testing of borehole samples or cores.

The measurement of pore water pressure during cone testing is possible with appropriate set-ups where a porous element is included in the apparatus, with an electronic pore pressure transducer mounted in a cavity behind it. There is now an awareness of the tremendous potential of this tool, especially when testing in soft, primarily cohesive, deposits.

CPT devices can range from small 1-2Tonne machines to 20Tonne truck mounted machines. Recording of data can vary from visual reading of dials on the simplest and lightest machines to the automatic electronic downloading and interpretation in the most up-to-date machines. Small light CPT machines are ideal for remote or difficult access sites. For larger projects extending over wide areas of soft ground the use of more sophisticated CPT machines can be of significant cost-benefit in terms of the quality and usefulness of the data recovered for bridge or high embankment foundation investigations. For detailed information on Cone Penetration Testing and its use see ASTM D5778 and for analysis see Sabatini P.J, et al, 2002, Evaluation of Soil and Rock Properties.

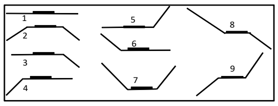
Form IV.1 Typical Walkover Sheet

ROAD CONDITION ASSESSMENT FORM

PROVINCE: ROAD NAME: TRAFFIC
DISTRICT: ROAD CODE: INSPECTOR: DATE: PAGE: 01

STRIP MAP		3		4		5		6	
Village / urban area:	Stream	F:	FLAT	IRR:	IRRIGATED LAND	RO:	ROCK	A	0 -2%
Rock Outcrop:	River	R:	ROLLING	DRY:	DRY FARMLAND	ST:	STONE	B	2 - 6%
Road:	Gully	H:	HILLY	FOR:	FOREST	GR:	GRAVEL	C	6 -8%
Path/Track:	Culvert:	M:	MOUNTAIN	SHR	SHRUB	BO:	BOG / PEAT	D	8 - 10%
Quarry	Bridge:			RES:	RESIDENTIAL	CL:	CLAY	E	10 - 15%
Gravel pit:	Outlet:					SA:	SANDY	F	>15%

7. PROFILE



Strip Map

Chainage (m)	500	1000	1500	2000	2500	3000
1. WAYPOINT NUMBER						
2. PICTURE REFERENCE No						
3. TOPOGRAPHY (F / R / H / M)						
4. LANDUSE / VEGETATION (IRR / DRY / FOR / SHR / RES)						
5. ROAD SURFACE (CL / SA / GR / LA / BIT / CO)						
6. ROAD GRADIENT						
7. PROFILE						
8. CARRIAGEWAY PROBLEMS						
9. ROAD CARRIAGEWAY WIDTH (M)						
10. OVERALL ROAD DRAINAGE CONDITION						
11. MISSING SIDE DRAIN(S) IF REQUIRED ? (L, R, L+R)						
12. MISSING OR DAMAGED CULVERT (MC/DC)						
13. CUT (C) OR FILL (F) SLIPS; (S,M,L)						
14. FLOOD VULNERABILITY (1-5)						

8. Carriageway Problems
- **1** Poor shape — a Minor
- **2** Rutting — b Moderate
- **3** Erosion — c Severe
- **4** Potholes
- **5** Low alignment

10. Road Drainage Condition
- **DR 1:** **'Good':** No water on the road during rain, no erosion, no silt, good working drainage system
- **DR 2:** **'Fair':** Some water on the road during rain, some erosion in side drains or half silted
- **DR 3:** **'Poor':** Much water on the road during rain, severe erosion/siltation of side drains
- **DR 4:** **'Bad':** Non existing / non functioning drainage system

13. Cut or Fill Slips
- C/F 1 No slips
- C/F 2 Minor slips < 3Cu M
- C/F 3 Moderate;3-10 CuM up to 25% carriageway
- C/F 4 Significant, up to 50% carriageway impacted
- C/F 5 Major 50-100% carriageway impact

14. Flood Vulnerability
1. No risk to access
2. Slight risk, little impact on access
3. Moderate risk -
4. High risk of access being compromised
5. Very high risk - severe engineering issue

Test Pits and Trenches

Test pits and trenches are used to provide access for visual in situ examination and taking samples for testing of surface soil and rock masses.

The location, frequency and depth of pits and trenches depend on the aspect of a road being investigated and the general characteristics of the project area (the soil type and variability). The DCP testing surveys can be used to target areas for pitting and trenching.

The depth of pits and trenches is determined by the nature of the subsurface. In pavement design, the depth of influence is related to the magnitude and distribution of traffic loads. Current AASHTO and many other standards limit this depth to 1.5m below the proposed subgrade level. For the purpose of sampling and description, pits should be dug to at least 0.5 m below the expected natural subgrade level. In cut sections, the depth can be reduced to 0.3 m. For upgrading and rehabilitation projects there is usually vehicular access hence pits can be excavated using a backhoe through all the existing pavement layers. In these circumstances the depth could be increased to 1.5 m below the subgrade if required, but this will rarely be necessary for such projects.

For a new alignment, the depth of any pit should not be less than 2m unless a rock stratum is encountered. Some problem conditions may require deeper exploration. Greater depths may also be needed for high embankment design. A limited number of deep pits may also be needed to ascertain groundwater influence and irregular bedrock. Great care needs to be taken in entering pits >1.0m in terms of providing adequate safety measures. These measures must be in line Myanmar or specific project Health and Safety regulations

The location of each test pit should be precisely determined on the preliminary route alignment and all layers, including topsoil, should be accurately described and their thicknesses measured. All horizons, below the topsoil should be sampled. This will promote a proper assessment of the materials excavated in cuts to be used in embankments. The samples should be taken over the full depth of the layer by taking vertical slices of materials.

It is vitally important that entrance into pits or trenches is governed by considerations of Health and Safety. It is normal practice that any pit or trench greater than 0.5-1.0m must be adequately shored or braced to prevent collapse before any person is allowed into it for inspection and sampling. Exact procedures will be governed by the Myanmar Health and Safety regulations at the time of survey.

Auguring and Boring

It is may be impossible to dig trial pits to the depth of all layers of soil or weathered rocks affected by foundation loads, such as bridge sites. Borings could also be necessary to investigate the potentially weak materials that lie below pavement layers. This is especially true in areas where thick problem soils and soft deposits exist, and when the road alignment passes through landslide zones, solution cavities, and unconsolidated soils. Geotechnical borings are then a critical component of subsurface exploration programmes. They are performed for.

- Identification of the subsurface distribution of materials with distinctive properties, including the presence and geometry of distinct layers;
- Determination of data on the characteristics of each layer by retrieving samples for use in evaluating engineering properties;
- Acquisition of groundwater data;
- Providing access for introduction of in-situ testing tools.

There is a wide range of augering or boring methods; those most likely to be used for LVRR investigations are summarised in Tables IV-1 and IV-2.

Table IV.1 Typical Augering and Non-Rotary Boring Options

Method	Procedure	Applications	Limitations / Remarks
Auger boring	Dry hole drilled with hand or power auger; samples recovered from auger flights.	In soil and soft rock; to identify geologic units above water table.	Soil and rock stratification destroyed; sample mixed with water below the water table.
Hollow-stem auger boring	Hole advanced by hollow-stem auger; soil sampled below auger.	Used in soils that would require casing to maintain an open hole for sampling.	Sample limited by larger gravel; maintaining water balance below water table is difficult.
Wash-type boring	Light chopping and strong jetting of soil; cuttings removed by circulating fluid.	Soft to stiff cohesive materials and fine to coarse granular soils.	Coarse material tends to settle to bottom of hole; Should not be where undisturbed samples are desired.
Bucket Auger boring	A 600 to 1200-mm diameter drilling bucket with cutting teeth is rotated and advanced. The bucket is retrieved and soil examined.	Most soils above water table; can dig harder soils than above types and can penetrate soils with cobbles and boulders.	Not applicable in running sands; used for obtaining large volumes of disturbed samples.
Light cable percussive or "Shell & Auger" boring	The borehole is formed using a 'clay cutter' for cohesive soils or a 'shell' (or bailer) for non-cohesive materials. A chiselling tool can be employed to penetrate very hard ground or obstructions. The sides of the borehole are supported using steel casing which is lowered into the ground as the boring proceeds.	Light rigs that may easily towed and manhandled to most road side sites. The casing is used to support the borehole sides to allow in-situ testing and sampling. In situ testing can used eg vane shear and SPT. Usable in most clay, sand and gravel materials. Rotary attachment can be used for short sections of rock.	Difficult to obtain satisfactory undisturbed samples Not to be used where good quality samples required for strength or consolidation testing unless used in conjunction with thin-walled or piston samplers

Table IV.2 Typical Rotary Boring Options

Method	Procedure	Type of sample	Applications	Limitations / Remarks
Rotary coring of rock	Outer tube with diamond (or tungsten carbide) bit on lower end rotated to cut annular hole in rock; core protected by stationary inner tube; cuttings flushed upward by drill fluid.	Rock cylinder 22 to 100 mm wide and as long as 3 m, depending on rock soundness. Standard coring size is 54 mm diameter.	To obtain continuous core in sound rock (percent of core recovered depends on fractures, rock variability, equipment, and driller skill).	Core lost in fractured or variable rock; blockage prevents drilling in badly fractured rock; dip of bedding and joint evident but not strike.
Rotary coring of rock, wire line	Stationary inner tube retrieved from outer core barrel by lifting device or “overshot” suspended on thin cable (wire line) through special large-diameter drill rods and outer core barrel.	Rock cylinder 28 to 85 mm wide and 1.5 to 3 m long.	To recover core better in fractured rock which has less tendency for caving during core removal;	Core lost in fracture or variable rock; blockage prevents drilling in badly fractured rock; dip of bedding and joint evident but not strike.
Rotary coring of swelling clay, soft rock	Similar to rotary coring of rock; swelling core retained by third inner plastic liner.	Soil cylinder 28.5 to 53.2 mm wide and 600 to 1500 mm long encased in plastic tube.	In soils and soft rocks that swell or disintegrate rapidly in air (protected by plastic tube)	Sample smaller; equipment more complex than other soil sampling techniques.

The Standard Penetration Test (SPT)

Standard Penetration Test (SPT) procedures are normally used in conjunction with augering or boring operations and consist of repeatedly dropping a 63.5-kg hammer from a height of 760 mm to drive a split-barrel sampler (or solid cone) three successive 150-mm long increments. The number of blows required to drive the sampler is recorded for each 150-mm increment. The initial 150-mm increment is considered a seating drive. The blows required for the second and third 150-mm increments are totalled to provide blows/300 mm. This total is referred to as the as the SPT resistance or "N-value".

The SPT is highly dependent upon the equipment and operator performing the test, and it is often difficult to obtain repeatable results. Nevertheless, long-standing correlations have been established between the "N" value and the engineering properties of solid and weak rocks and it is a commonly used tool for determining in situ ground conditions when used in appropriate conditions.. For LVRR investigation it most likely that ground investigations using SPTs would be used in bridge site investigations or in areas of deep cut or high embankment.

For detailed information on Standard Penetration Testing and its use see ASTM D6338, Weltman and Head, 1980 and for analysis see Sabatini P.J, et al, 2002,Weltman and Head, 1983.

Geophysics: Seismic Refraction

Geophysical testing is often used as part of the initial site exploration phase of a project and/or to provide supplementary information collected by widely-spaced observations (i.e., borings, test pits, outcrops). Geophysical testing can be used for establishing stratification of subsurface materials, the profile of the top of bedrock, depth to groundwater, limits of types of soil deposits, rippability of hard soil and rock, and the presence of voids, buried pipes, and depths of existing foundations. Data from geophysical testing should always be correlated with information from direct methods of exploration.

In the seismic refraction method an impact load is applied to the ground surface by either a small explosive charge or use of large hammer impacting on a steel plate. Seismic energy refracts off soil/rock layer interfaces and is recorded on the ground surface using several dozen geophones positioned along a line or performing repeated events using a single geophone.

Seismic refraction provide very useful information for LVRRs at low cost on:

- Depth to bedrock;
- Depth to water table;
- Thickness and relative stiffness soil/rock layers.

The procedure does not work if strength or stiffness decreases with depth or if soft layer underlies a stronger layer. It works best when there is sharp difference between layers, for example soft clay over strong bedrock.

Visual Descriptions and Estimations

Systematic visual descriptions of soil and rock materials are an essential part of any LVRR ground or materials investigation. Accurate descriptions are vitally important in low-cost investigations in the assessment of soil-rock characteristics and extrapolation of engineering properties and performance from previous experience.

If soil-rock testing facilities are not available estimations of geotechnical properties can be made, based on visual description. Tables IV.3, IV.4 and IV.5 may be used for determining approximate characteristics of in situ rock or soil material.

Detailed procedures for undertaking and presenting visual descriptions and assessments are contained in ASTM, D 2487 and 2488, Practice for Classification, Description and Identification of Soils, Norbury (2010) provides comprehensive guidance on the description and classification of soils and rocks.

Table IV.3 Rock Bearing Capacity

Action on Rock Sample	Rock strength	Allowable bearing capacity (kN/m²)	Uniaxial compressive strength (MPa)
A hammer blow required to break specimen, can be scratched with firm pressure from knife	Strong	10 000	50 - 100+
Easily broken with hammer, can be easily scratched with knife and pick end indents approx. 5mm	Moderately strong	2000	12.5 - 50
Broken in hand by hitting with hammer, can be grooved 2mm deep with a knife	Moderately weak	1000	5.0 - 12.5
Broken by leaning on sample with a hammer, can be grooved or gouged easily with a knife	Weak	750	1.25 - 5.0
Can be broken by hand and knife will penetrate approx. 5mm	Very weak	250	0.6 - 1.25

Table IV.4 Clays and Silts Strength

Action on Soil Sample	Strength Description	Allowable bearing capacity (kN/m²)	Undrained shear strength (kN/m²)
A thumb nail will not indent the soil	Hard	600	300+
Indented by a thumb nail, penetrated about 15mm with a knife	Very stiff	300	150 - 300
Indented by a thumb with effort, cannot be moulded by fingers	Stiff	150	75 - 150
Penetrated by thumb with pressure, moulded with strong finger pressure	Firm	75	40 - 75
Easily penetrated by thumb, moulded by light finger pressure	Soft	25	20 - 40
Extrudes between fingers when squeezed in hand	Very soft	0	< 20

Note: Dry weather visual assessment is certainly no indication of likely wet season performance.

Table IV.5 Sands and gravels bearing capacity

Action	Strength	Allowable bearing capacity (kN/m^2)	Standard penetration test N-Value
High resistance to repeated blows with a pick	Very dense	500	>50
Requires pick for excavation, a 50mm diameter peg is hard to drive in	Dense	300	30 - 50
Considerable resistance to penetration by sharp end of pick	Medium dense	100	10 - 30
Can be excavated by spade, a 50mm peg is easily driven, can be crushed between fingers	Loose	50	5 - 10
Crumbles very easily when scraped with a pick	Very loose	Negligible	<5

References

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ASTM D5778 Standard Test Method for Electronic Friction Cone and Piezocone Penetration., American Society for Testing Materials.

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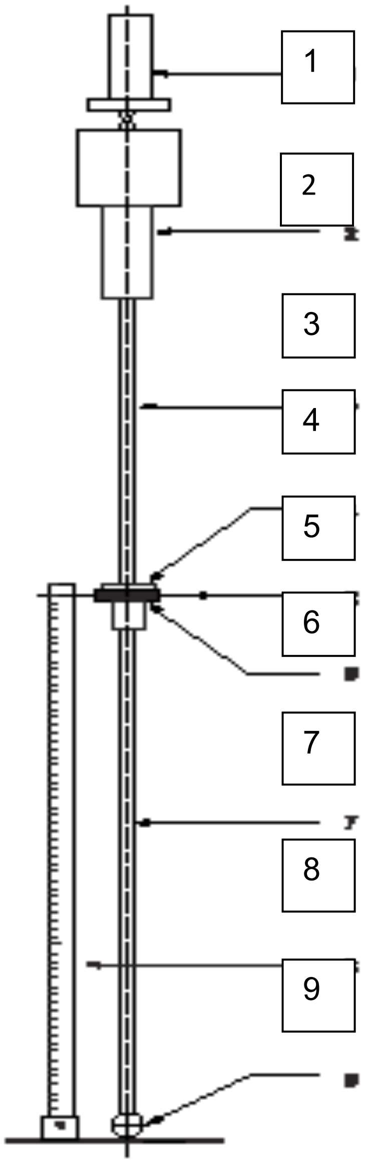
ANNEX V. DCP DESIGN OPTIONS

Introduction

The DCP is an instrument designed for the rapid in-situ measurement of the structural properties of existing road pavements constructed with unbound materials (Figure V.1). Continuous strength measurements can be made down to a depth of approximately 800mm or, when extension shafts are used to a recommended maximum depth of around 1.60m metres. Where pavement layers have different strengths the boundaries can be identified and the thickness of the layers determined. For LVRR alignment subgrade investigations the DCP tests would normally be taken every 200-250m (2 at each chainage sunk at least 700mm or refusal). This spacing may be increased to 500m in cases where there is no change in terrain, earthworks or general environment. Samples should be taken for examination and possible testing at DCP section locations.

Figure V.1 The Assembled DCP

1. Handle
2. 8kg Hammer
3. Hammer shaft
4. Coupling
5. Handguard
6. Clamp ring
7. Standard shaft
8. 1m rule
9. 60 degree cone



Apparatus

Correlations have been established between DCP measurements and CBR (California Bearing Ratio) so that results can be interpreted and compared with CBR specifications for pavement design. A typical DCP test takes only a few minutes and therefore the instrument provides a very efficient method of obtaining information.

The design of the DCP uses an 8Kg weight dropping through a height of 575mm and a 60^0. cone having a diameter of 20mm.

After assembly, the first task is to record the zero reading of the instrument. This is done by standing the DCP on a hard surface checking that it is vertical and then entering the zero reading in the appropriate place on the test sheet (FigureV.2).

The DCP needs three operators, one to hold the instrument, one to raise and drop the weight and one to record the results. The instrument is held vertically with, the weight just touching the handle, but not lifting the instrument. The operator then lets it fall freely. If during the test the DCP leaves the vertical, no attempt should be made to correct this as contact between the bottom shaft and the sides of the hole will give rise to erroneous results.

It is normal practice to take a reading after a set number of blows. It is therefore necessary to change the number of blows between readings according to the strength of the layer being penetrated. For good quality granular bases readings every 5 or 10 blows are normally satisfactory but for the weaker sub-base layers and sub-grade readings every 1 or 2 blows may be appropriate. There is no disadvantage in taking too many readings, but if too few are taken, weak spots may be missed and it will be more difficult to identify layer boundaries accurately hence important information will be lost.

Little difficulty is normally experienced with the penetration of most types of granular materials. It is more difficult to penetrate granular materials with large particles and very dense, high quality crushed stone. The instrument has been designed for strong materials and therefore the operator should persevere with the test. Penetration rates as low as 0.5mm/blow are acceptable but if there is no measurable penetration after 20 consecutive blows it can be assumed that the DCP will not penetrate the materials. If only occasional difficulties are experienced in penetrating granular materials it is worthwhile repeating any failed tests a short distance away from the original test point.

If the DCP is used extensively for hard materials, wear on the cone itself will be accelerated. The cone is a replaceable item and it is recommended by many authorities that replacement be made when the diameter has reduced by 10 percent. However other causes of wear can also occur hence the cone should be inspected before every test. Typically, the cone will need replacing after about 10 holes in hard material and in the absence of damage other than shoulder wear this is the recommended practice

The results of the DCP test are usually recorded on the field test and the results can then either be interpreted by hand calculator or transferred to a standard EXCEL-type spread-sheet and processed by computer, Figure V.3. Alternatively, there is available a DFID funded TRL computer programme that can now be used to calculate not only layer depths and CBRs but other related relationships and plots[1]

The boundaries between layers are easily identified by the change in the rate of penetration. The thickness of the layers can usually be obtained to within 10mm except where it is necessary to core (or drill holes) through materials to obtain access to the lower layers. In these circumstances the top few millimetres of the underlying layer is often disturbed slightly and appears weaker than normal.

Several similar relationships between the DCP readings and CBR have been obtained; the one currently used by the TRL is as follows:

TRL, Overseas Road Note 18 (60^o cone) $Log_{10}(CBR) = 2.480 – 1.057\ Log_{10}(mm/blow)$

Agreement is generally good over most of the range but differences are apparent at low values of CBR, especially for fine grained materials. It should be remembered that DCP-CBR figure refers to specific index strength for specific in situ conditions of moisture and density and great care needs to be taken in relating this to laboratory based CBR values. Therefore, if precise values are needed, it is advisable to calibrate the DCP for the materials in question. Nevertheless, if the testing is undertaken at worst case soaked (rainy season) conditions it will give a reasonable representative picture of existing actual pavement or sub-grade strength conditions.

The DCP-CBR Design Process

Whilst TRL, Overseas Road Note 18 provide guidance on undertaking the test, the UK DCP 3.1 User Manual (TRL, 2006) provides a comprehensive guide to the application of the UK DCP software in the design of pavements. The UK-DCP manual provides step-by-step guidance on using the pavement design functions of the software with the aid of appropriate screen-grabs, Table V.1.

Although this document provides a comprehensive guide to a design process, parts of it may utilised merely to edit, store and report DCP data for simpler design approaches.

It has to be emphasised that the DCP pavement design approach is not relevant in situations of cut or embankment greater than about 1.5m deep; that is situations where the tested DCP profile is not within the proposed pavement's zone of influence.

[1] This programme together with the User Manual may be downloaded via https://www.gov.uk/dfid-research-outputs/uk-dynamic-cone-penetrometer-dcp-software-version-3-1.

Figure V.2 Standard DCP Field Sheet

SITE/ROAD	DATE
	TEST NO
SECTION NO/CHAINAGE	DCP ZERO READING mm
DIRECTION	TEST STARTED AT
WHEEL PATH	

No OF BLOWS	TOTAL BLOWS	READING mm	No OF BLOWS	TOTAL BLOWS	READING mm	No OF BLOWS	TOTAL BLOWS	READING mm

Figure V.3 Typical Worked Sheet

HUE DCP FIELD SHEET

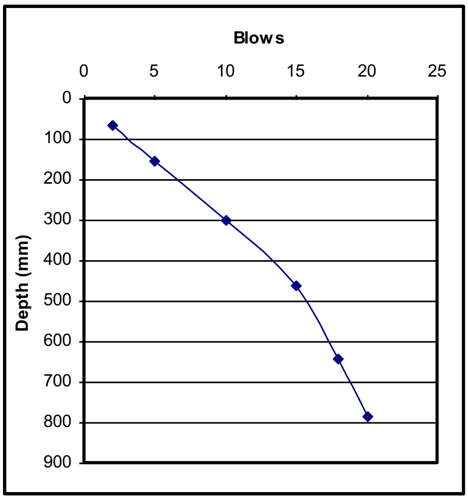
Site/Road	Phu Loc Road	Date	19/11/2002
Test No.	PL.07	Operator	Pham Gia Tuan
Site Location		Zero Reading (C0)	107.0
Test Location	RS	Depth of Start	0.0

No.Blows	Total Blows	Total Penetration (mm)	Total Corrected Penetration	ΔPen	Pen/blow	LogP	Numeric factor	CBR
a	b	c	d	e	f	g	h	j
		107						
2	2	174	67	67	33.5	1.5250	0.8680	7.4
3	5	259	152	85	28.33333	1.4523	0.9449	8.8
5	10	406	299	147	29.4	1.4683	0.9280	8.5
5	15	570	463	164	32.8	1.5159	0.8777	7.5
3	18	750	643	180	60.0	1.7782	0.6005	4.0
2	20	890	783	140	70.0	1.8451	0.5297	3.4

Formulas for Excel

a	b	c	d	e	f	g	h	i
Input	$b_n=a_n+a_{n-1}$	Input	$d_n=c_n-c_0$	$e_n=d_n-d_{n-1}$	f = e/a	$g=\log_{10}(f)$	h=2.48-1.057*g	i=10*h

Figure V.4 Typical EXCEL Calculation Sheet and Plots for DCP Data



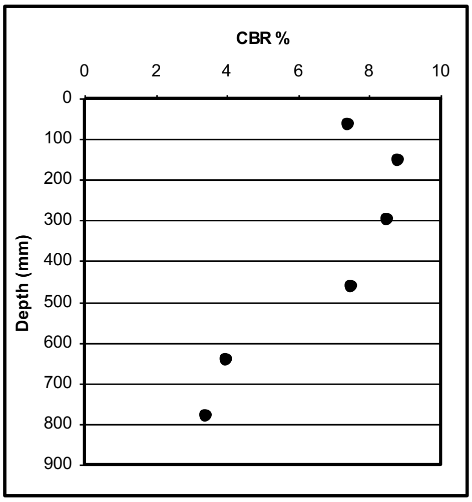


Figure V.5 Contents of the UK DCP Manual

Chapter	Content
1. Introduction	Installation Obtain and install UK DCP 3.1
2. Start up	Run UK DCP 3.1 and open a new or existing project. The term 'project' refers to a set of related sites, at each of which a penetration test has been carried out and which will be analysed together. In normal use, a project will be a single road or a shorter length of uniform construction.
3. Test data input	Input site details and penetration data for the tests within a project
4. Layer analysis	Analyse the penetration data from a test to identify and determine the thicknesses of the distinct Test layers within the pavement .Penetration data can be analysed manually or automatically
5. Structural Number calculation	Assign the Test layers to specific pavement layers and calculate the Structural Number of each pavement layer.
6. Query	Produce histograms of strengths and pavement layer thicknesses along the project
7. Sectioning	Divide the project into sections which are uniform in thickness and/or strength.
8. Design data input	Input road condition, structures condition, surface gravel thickness, crown height details, road geometry, land use, design standard, costs and traffic details of a project
9. Design Sections	Divide the project into sections which are uniform in a variety of characteristics.
10. Pavement Design	Design the pavement improvement for a low volume road.
11. Reporting	Produce reports of the analysis and design process for printing and/or export.

The DCP-DN Design Process

Within the last decade an alternate design process has been developed using the DCP penetration rate (blows/mm) as a direct design index without recourse to the laboratory CBR. The procedure is based on work primarily on the soils of Southern Africa within the context of upgrading gravel roads to sealed roads. In this context it offers savings in pavement layers thicknesses within defined parameters.

In general, based on assessment of performance information to date (ReCAP, 2019), the DCP-DN method, within appropriate road environments, is the most cost-effective design option at relatively low traffic, up to about 0.7 million esa and across all subgrade strengths. However, above 0.7 million esa the method gradually becomes less cost effective than the other methods, particularly ORN31 (TRL, 1993), which become more cost-effective in many situations.

The current recommendation from DFID and ReCAP is that this DCP-DN procedure requires further correlation work with the distinct road environments of Myanmar before it is adopted for LVRR design work by DRRD.

Recent Design Guidance

Recent ReCAP research has produced additional guidance on selection pavement design options and this includes the use of the DCP as an aid to the design of LVRRs ranging from earth roads, through gravel to sealed pavements (ReCAP, 2020).

References

ReCAP, 2019. Evaluation of Cost-Effectiveness and Value-for-Money of DCP-DN Pavement Design Method for Low-Volume Roads in Comparison with Conventional Designs Project Reference Number. RAF 2128A

ASTM D6951-03, Standard Test Method for Use of the Dynamic Cone Penetrometer in Shallow Pavement Applications

Kleyn E.G.& van Zyl G.D., 1988. Application of the DCP to Light Pavement Design. 1st Symposium on Penetration Testing, Orlando USA.

ReCAP, 2020. A Guide on the Application of Pavement Design Methods for Low Volume Rural Roads. Draft Report by TRL LTd ReCAP ref No. GEN 2166B.

TRL , 2006. UK DCP 3.1 User Manual. Measuring Road Pavement Strength and Designing Low Volume Sealed Roads using the Dynamic Cone Penetrometer. Project Report UPR/IE/76/06 Project Record No R7783. https://uk-dcp.software.informer.com/3.1/

TRL, 1999. ORN 18. A guide to the pavement evaluation and maintenance of bitumen-surfaced roads in tropical and sub-tropical countries. TRL Ltd, for DfID, UK.

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ANNEX VI. SPOT IMPROVEMENT

Introduction

When funds are limited they should be used to improve sites which do not currently provide safe and reliable access, for example a badly degraded or flooded section. Sites which do provide this level of access could be left without improvement, for example a length of track which is not eroding, slippery or damaged, allowing the funds to be used to improve access on other roads. Sites which are improved in such a way are referred to as 'spot improvements'.

The spot improvement approach allows more road kilometres to be improved and therefore development benefits to be spread more effectively to maintain or improve network connectivity as whole than if the same funds were used for full rehabilitation on a single road.

The spot improvement approach is a very efficient form of engineering design as each section of the road is designed according to its specific conditions. The road therefore meets the needs of the traffic and the road environment with minimal wasted effort and cost.

Spot Improvement Selection

The choice of spot improvement should be based on the location features and the materials and skills available locally. Great care should be used in using gravel as a road surface in some circumstances. It is unlikely that it will be most suitable option in some locations due to high costs of routine maintenance and periodic replenishment of the surface material.

Spot Improvement concerns all the road assets and can involve one or of a number of options, Table VI.1.It is important to note that Spot Improvement differs from periodic maintenance in that it is an **engineering upgrade** aimed at addressing the fundamental engineering issues that are causing or likely to cause an access problem; it is **not** a rehabilitation or repair aimed restoring the status quo.

Table VI.1 Typical Spot Improvements

Spot	Improvement
Pavement	Existing pavement sections are upgraded, typically from unsealed to sealed, using the options and approaches outlined in Chapter 7
Drainage	Side drainage may be constructed where there was none before, or improved – for example from un-lined to lined. Additional culverts may be constructed where none existed; or enlarged where they are proving inadequate.
Earthworks	Earthwork slopes may be adjusted or their erosion resilience may be upgraded by means of bio-engineering allied to low cost engineering works.
Bridge/Causeway	Spot Improvement of stream or river crossings can be a major contributor to network connectivity upgrade; for example the upgrade of trail bridges for motor-vehicle access.
Safety Issues	Spot Improvement can also be applied to the construction of safety features; for example safety barriers on mountain roads.

Spot Improvement can be closely linked to the strengthening of LVRR links for increased Climate Resilience, Table VI.2.

Table VI.2 Climate Resilience Spot Improvement Options

Ref.	Adaptation Group	Comment
1	Pavement strengthening	Usually required for steep (>8-10%) gradients on unsealed roads, within village area and areas subject to erosive flood not mitigated by raised alignment.
2	Pavement and earthwork drainage	Lined drains likely required where gradients >6%. Additional side drains and associated turn-outs may also be recommended. Drainage required above slopes and on earthwork benches.
3	Cross drainage	Bridge, causeways and culvert designs adjusted to take account of forecast increased river/stream flows and storm intensities. Additional cross culverts recommended where considered essential to improve overall road drainage, as relief culverts for example on steep sections. Occasional existing fords or low-level bridges might be replaced by climate resilient structures such as vented fords, or submergible multiple culverts.
4	Alignment	Horizontal alignment may be shift to avoid high climate vulnerable sections. Vertical alignment rising of earth embankments will be recommended where the alignment is too low and is being impacted by flooding and/or the weakening of the pavement by saturation.
5	Earthwork slope protection	Protection where erosion of exposed soil or rock slopes either in cut or embankment is identified are at significant risk from climate impact.
6	River/stream bank protection	May be recommended where erosion of the alignment by rivers or streams is identified as a significant risk from project future flows and floods.
7	Bioengineering	An option group that is cross-cutting over the range vulnerable road network assets

The planning of Spot Improvement, because of its very nature, is closely linked to a process of prioritisation. The process of identification and prioritisation is outlined in SEACAP (2009) and Table VI.3 presents a typical ranking from this Spot Improvement Manual.

Table VI.3 Typical ranked spot selection criteria

Priority criteria	Description
1 Unsafe – high risk	Safety concerns put road users or others at high risk of injury or death.
2 Impassable at any time	Road users are unable to pass along the road at any time of the year.
3 Impassable in wet season only	Road users are unable to pass along the road in the wet season, although closures up to 24 hours after rainfall are accepted.
4 Unstable slope	The slopes above or below the road are unstable and at risk of slipping.
5.Condition likely to deteriorate	Vehicles or rainfall are likely to cause significant deterioration of the road in the next year.
6 Health risk	The health of road users and others is at risk, typically due to dust from a gravel road.
7 Drainage in poor condition	Drainage capacity or performance is reduced and retained water is likely to damage the road.
8. Unsafe – medium risk	Safety concerns put road users or others at medium risk of injury.
9 Environmental concerns	Construction or future usage may cause environmental concerns along the road such as erosion of bare soil, disruption of a water course or contamination of a water supply.
10 Very slow travel	Vehicles travel very slowly along the road due to its poor condition.
11 Geometric cross section below standard	The width and camber of the carriageway and shoulders do not meet the required standard.
12 Geometry below standard	The curvature, sight distance or gradient of the road do not meet the required standard.
13 Surface below standard	The surface is dusty, slippery or gravel on a steep hill.
14 Pavement below standard	The pavement although passable does not meet the required design specifications.

Implementation Issues

There are some potential challenges to a spot improvement approach which must be addressed.

1. The Spot Improvement approach requires that spot-sites must be identified and prioritised on rational basis. This may not be an easy task and can require significant engineering judgments as well as knowledge of local conditions. Appropriate training and guidance will be required if this is to undertaken by local DRRD staff.
2. Secondly, there may be resistance to the approach from local communities who may regard an apparently "unfinished" road as a consequence of poor management, bad contracting practice or corruption. Cooperation with local communities in the selection of spot-sites and on-going involvement of local stakeholders in the road rehabilitation programme will do much to allay these fears.
3. There has in some instances been some confusion of Spot Improvement with periodic maintenance. Spot Improvements must be seen and designed as fully engineered responses to defined requirements and not as repairs of existing designs that may just perpetuate an underlying problem.

References

SEACAP, 2009. Low Volume Rural Road Environmentally Optimised Design Manual. SEACAP 3.02 manual for MPWT, Laos.

ANNEX VII. LVRR DESIGN WITH HIGHER VOLUME TRAFFIC

Introduction

For pavement design purposes, a low-volume rural road (LVRR) has been defined in this Manual as one designed to carry a cumulative traffic loading of up to about 1 Mesa (or around 300 AADT). This follows general regional practice (SEACAP, 2008; 2009b; JKR Malaysia, 2012; Cook et al, 2013) and is line with the defined requirements of the NSRAA and the 3-fold classification of LVRRs in Myanmar (GoM, 2017).

It is recognised that DRRD may have a responsibility for the design of roads where the traffic is above the LVRR limit. This Annex presents and discusses the issues that arise when considering roads in the 300-750 AADT/1-3Mesa envelopes. The discussion is focussed on pavement issues, although it is recognised that other road assets such as bridges, culverts and earthworks may be impacted by a higher level of traffic.

The Rationale for LVRR Limits

The definition of what comprises an upper limit of "Low Volume" traffic varies internationally; some countries use 200 AADT, others as high as 500 AADT, whilst others consider only "commercial vehicles". Giummarra (ARRB, 2001) provides a comparison of AADT and their geometry (Appendix A). This was taken further forward in SEACAP in their 2009 review of LVRR carriageway geometry, as summarised in Figure 6.2 in Chapter 6 of this Manual (TRL, 2009).

The concept of low volume road pavement design being different from "normal" pavement design approaches (e.g. TRL, ORN 31) has been driven by the recognition that significant savings could be made in terms of pavement layer thickness, surfacing type and relaxed material specifications at low traffic volumes of light traffic. Underpinning this concept was the recognition of the different modes of deterioration between high and low volume roads as shown in Figure 3.3 in this Manual (this derived from a diagram by Rolt as shown in Cook, Rolt and Petts, 2013).

They key point from the above is that the division between low volume and high volume roads is science/engineering based and <u>not administration or management based</u>. On current evidence, although DRRD on administrative ground may well have to consider design issues for higher volume rural roads, this is a not valid reason for including them within the LVRR design process without careful consideration. In general, roads with traffic greater than 1Mesa cannot be adequately dealt within under LVRR guidance as it stands and the more traditional approach should be considered in line with a guidance given by such documents as ORN 31 (TRL, 1993).

Recent Research

Recent and ongoing research funded by DFID (Table VII.1) has indicated that in some circumstances, particularly in drier road environments, the natural materials from which low volume sealed roads are made can carry well in excess of 3 Mesa and that future adjustments could be made in methods of pavement layer design and their constituent materials. Outcomes from this work are incorporated in the following sections of this Annex.

Table VII.1 Recent ReCAP Research on LVRR Design

ReCAP Project	Key Objective	Outputs
Development of Guidelines and Specifications for Low Volume Sealed Roads through Back Analysis. ReCAP Ref. RAF2069A.	Refine existing catalogues for pavement design of sealed LVRRs. Provide a base level for information on material specifications in comparison with conventional designs and specifications for roads carrying >300 ADT.	Draft Report (2019) submitted that includes deign charts and material guidance on pavement design up to 3Mesa
A Guide on the Application of Pavement Design Methods for Low Volume Rural Roads. ReCAP Ref GEN 2166B.	To provide background and guidance on pavement design methods used in the design of LVRRs ranging from earth roads, through gravel to the various unbound, natural stone, bituminous, cement-based and clay brick surfacing and pavement layers.	Draft Report (2020) submitted that includes advice on application of various pavement design methods for LVRR pavements up to 3Mesa

Implications and Recommendations

For pavement design the implications of traffic being outside the LVRR upper limits are primarily in terms of:

Carriageway geometry: This Manual works within the existing NSRAA recommendations on carriageway width; that is single lane with a maximum width of 5.5m. Traffic significantly above 300 AADT/<1Mesa may require a double lane. In this case the designer should work within Myanmar main in road guidelines (e.g. MoC, 2015). The general principles of design speed, radii of curvature, and super-elevation, remain as noted in Chapter 6 of this Manual. However, the detail may need to be reworked for higher volume traffic, particularly if the design involves a shift to a 2-lane carriageway.

Pavement and surfacing options: A number of the options discussed in Chapter 7 would not be suitable for higher volume (1-3Mesa) traffic. Table VII.2.

Table VII.2 Suitability of LVRR Options for Higher Volume Traffic (1-3Mesa)

Option	Suitability		
	Yes(Y)	Possible (P)	No (N)
DBST	Y. DBST only		
Sand Seal			N
Otta Seal	Y		
Slurry Seal			N
Cape Seal	Y		
Penetration Macadam			N
Pre-Mix	Y		
ENS			N
Unsealed gravel			N
WBM/DBM		P. Under a good seal	
Hand Pack Stone			N
Block Stone			N
Brick/Block			N
Laterite		P, if good quality	
Graded Crushed Stone	Y		
Stabilised Soil	Y		
Concrete Slabs	Y Reinforced		
Concrete Cells		P. Unproven	

Structural thickness: This design charts in this, and other, LVRR Manuals have a limit of 1 Mesa. An increase in traffic may require the designer to use alternative design charts for example Table VII.3, based on ORN 31 (TRL 1993). Alternatively, Table VII.4 is based on draft recommendations in ReCAP, 2019.

Material Specifications: Most of the reductions or flexibility in materials specification valid for LVRRs (as discussed in Annex III) could not be recommended for higher volume roads without detailed investigation. This includes, for example, the reduction in base aggregate strength below 80% CBR. The designer should refer to Main Road Specifications.

Table VII.3 Thin Bituminous Pavement Design Chart for Structural Layers (mm) up to 3.0 Mesa

ORN 31 Traffic range (Mesa)		T3	T4
Subgrade class (CBR)	**Layer**	**0.7 - 1.5**	**1.5 - 3.0**
S1 (2%)	Base	200 G80	200 G80
	Sub-Base	200 G30	250 G30
	Subgrade	300 G15	300 G15
S2 (3-4%)	Base	200 G80	200. G80
	Sub-Base	200.G30	225.G30
	Subgrade	200.G15	200.G15
S3 (5-7%)	Base	200. G80	200. G80
	Sub-Base	225.G30	275.G30
S4 (8-14%)	Base	200. G80	200. G80
	Sub-Base	150.G30	200.G30
S5 (15-29%)	Base	175. G80	200. G80
	Sub-Base	100.G30	125.G30
S6 (>30%)	Base	175. G80	200. G80

Note: This chart extracted from ORN 31

Table VII.4 Revised Design Table for Thin Sealed LVRRs with axles < 8 tonnes

Subgrade Class	Layer	Traffic (mesa): Axle loads < 8 tonnes				
		<0.1	0.1-0.3	0.3-0.5	0.5-1.0	1.0-3.0
S1 and S2 CBR ≤4%	Base	150 G45	150 G45	150 G60	150 G60	150 G60
	Sub-Base	125 G25	125 G25	125 G25	150 G25	150 G25
	Subgrade	125 G15	125 G15	150 G15	150 G15	150 G15
S3 and S4, CBR 5-14%	Base	125 G45	125 G45	150 G60	150 G60	150 G60
	Sub-Base	125 G25	125 G25	125 G25	125 G25	150 G25
	Subgrade	125 G15	125 G15	100 G15	100 G15	125 G15
S5 and S6 15-CBR>30%	Base	175 G45	175 G45	175 G45	175 G45	200 G45

Note: Extracted from (TRL, 2019, Draft Report)

Increases in Traffic Volume and Types

Traffic may exceed the LVRR limits in a number of ways with correspondingly different design implications. The following examples are based on the traffic data from the DRRD Taungyi pavement trials and show the implications of the increase of different types of vehicle on the pavement design.

A. Numbers of vehicles remain roughly similar but there is an increase percentage of heavier vehicles and/or the loads they carry.

B. General overall rise traffic

C. Large number of light agricultural type vehicles

D. Increase in pedestrians and NMT

An analysis of typical traffic changes (A to D) is summarised in Table VII.5

Table VII.5 Examples Changes in Traffic

Traffic Type		3-Day Traffic Counts				
		Example from Annex I	Increased Traffic Example Models			
			A	B	C	D
A	Minibuses	10	10	30	5	10
B	Light Trucks <5 t	10	25	40	5	10
C	Heavy Trucks >11t	5	35	30	3	5
D	Very Heavy Trucks (13t)	1	10	5	0	1
E	Pickups	3	3	20	3	3
F	SUV/4WD	9	3	30	9	9
G	Cars/Saloon	4	2	25	4	4
H	Motorcycle trailers	10	10	25	50	10
J	Power tillers	50	50	100	200	95
K	Agriculture tractor	1	1	10	30	1
L	Motorcycles	150	325	500	700	350
M	Total Motorized Vehicles	253	474	815	1009	498
N	Bicycles	25	25	50	100	1000
O	Ox carts	5	5	10	25	75
P	Total Non-Motorized Vehicles	30	30	60	125	1075
Q	Pedestrian	120	120	250	250	1500
	AADT Year 1	129	255	476	276	255
	AADT after 8 years	194	383	714	405	383
	PCU Year 1	398	580	1110	1396	1957
	PCU after 8 years	597	870	1665	2094	2935
	Cum esa 12 year design life	238,272	1,315,460	1,772,148	213,452	253,164

To exemplify the impacts of changing traffic Table VII.6 presents the examples of higher traffic models (A to B) in terms of their implications.

Table VII.6 Increased Traffic Implications

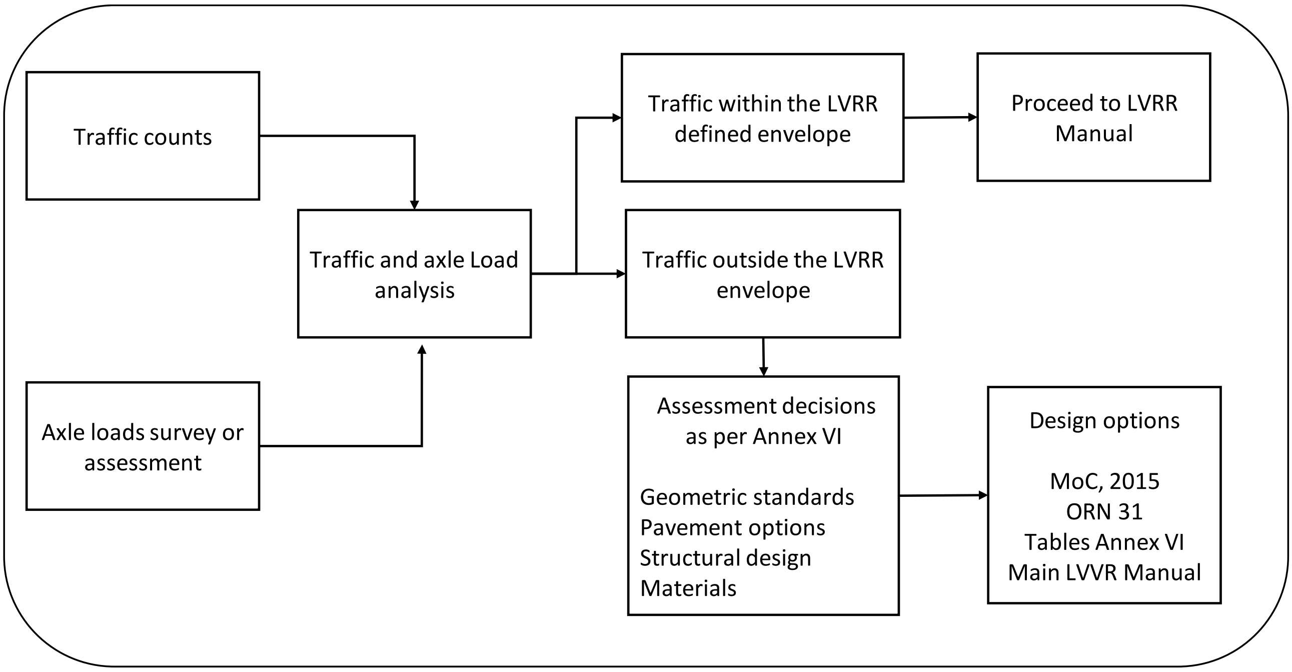
Design Issues	Model A	Model B	Model C	Model D
Carriageway geometry	A single lane LVRR geometry could be possible with passing places and climbing lanes (on steep grades)	LVRR single lane geometry not possible.	Single lane LVRR geometry possible.	Single lane LVRR geometry but requires wide shoulders for NMT
Pavement and surfacing options.	Limited as per Table VI.2.	Limited as per Table VI.2.	LVRR Manual applicable	LVRR Manual applicable
Structural thickness	Outside LVRR designs – see ORN 31 and Tables VI.3 and VI.4	Outside LVRR designs – see Tables VI.3 and VI.4	LVRR Manual applicable	LVRR Manual applicable
Material specifications	No scope for relaxation without specific investigation	No scope for relaxation	LVRR Manual applicable	LVRR Manual applicable

Summary

When considering the design of a LVRR pavement carrying traffic greater than the limits defined for the Manual it is necessary for the designer to consider in detail the traffic types and axle loading and then decide which, if any, elements of the Manual may be used Figure VII.1.

Some guidance is given in this document on pavement structural design up to 3Mesa, otherwise the designer may have to reference either the Myanmar main road documentation or ORN 31.

Figure VII.1 Higher Volume Traffic and the Use of the LVRR Manual



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