CHARPTER III

CASE STUDY OF THE SARABURI-LOMSAK HIGHWAY

General Description

The Saraburi - Lomsak Highway is situated along the Pa Sak River Valley. Its construction was financed jointly by the Thai Highway Department and a loan from the International Bank for Reconstruction and Development (World Bank). The project extends 282 kilometers from the city of Saraburi (108 kilometers north of Bangkok), through Lamnarai, Wang Chomphu and Petchabun, to Lomsak as shown in Fig.3. The project lies between latitude 14° 30′ North and 17° 00′ North ; its longitude is approximately 101° 00′ East. A general description of the route and the territory it serves is given by PUMPKHEM (1975). For final bidding and the awarding of construction contracts, the project was divided into four nearly equal sections. Only Contract Section I, which is 66.6 kilometers in length, has been used to test the RTIM. Section I is at elevation of 15 to 145 meters above sea level, in a region where the mean annual rainfall averages about 1460 mm.

From Saraburi to Ban Phu Kae, the highway extends along the east side of the Chao Phraya River Valley in the Central Plain region. This great valley is filled with a vast quantity of mud and silt that was deposited by the river during the latest geologic epoch. North of Ban Phu Kae, the highway crosses an area of rolling land and hills underlain by paleozoic

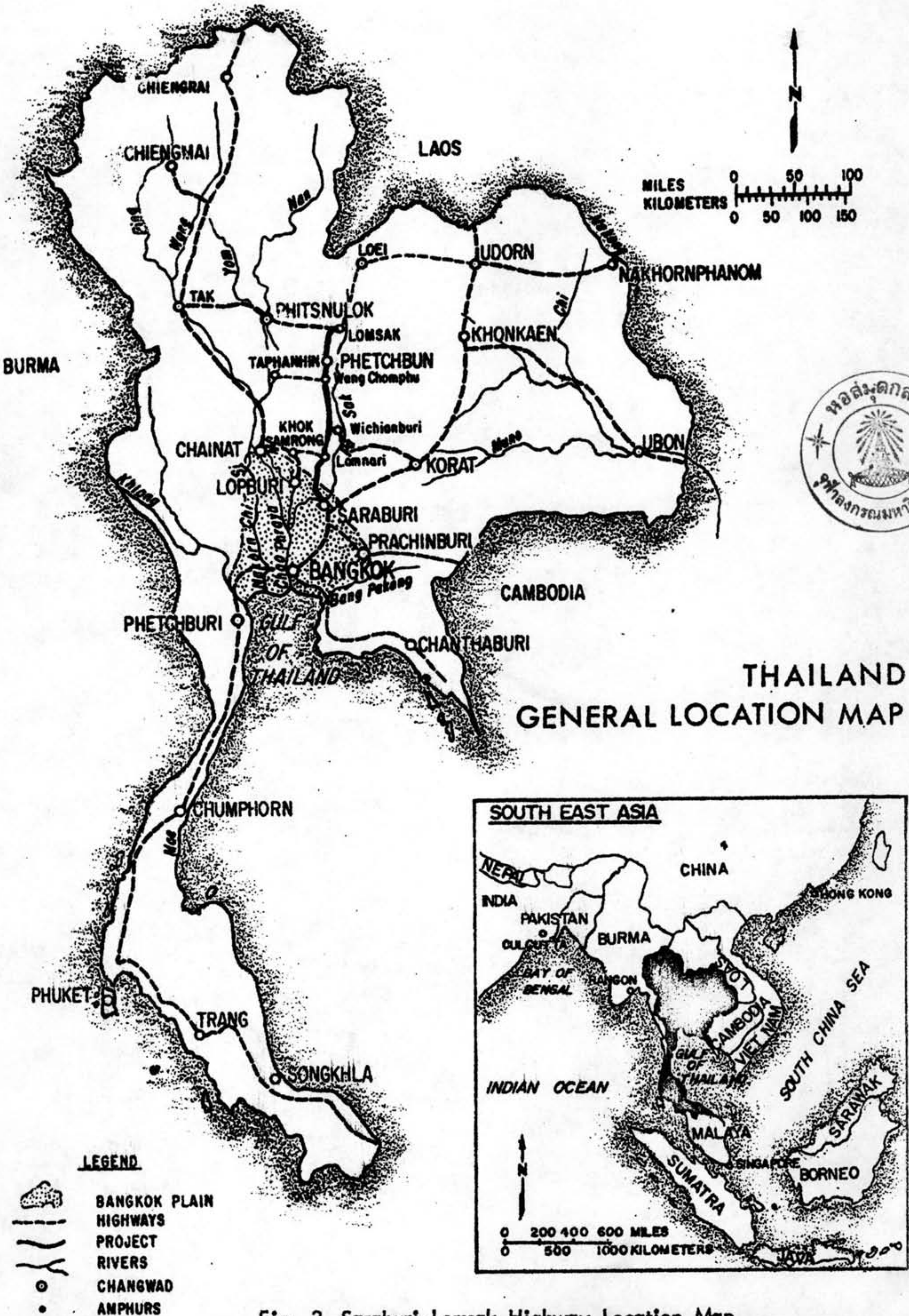


Fig. 3 Saraburi-Lomsak Highway Location Map

sandstone, shale, and limestone. These rocks have been eroded almost to the level of the present valley, leaving only isolated hills which are mainly composed of limestone. The soil above bedrock is only two or three meters thick in some locations. In general, the soils of the Pa Sak Valley are clay and silt, ranging up to as much as 30 meters in thickness. As a construction material, these soils were generally only suitable for use for embankments. At some localities, such as the area just north of Saraburi, accumulations of very unstable, carbonaceous mud were encountered. It was necessary to remove much of this material prior to placing the highway embankments.

Section I starts at the Pa Sak River Bridge in Saraburi as a six-lane roadway; it continues for 0.8 kilometers. From that point it becomes a four-lane roadway for an additional 3.0 kilometers. This section was first constructed as a two-lane road and subsequently modified prior to completion of the project. The new construction then separates from the existing road to form a four-lane divided highway for the next 11.0 kilometers. The old road is used for the northbound lanes in this segment. The northern end of this section is at the junction at which the new road leaves the existing Saraburi-Lopburi road and traverses hilly terrain on new alignment for 51.0 kilometers to a junction with Route 205 where Contract Section II begins.

The section I contract consisted of the construction of 66.6 kilometers of roadway, one major intersection with existing Route I, 22 new reinforced-concrete stream bridges, four reinforced-concrete box culverts, alterations or additions to an existing river bridge, the removal of two existing bridges, construction of reinforced - concrete culverts, and the construction of a storm drainage system.

Design Criteria

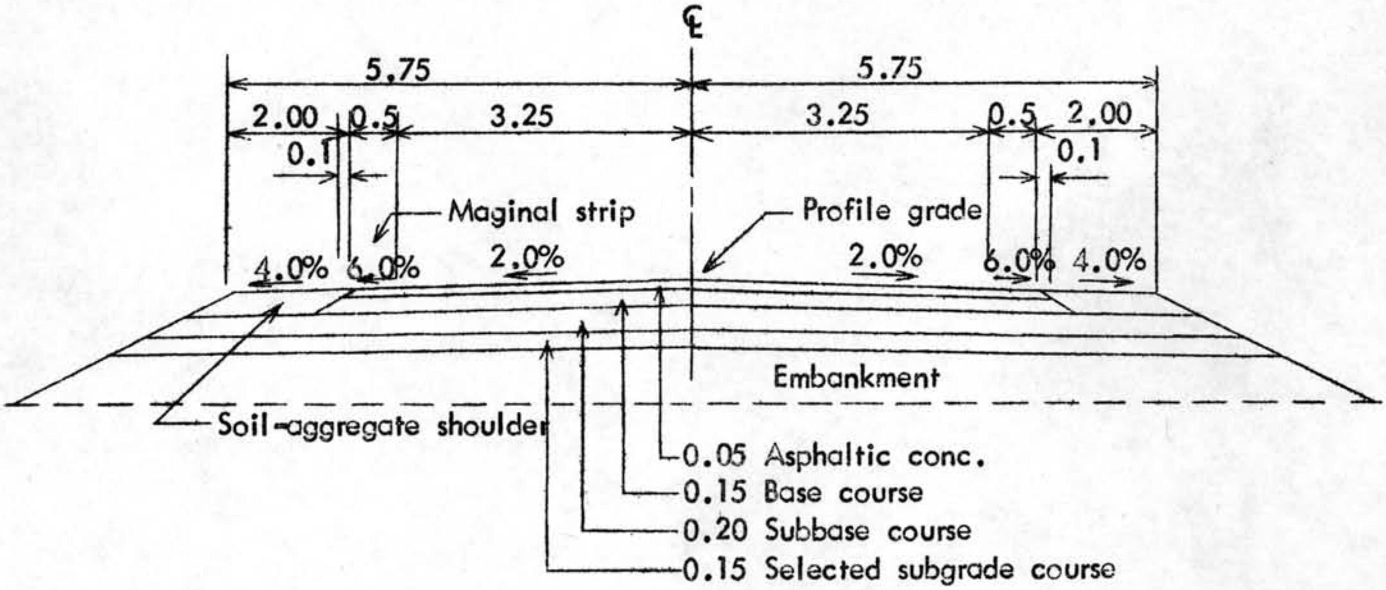
The geometric design criteria outlined herein were primarily adapted by the consulting engineers (De Leuw, Cather and Co., Ltd) from American Association of State Highway Officials (AASHO) standards and were based upon the Minimum Design Standards for Highways, P.11 - Desirable, of the Thai Highway Department. The two - lane roadway pavement section standard — consisting of a 7.50 - meter pavement width, including two 0.5-meter marginal strips, and two 2.0-meter shoulders — is maintained throughout most of the project. Typical sections for the roadway are presented in Fig.4.

Datum and Horizontal Control

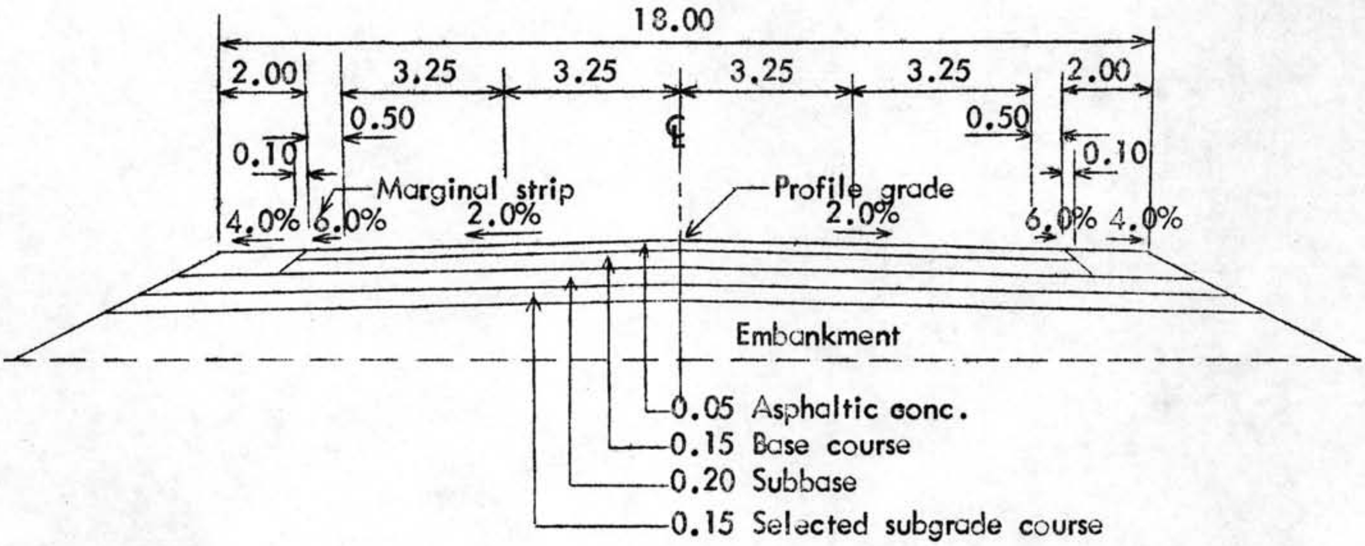
All elevations were established from the Royal Thai Survey Department Datum (RTSD), referenced to the mean sea level at Ko Lak. Horizontal control for traverses and triangulation was derived from previously established RTSD stations.

Design Speed

Flat terrain	95 km/hr
Rolling terrain	80 km/hr
Mauntainous terrain	65 km/hr



Standard 2-lane pavement section



Standard 4-lane pavement section

Fig. 4 Typical Roadway Sections

Horizontal Aligment

Minimum Radius :	Flat terrain	360 met
	Rolling terrain	260 met
	Mountainous terrain	170 met

(spiral transitions were not used)

Vertical Alignment

Maximum Gradient :	Flat terrain	5 percent
	Rolling terrain	6 percent
	Mountainous terrain	8 percent

Non - Passing Sight Distance

Flat terrain	140 met
Rolling terrain	110 met
Mountainous terrain	85 met

Superelevation

By formula : $E = 0.004 \frac{V^2}{R}$ subject to Emax = 0.10

Crown runoff 1 : 400

Superelevation runoff 1 : 200

Pavement Widening

Superelevation 2 percent, widening = 0 met

Superelevation 3 - 6 percent, widening = 0.50 met

Superelevation 7 - 10 percent, widening = 2.00 met

Cross Section Elements

Traffic lane (one direction)	3.25 met
Marginal strip	0.50 met
Shoulder width	2.00 met
Minimum height of profile above high-water level	0.50 met
Minimum width of right of way	60 met
Embankment slopes, 0 - 1 met in height	4 (H) : 1 (V)
Embankment slopes, 1 met or greater height	2 (H) : 1 (V)

Roadway Structure

Design of the roadway was based on a California Bearing Ratio (CBR) of 5, derived from subgrade investigations, traffic counts developed in the original survey, a design wheel load of 10,000 pounds (4500 kg), and the design curve of the Wyoming Highway Department (Fig.5). A minimum of 95 percent Marshall compaction (75-blow test) was specified for the wearing surface. A minimum of 95 percent modified AASHO compaction was specified for the base and subbase. Both common and selected subgrades were required to satisfy a compaction of 90 percent modified AASHO standard.

Roadway Structural Section

Asphalt pavement	Thickness = 5 cm
Base, 80 CBR minimum	Thickness = 15 cm
Subbase, 20 CBR minimum	Thickness = 20 cm

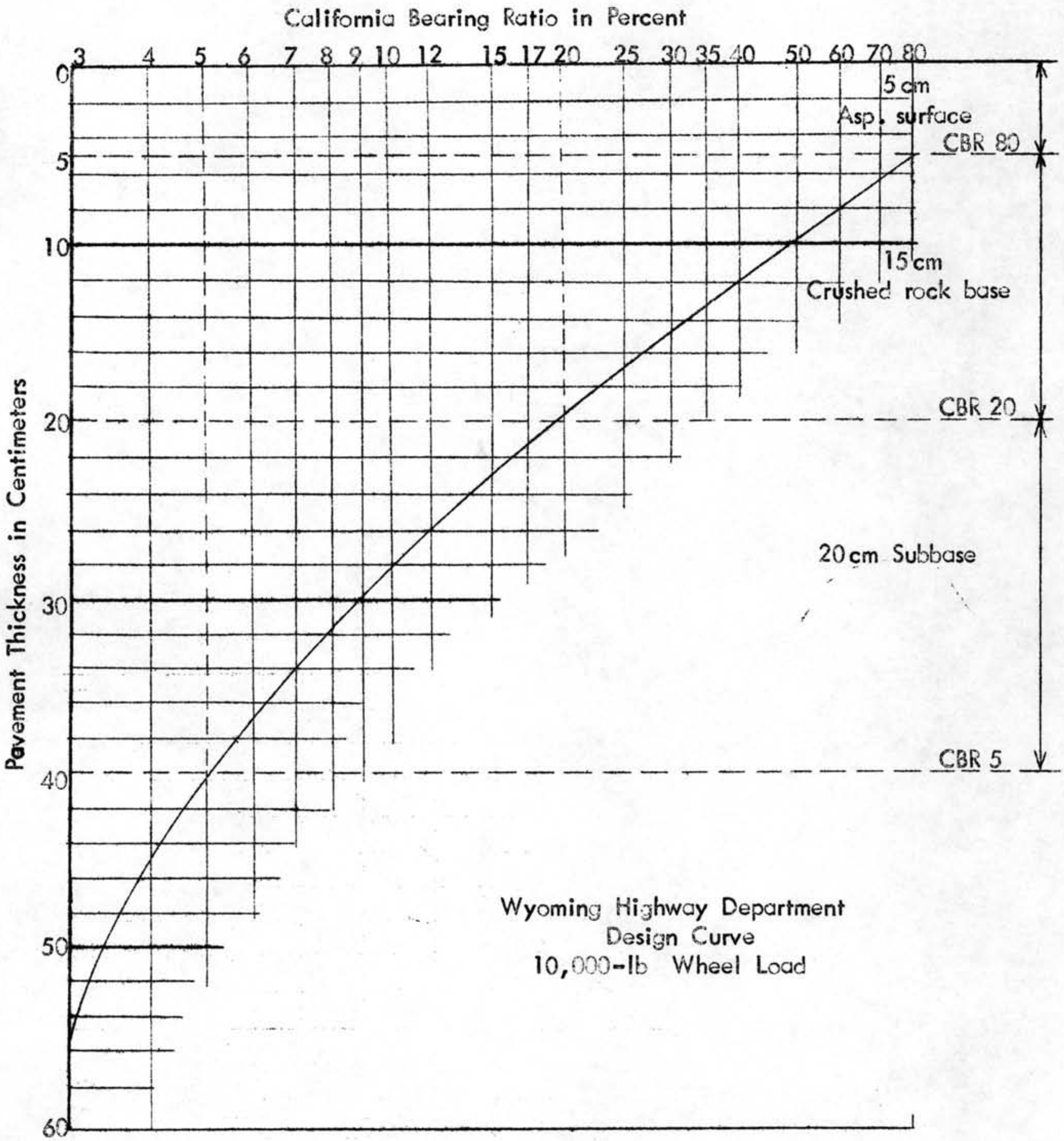


Fig. 5 CBR Design Curve

Selected subgrade and common subgrade, 5 CBR minimum	Thickness = 15 or 20 cm (where required)
Normal cross slope of pavement	2 percent
Normal cross slope of marginal strip	6 percent
Normal cross slope of shoulder	4 percent

<u>Bridge Design</u>

Design specification : American Association of State Highway Engineers (AASHO), Standard Specifications for Highway Bridges, 1961.

Design load : H 20 - S 16 - 44

Design stresses :

concrete $f'c = 210\ kg/cm^2$ cylinder strength at 28 days.

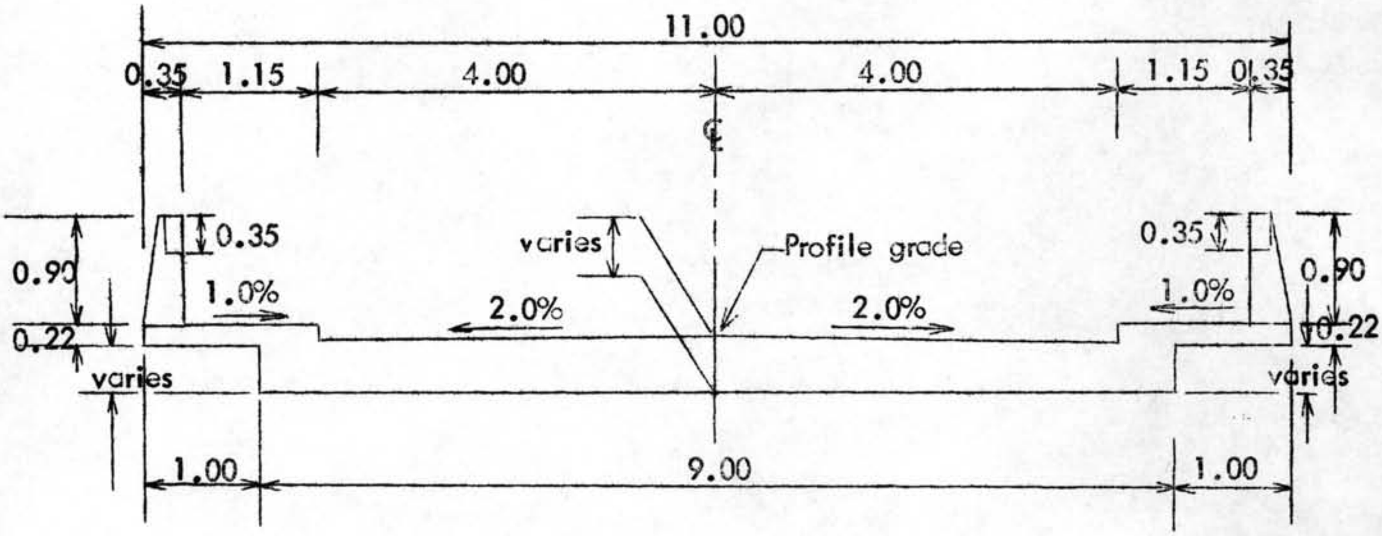
$fc = 84\ kg/cm^2$

$n = 10$

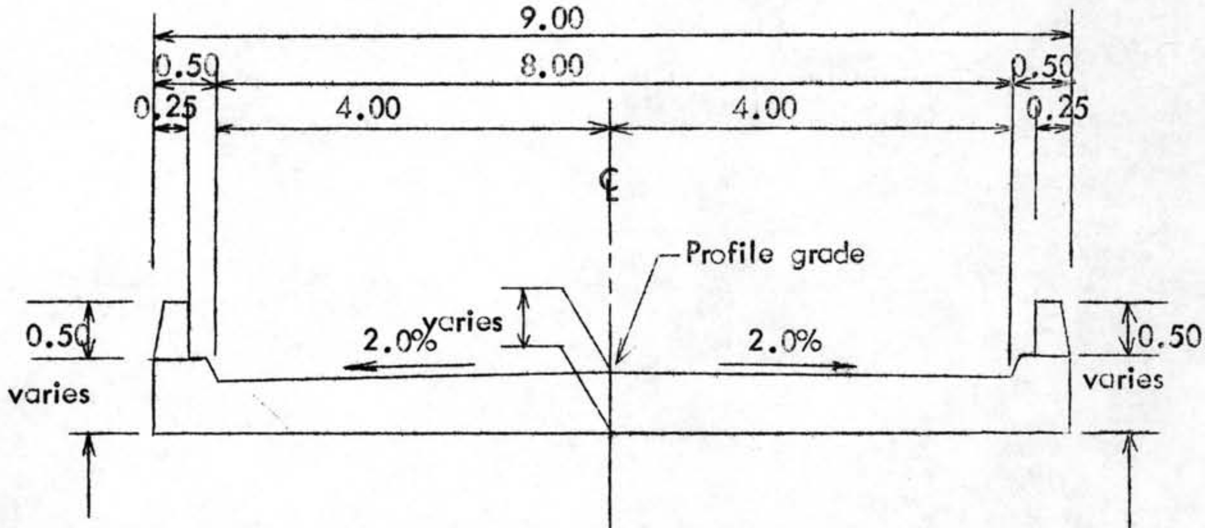
reinforcing steel $f_s = 1,400\ kg/cm^2$

Width between curbs, 2 - lane roadway	8.00	met
Width between curbs, 4 - lane roadway	12.00	met
Deck cross - slope	2	percent
Freeboard above high water	0.6	met

Fig.6 shows typical sections of the deck slabs.



Reinforced concrete slab bridge deck high railing with sidewalk for 2 lanes



Reinforced concrete slab bridge deck low railing with safety curb for 2 lanes

Fig. 6 Typical Bridge Sections

Drainage

The principal design rainfall intensities were derived from hydrological records of the Thai Government Meteorological Department. These data were obtained from automatic rainfall gauging stations situated within the project area. These records covered the years from 1961 to 1964. Bridges, culverts, and other drainage structures serving areas of less than 25 square kilometers were designed for floods of 10 - year frequency ; those for larger drainage areas were designed for floods of 50 - year frequency. The drainage structures serving the smaller drainage areas were designed by a modification of the " rational formula "; those for the larger areas were designed by the Snyder' s unit hydrograph method. Structure heights and lengths were established to provide a minimum freeboard of 60 centimeters above maximum high water and a maximum velocity through the structures of approximately 2 meters per second.

In using Section 1 (the southernmost 66.6kilometer portion of the Saraburi - Lomsak Highway) to test the RTIM, it was found necessary to further sub - divide it into four sections in order to carry out the analysis. Thus :

Section 1	0.8	km	of	6 - lane roadway
Section 2	14.1	km	of	2 - lane roadway[1]
Section 3	24.8	km	of	2 - lane roadway
Section 4	26.9	km	of	2 - lane roadway

[1] This segment represents only the new roadway. The old two - lane highway continues as the northbound roadway but was not considered in the present analysis except that it was assumed that the old roadway carried half the total two - directional traffic through this segment.

CHAPTER IV

CONSTRUCTION COSTS

Construction costs were determined from estimates of the costs of earthworks and retaining walls, site clearance, pavement, shoulders, drainage, and miscellaneous costs. In the present study, the contract construction costs were used to calibrate the model. The objective was to determine the appropriateness of RTIM to estimate the construction costs of a proposed highway, or for analysis of sensitivity of stage construction, for application in Thailand. A comparison was made between the model-generated total construction cost with the actual cost of the Saraburi-Lomsak Highway. The latter were obtained from the final report of De Leuw, Cather and Co., Ltd., the consultants responsible for supervising the construction. This comparison is given in Chapter VII : Results and Discussion of Results.

Ground Data

The ground model consists of up to 500 cross sections, each defined by chainage, center-line elevation, and average crossfall. The cross-section data may be specified by its chainage and the elevation at two given offsets from the center line. Any cross section may be specified by either method. There must be at least one cross section for every 120 met ; this restriction applies to the entire earthworks cost model. However, if the cross sections specified are further apart than 120 met, the model **automatically** inserts extra cross sections in the longitudinal section by linear interpolation to conform to the constraint.

In the present study, the ground data were obtained from the Construction Plans (DE LEUW, CATHER, 1967) and have been defined by chainage and center -line elevation where the cross section is for essentially level ground. Where the ground slopes transversely, the cross section was defined by its chainage and the elevation at each of two given offsets from the center line. Where the ground falls away in both directions transversely from the center line (or rises in both directions from the center line), judgement must be used in selecting an appropriate definition of the ground data to be used. The model as presently constituted can only accept a one - point (center line) definition, or a two - point definition (two offset points).

Horizontal and Vertical Alignment

Both the horizontal and vertical alignments of a road are to be specified by the user, and are subject to defined standards of geometric design. The model assumes that the horizontal alignment, which is defined only in terms of the average degree of curvature per kilometer[1], conforms to all of the design standards. The vertical alignment is constrained by limitations on maximum

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[1] Average degree of curvature per kilometer is the sum of the absolute values of central angles for all horizonatal curves in the section (in degrees) divided by the length of the section (in kilometers)

gradient, minimum radii of vertical curve (with different values for sags and summits), and minimum curve length. The maximum allowable gradient and the minimum acceptable curve lengths are specified by the user. The minimum allowable radii for summit and sag curves may be specified directly or may be determined from the highway design speed, as shown below. It should be noted that if a design speed is specified, it is only used as an index to determine the minimum radii ; it has no direct effect on the operating speed of vehicles on the road, although this will clearly depend on the road geometry. The model uses a conventional vertical alignment consisting of parabolic curves joined by straight tangent lines. Such an alignment can be conveniently specified by the chainage, the elevation of each intersection point, and one of the following : curve length, radius of the curve, or the rate of change of gradient. Alternatively, the engineer may specify only the chainage and elevation of the intersection points ; the model will then provide vertical curves of suitable lengths using a procedure extracted from Venus II (ROBINSON 1972). If no vertical alignment has been designed for the scheme, a suitable alignment will be generated automatically by the model using a method based on Program Venus. The alignment will follow the ground approximately and conform to specified geometric design standards, but will not necessarily balance nor minimize cut and fill quantities. All vertical alignments, whether submitted by the user or generated automatically, must obey constraints of gradient, radius, and curve length. If a constraint is violated, the alignment will be modified by the model so the violation is corrected.

In this study the average degree of curvature per kilometer was obtained from the construction plans. Also, the chainage and elevation of each intersection point, and the length of each vertical curve were obtained from the profile in the construction plans.

Minimum Radius Summit Vertical Curves

The " radius " of a parabolic curve is arbitrarily defined as the quotient of length of the curve divided by the central angle subtended by orthogonals to its tangents at the begining and end of the curve. The minimum radius of a summit curve is determined from the stopping sight distance which is the distance required to allow safe stopping of a vehicle travelling at the design speed. This distance is measured from a point 1.30 meters above the carriageway, which represents the height of the driver' s eye, to a point 0.10 meter high, representing a stationary object on the road surface.

This relationship between design speed and stopping sight distance used in the model is given in Eq.(1)

$$S = 1.88\,V - 34 \qquad (1)$$

where S = stopping sight distance, meters, and

V = design speed, kilometers / hour

To compare Eq. (1) to the minimum " desirable " design standard of the THAI HIGHWAY DEPARTMENT (1972), the perception - reaction distance and braking distance must be evaluated from Eq.(2) and (3) which are given in the THD Standards.

Perception - reaction distance = $0.278\ Vt$, meters (2)

Braking distance = $0.0039\ \frac{V^2}{f}$, meters (3)

where V = design speed, kph,

t = perception plus reaction time, seconds, and

f = coefficient of friction.

A comparison can be made between the model - generated values of stopping sight distance, from Eq.(1), with those of the Thai Highway Department. The latter can be derived from Eqs.(2) and (3) using the THD values of the coefficient of friction and a perception - reaction time of 2.5 seconds. This comparison is given in Table 1.

Table 1 Comparison of Stopping Sight Distances : RTIM and THD

Design speed, kph	Perception-reaction, m	Coefficient of friction[1]	Braking distance on level, m	Stopping sight distance meters	
				THD	RTIM
50	32.7	0.36	23.9	57	60
60	37.5	0.34	33.4	71	79
70	43.8	0.32	48.4	92	98
80	49.3	0.31	63.4	113	116
90	54.9	0.30	81.1	136	135
100	60.5	0.30	98.4	159	154
110	64.6	0.29	116.3	181	173

[1]From THAI HIGHWAY DEPARTMENT (1972)

The stopping sight distance generated by the model is greater than the THD Standard between design speeds of 50 and 80 kph. But, when the design speed is 90 to 110 kph, the model generates a lower stopping sight distance than the minimum "desirable" specification of the Thai Highway Department.

The relationship between minimum radius of a summit vertical curve (r), stopping sight distance (S), the height of eyes (h_1), and height of object (h_2) used in the model is

$$r = \frac{S^2}{2(h_1^{1/2} + h_2^{1/2})^2} \tag{4}$$

The value of the minimum radius of a summit vertical curve of Thai Highway Department is based on the following assumptions:

height of eye = 1.125 meters, and

height of object = 0.15 meters

The formulas used are:

$$L = \frac{AS^2}{420} \qquad \text{when } S < L \tag{5}$$

$$= 2S - \frac{420}{A} \qquad \text{when } S \geqslant L \tag{6}$$

where L = length of vertical curve, meters,

A = algebraic difference in, percent, and

S = stopping sight distance, meters.

The summit-curve radius is:

$$r = \frac{L}{A} \tag{7}$$

Combining Eq.(7) with either Eq.(5) or Eq.(6), as appropriate, with values of the stopping sight distance given from Eq.(1), results in the values shown in Table 2. It should be noted that the minimum summit-curve radius generated by the model is less than the minimum "desirable" standard of the Thai Highway Department when the design speed is 90 to 110 kph.

Table 2. Comparison of Summit - Curve Radius : RTIM and THD

Design speed, kph	Minimum summit - curve radius, meters	
	THD	RTIM
50	774	849
60	1200	1464
70	2015	2247
80	3040	3196
90	4404	4311
100	6019	5623
110	7800	6571

Minimum Radius Sag Vertical Curves

The minimum radius of a sag vertical curve can be calculated either from the maximum rate of centripetal acceleration in a vertical plane that is allowable for comfort, or from the minimum required stopping sight distance for night-time driving when the roadway is only illuminated by the vehicle's headlights.

The relationship between radius and centripetal acceleration is shown in Eq. (8):

$$r = \frac{V^2}{a} \qquad (8)$$

where r = radius, met,

V = design speed, meters/sec and

a = centripetal acceleration, meters/sec^2.

RTIM assumes that the maximum rate of centripetal acceleration allowable for comfort is 0.75 m/sec^2.

The stopping sight distance required in the case of visibility when the road is illuminated by headlights is the same distance at a given design speed as that for daytime operation. The model assumes that the headlights are 0.7 meter above the roadway and that the "hot spot " of the high beam rises 1° above a reference plane which is parallel to the roadway directly under the vehicle. The relationship between minimum radius (r), stopping sight distance (S), headlight (h), and angle of headlight (θ) is given in Eq.(9)

$$r = \frac{S^2}{2(h + S \sin \theta)} \qquad (9)$$

The model calculates the minimum radius of each sag vertical curve by both of these methods (Eqs.7 and 8) and selects the larger of the two.

It is of interest to compare the headlight sight distance on sag vertical curves with design standard of the Thai Highway Department. The Highway Department Standard is based on a height of headlights of 0.75 meters, with the beam rising 1 degree above the reference plane.

The formulas used are :

$$L = \frac{AS^2}{152 + 3.5S} \qquad \text{when } S < L \qquad (10)$$

$$L = 2S - \frac{152 + 3.5S}{A} \qquad \text{when } S > L \qquad (11)$$

where L = curve length, meters,

A = algebraic grade difference, percent, and

S = stopping sight distance, meters

As before, the "radius" of the parabolic vertical curve is arbitrarily defined for sag vertical curves as shown in Eq.(7)

In comparing the radii of sag vertical curves for design speeds of 50 kph through 110 kph, it was found that the headlight visibility criterion always governed. A comparison is shown in Table 3.

Table 3 Comparison of Sag - Curve Radius: RTIM and THD

Design speed, kph	Sag - curve radius, meters	
	THD	RTIM[1]
50	924	1030
60	1259	1496
70	1786	1982
80	2336	2480
90	2945	2988
100	3568	3501
110	4171	4018

[1] All RTIM radii shown are governed by the highlight visibility criterion, not that of centripetal acceleration (comfort).

It should be noted that the RTIM minimum radius of sag vertical curves for design speeds of 100 and 110 kph is less than the minimum "desirable" standard of the Thai Highway Department.

Roadway Cross Section

The roadway cross section used by the earthwork model in RTIM is shown in Fig.7. The cross section is for a single carriageway road and is specified in terms of carriageway width and crossfall, shoulder width and crossfall, cut slope, fill slope, ditch depth, ditch bottom width and slope, and ditch side slope. Ditches are provided only when the road is in cut. The width of the strip which must be cleared of vegetation is also specified.

If the engineer requires any slope (except the cut slope or the fill slope) to be horizontal, for example the ditch bottom, no slope need be specified and the model will interpret this correctly. If the cut slope or fill slope is not specified, the model assumes these are 1 : 1.

Most of the roadway cross - section data : for example, the carriage-way width and crossfall, the shoulder width and crossfall, the ditch depth, and the ditch bottom width, have readily specified values. But the slope of cuts, fills or ditch sides may vary depending on the stability of the soil and the height of the slope as shown in Table 4, which shows 1972 standards of Thai Highway Department. The THD also specifies that side ditch slopes, from the edge of shoulder to the bottom of the ditch, shall be not steeper than 4 : 1. Flatter slopes should be used in soil highly susceptible to erosion.

As shown in Fig.7, the cut slope used by the model is constant throughout its height. However, the rock - cut slopes for the Saraburi - Lomsak Highway were benched as shown in Fig.8. This required selection of an average value for use in the model. It was found to be very difficult to select an appropriate single value of such benched cut slopes, because of the variation of the design with the depth of the cut. An analysis of the sensitivity of cut and fill slopes to the total volume of earthwork and to the construction cost has been carried out in the present research and will be discussed latter.

Table 4 Maximum Slopes[1] for Cuts and Fills
(THAI HIGHWAY DEPARTMENT, 1972)

Height of cut or fill, meters	Earth		Soft rock		Hard rock	
	Cut	Fill	Cut	Fill	Cut	Fill
0.0 - 1.0	3 : 1	4 : 1	2 : 1	3 : 1	0 : 1	$1\frac{1}{2}$: 1
1.0 - 3.0	2 : 1	2 : 1	1 : 1	$1\frac{1}{2}$: 1	0 : 1	1 : 1
over 3.0	$1\frac{1}{2}$: 1	$1\frac{1}{2}$: 1	$\frac{1}{2}$: 1	1 : 1	0 : 1	1 : 1

[1] Run : rise

Table 4 gives the design standards for cut and fill slopes used in the design of the Saraburi - Lomsak Highway. Flatter slopes were used for soils highly susceptible to erosion. Typical fill sections and cut section for earth and rock as used for this highway are shown in Fig.9.

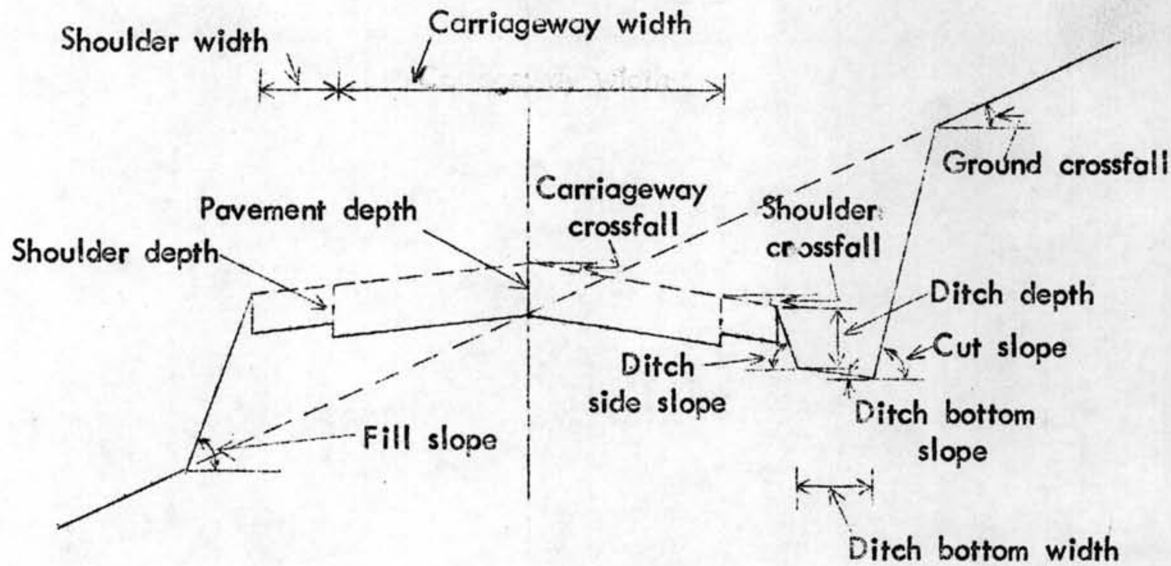


Fig. 7 Roadway Cross Section

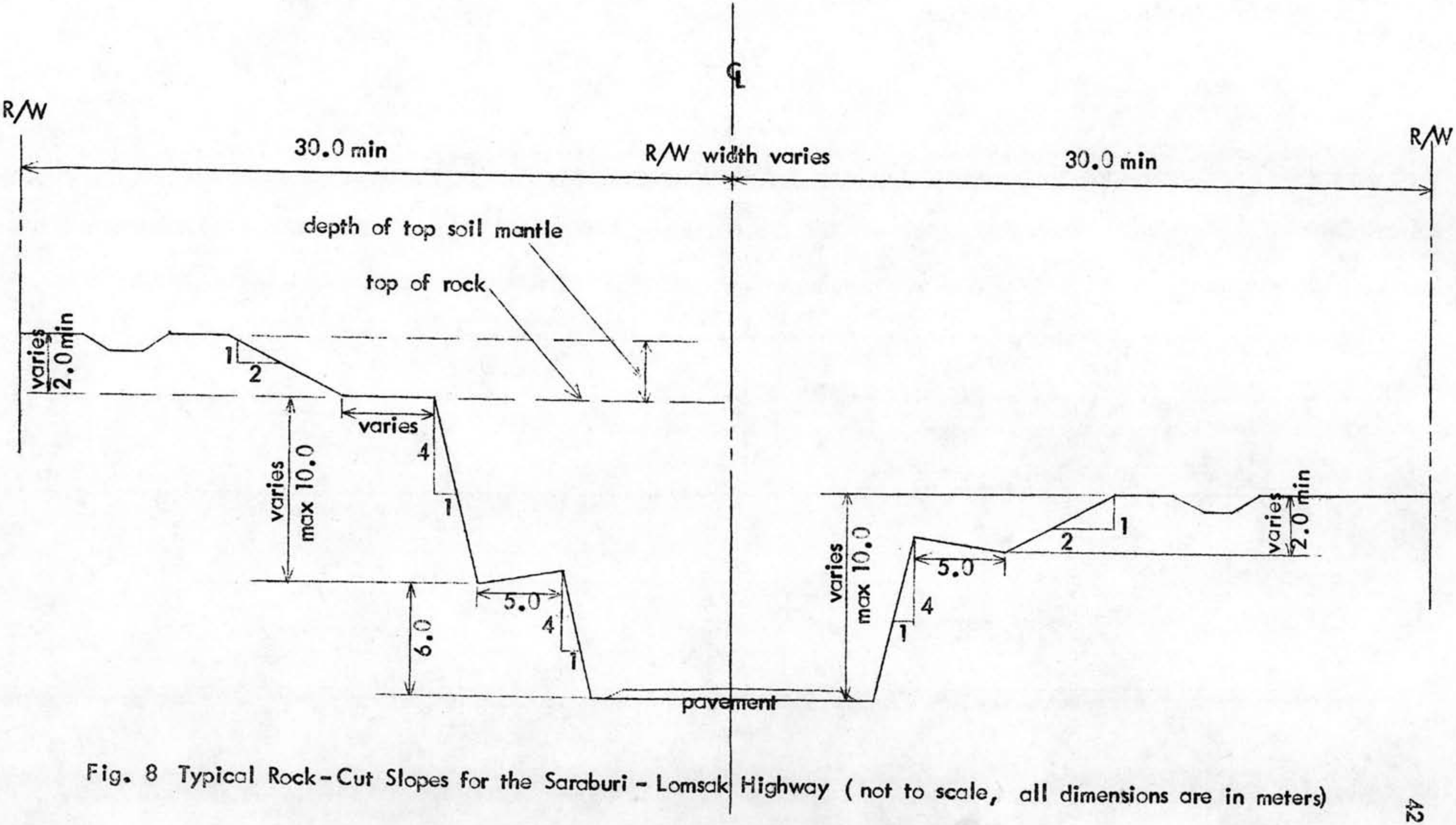
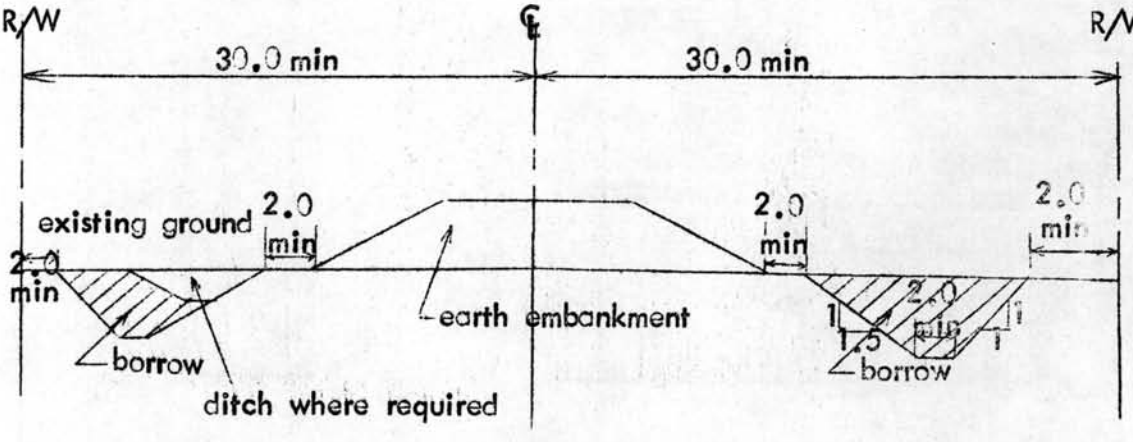
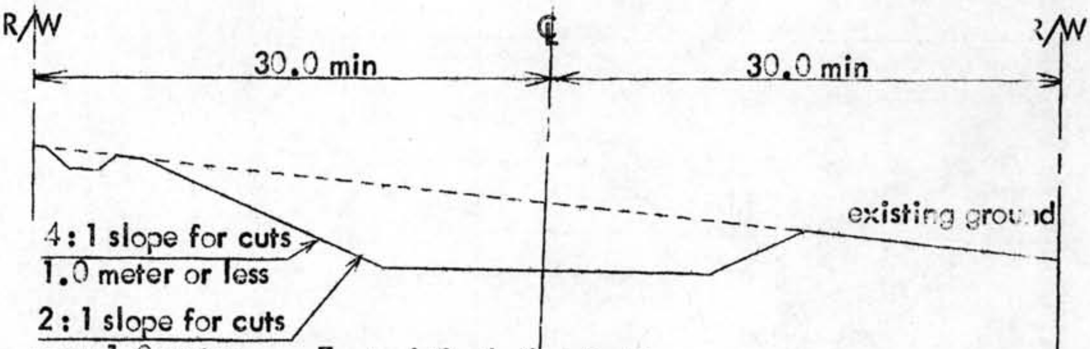


Fig. 8 Typical Rock-Cut Slopes for the Saraburi - Lomsak Highway (not to scale, all dimensions are in meters)



Typical Fill Section & Borrow Limits



Typical Earth Cut Section

Fig. 9 Typical Fill and Cut Sections for the Saraburi - Lomsak Highway

Soil Characteristics

The average percentage of cut material which is unsuitable for use as fill may be input to the model. The volume of all cut material is reduced by this percentage when determining the volume available for constructing embankments. The unsuitable material is costed as spoil. The change in volume of material during haulage and during construction compaction may also be taken into account by specifying the bulking and compaction factors. If no values are specified by the engineer, the model assumes a value of 1.25 for the bulking factor and 0.8 for the compaction factor. This assumption results in a given volume of excavated material being recompacted to its original volume.

When it is necessary to remove material prior to placing an embankment, there is no practical means to input this information into RTIM. It was found diffcult to select appropriate single values for the bulking factor and the compaction factor for the heterogeneous soil types encountered in the selected portions of the Saraburi - Lomsak Highway.

Earthwork Volumes

Earthwork volumes are calculated in RTIM by the average end - area method. The area of each cross section is found by determining the area of trapezoids and triangles between the sloping ground and the bottom of the pavement, the bottom of the shoulders, the cut slope, the fill slope, and the ditches. The volume between consecutive cross sections is assumed to be the product of the distance between them and the average of their areas. For a cross section which is a cut to fill transition, the areas of cut and fill are calculated separately. Thus transverse filling is taken into account directly by assuming that its cost is only the cost of excavation on one side of the road plus the cost of filling and compaction on the other.

Excavation and compaction are costed in the model on a unit volume basis. The compaction cost is based on the volume of compacted material,' rather than on the volume of loose material, as the latter may be different because of compaction.

There are many methods to calculate the earthwork volumes. The method used in the model is a basic one that is used in Thailand and can be accepted.

Retaining Walls

If the ground crossfall exceeds a specified value and the road is on fill at the shoulder above the lower side of the slope, a retaining wall is provided automatically by the model. The proportional dimensions of the walls may be specified by the engineer but, in the absence of this information, a dry - stone masonry wall is assumed and the following dimension are used. A dimension sketch is shown in Fig. 10.

Width at top	=	0.25	x	height,
Width at bottom	=	0.5	x	height, and
Foundation height	=	0.1	x	height.

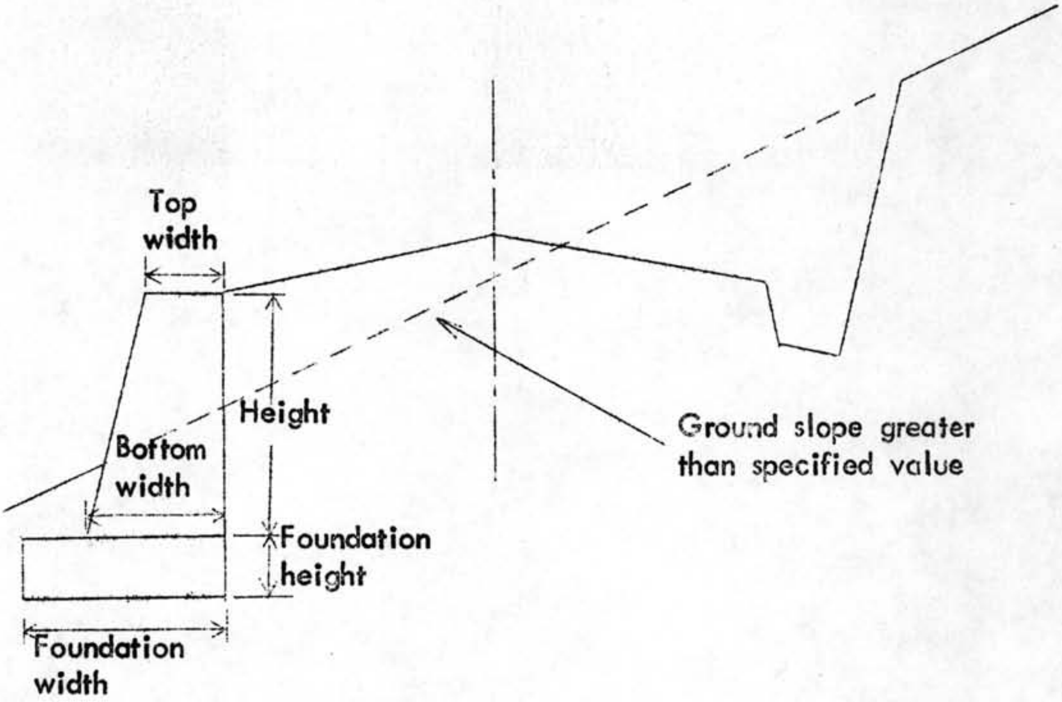


Fig. 10 Roadway Cross Section with Retaining Wall

The foundation is 1 meter wider than the base of the wall and is covered by a minimum depth of 0.3 meter of soil. The volume of material in the wall is calculated from these dimensions and the cost is based on this. Clearly, no fill slope is provided when a retaining wall is used.

In analysis of the Saraburi - Lomsak Highway, retaining walls were not considered, except at certain bridges. The costs of the latter were included in the costs of the major river crossing structures where appropriate. The retaining walls used for this highway were reinforced concrete structures, rather than the dry - stone masonry walls presupposed by RTIM.

Haulage, Borrow, and Spoil

Haulage costs in the model are calculated using a mass - haul diagram and are based on the fact that it is never economical to transport material further than " marginal haul" (sometimes called the limit of profitable haul).

$$\text{Marginal haul} = \frac{\text{unit barrow cost} + \text{unit spoil cost}}{\text{unit haulage cost}} \quad (12)$$

Haulage costs are found by multiplying the total area of all the balanced loops of the mass - haul diagram by the unit haulage rate which is expressed as the cost of hauling one cubic meter of material one kilometer. The total borrow volume found by the mass - haul strategy is multiplied by the unit cost of borrow material to give the total cost of borrow. The amount of unsuitable material found from the calculation of the useable cut volume is added to the spoil volume obtained by the mass - haul strategy to give the total volume of spoil. This total spoil volume is multiplied by the unit cost of spoil to give the total cost of spoil.

Highway construction contracts in Thailand do not consider the costs of haulage, borrow,or spoil. All of these costs are included in the lump sum costs of excavation and of constructing embankments. These are expressed in cost per cubic meter. The cost of loading, spreading, and compaction are included to the contract cost of embankment. In some areas along the Saraburi - Lomsak Highway, the contractor elected to go outside the right of way limits for borrow material whenever it was considered economical to do so. This was at his own discretion, and no additional payment was allowable under the contract. In summary, the msss - haul diagram is not used by the Thai Highway Department. Only two costs -- that for excavation and that for construction of embankments -- are contract items for earthwork.

Site Clearance

The model allows for consideration of five categories of vegetation for site clearance. The percentage of the total area to be cleared which is covered by each vegetation type must be specified, as must be the width of the strip to be cleared. The total area to be cleared is determined and the various proportions of this area are then costed at the appropriate rates and summed to give the total cost of clearing. In the present research only one type of vegetation to be cleared was specified with its unit cost of clearing.

Pavement and Shoulders

Up to six pavement layers may be used for analysis by RTIM and each is specified by its thickness after compaction. A bulking factor may be specified to enable calculation of the actual volume of material required. For paved roads, the strengths of the materials in the base and sub - base must be specified in terms of California Bearing Ratio (CBR), or as a 7 -day unconfined compressive strength for a cement - stabilized base. If these values are not known, the model assumes a CBR of 25 percent for all sub - base material, a CBR of 80 percent for a crushed stone base, and the 7 - day unconfined compressive strength of a cement - stabilized base to be 1.7 MN/m^2.

The cost of each pavement layer is the sum of material acquisition and haulage costs (cost per cubic meter), and the placement cost (cost per square meter). Areas and volumes are based on the specified carriageway width and the length of the road. The cost model for shoulders is identical to that for the pavement.

Grips (transverse French drains) may be provided to drain the pavement structure for paved roads. The longitudinal spacing of these must be specified and they are costed per unit length. The grips are 0.30 meter width and 0.15 meter deep.

The typical pavement section employed in this study of the Saraburi - Lomsak Highway was obtained from the construction plans and is shown in Fig.11.

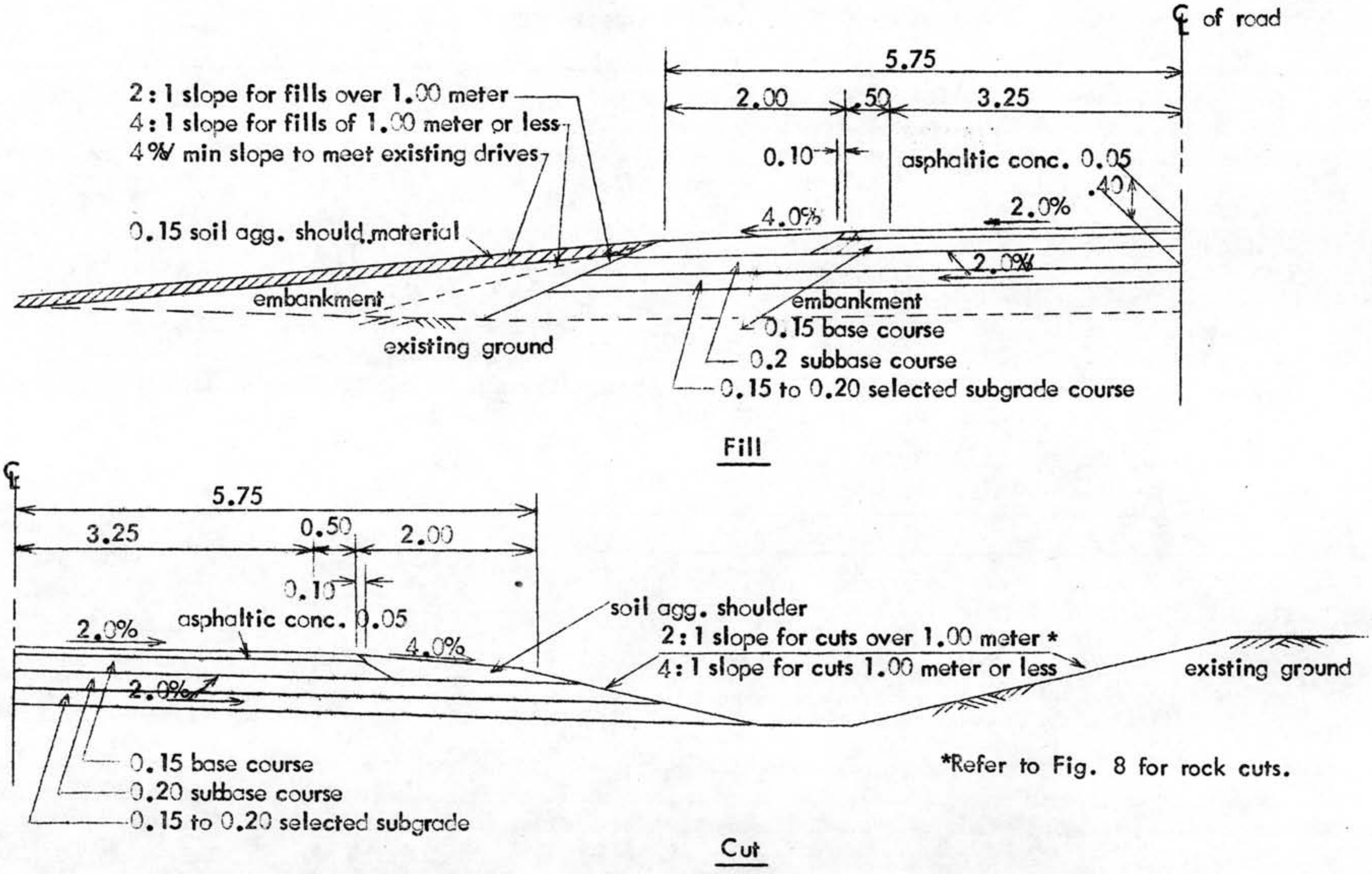


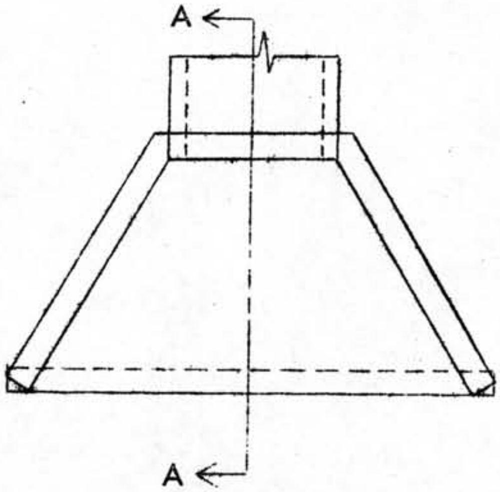
Fig. 11 Typical Pavement Sections of Saraburi – Lomsak Highway

Drainage

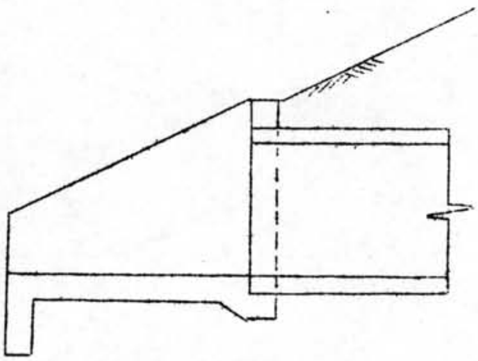
The model separately considers cross-flow drainage and minor and major river crossings. The maximum hourly intensity of rainfall, in mm/hour, with (suggested) a long recurrence interval, say 20 years, must be specified, and up to five standard culvert sizes may be used. The culvert sizes are specified by diameter, measured in meters, and should be input in increasing size. For each size, a unit cost per length of culvert placed on site must be specified. The average length that a culvert extends beyond the shoulder of the road may also be given. The size of headwall is dependent upon the culvert size and is calculated from the height above the culvert to the top of the headwall (0.25 meter), the depth below the culvert to the bottom of the headwall (0.25 meter), the width of the headwall on either side of the culvert (1.0 meter), and the thickness of the headwall (0.25 meter). The values shown in parentheses are those used by the model; there seems to be no practical way to substitute other values. The cost of headwalls is based on the unit cost per volume of the material required for their construction. A typical headwall as used for the Saraburi-Lomsak Highway is shown in Fig. 12, along with the simple design employed by the model. The construction costs of these two types would obviously be different, leading to differences in the comparison of cost of the model and the prototype.

Flooding must also be borne in mind. Although this will be localized and will therefore have little effect on the general deterioration of the

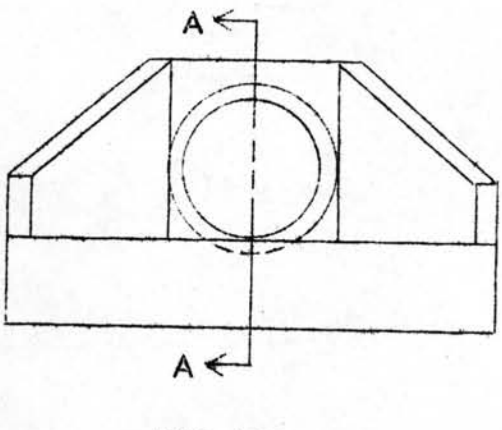
Fig. 12 Typical Head Walls



PLAN

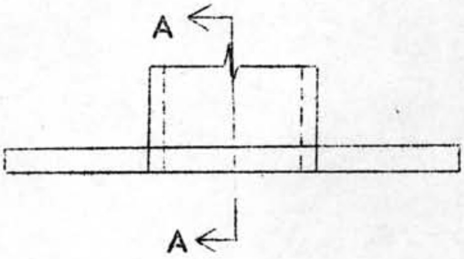


SECTION A - A

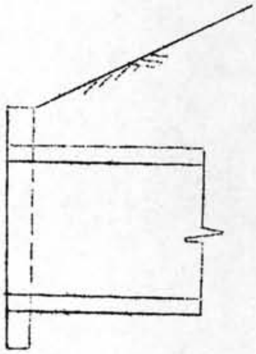


END VIEW

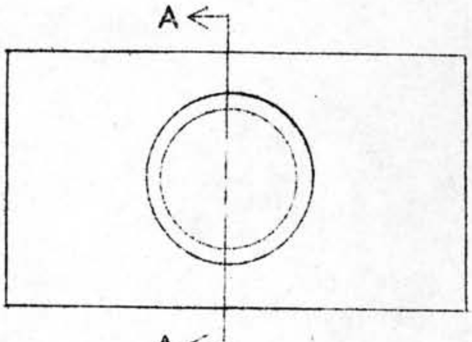
SARABURI - LOMSAK HIGHWAY



PLAN



SECTION A - A



END VIEW

RTIM

pavement, it is likely to cause closure of the road which will affect road user costs. Allowance has therefore been made in the model input for the user to specify the number of days each year that the road will be impassable.

Culverts may be provided at intervals to channel the surface water across the road from longitudinal ditches. The spacing and diameter of these culverts must be specified if they are required. The model uses an average longitudinal spacing between culverts to calculate the cost of culverts.

The area of waterway opening required at each minor river crossing is calculated from Talbot's formula, Eq.(13).

$$a = 0.1831 \; CA^{3/4} \tag{13}$$

where a = area of the opening, square meters,

A = catchment area, hectares, and

C = terrain coefficient with a value between 0 and 1.

Values of the terrain coefficient depend on the vegetation and the undulation of the ground. Heavy scrub growth calls for a lower value of this coefficient, whereas the value is higher for rocky or barren slopes. Predominantly sandy or gravelly soils which are permeable require lower values, and impervious heavy clays result in higher values of the terrain coefficient. The model determines the number and size of pipes required to provide this opening and employes these to calculate the cost of transverse drainage through culverts.

A major river is presumed to be crossed by a bridge and the cost of this is specified directly as a lump - sum construction cost.

The maximum hourly rainfall intensity used to calculate the drainage requirements of the Saraburi - Lomsak Highway were obtained from a combination of rainfall data from automatic recorders in Lopburi and Petchabun provinces. The data base for these stations was from 1931 to 1960. Along the Saraburi - Lomsak Highway, culverts of single or multiple barrel arrangements were provided as required by drainage considerations. Culvert diameters of five different sizes were employed in the design. In the RTIM model, only one diameter of culvert can be specified. To overcome this problem in the present study, the culverts having the largest diameter which occurred most frequently were assumed to be regularly spaced along the study sections. All other culverts (of diameters less than the largest) were costed separately and the cost added to the lump sum cost of major river crossing structures. The catchment areas for the minor river crossings were taken from the drainage design records of De Leuw, Cather and Co., Ltd.

The Thai Highway Department no longer employs the Talbot formula for drainage design. There are sound hydrological reasons for believing that this empirical formula is unreliable, even for those areas for which it was derived in the United States. Having regard for the now more generally available climatic and topographic data, the flood estimation method employed in the present study is based on a modification of the " rational formula ", using local data for the rainfall intensity and infiltration capacity ; alternatively, a modified from of Synder' s synthetic unit hydrograph was used. The first method was used for catchments not exceeding 25 km^2, the second method for catchments of 25 to 1,000 km^2. Accordingly, the areas of waterway openings required at minor river crossings calculated for the Saraburi - Lomsak Highway are not the same as those calculated by the model. This leads to differences in the size of culverts specified , and differences in the costs.

Other Items

Additional costs which are not accounted for elsewhere in the model are divided into two types : those which are known directly, and those which depend on the total cost of construction.

The miscellaneous costs which make up the first group comprise such things as fencing, road marking and sign posting, but may include any cost which is known directly and is not specified elsewhere. Large structures which are not included in the drainage model should be included under this heading. The total miscellaneous cost is simply added to the total construction cost.

Overheads and supervision by consultants, etc., are usually accounted for as a percentage of the total cost of the project. Any cost which is determined in this way may be specified under this heading and the cost will be taken as the required percentage of the earthworks, site preparation, pavement and shoulders, drainage, and miscellaneous costs. This percentage may be further subdivided into a percentage which is spent on labour and a percentage which is spent on plant.

In Thailand, the contractor's overhead and supervisory costs are included in the construction bid costs. The expression used to describe these relationships is given by Eq. (14).

$$\text{Bid cost} = f\,(\text{Material costs} + \text{Operating costs}) + \text{Depreciation of mechanical equipment} \qquad (14)$$

$$\text{where } f = \left(1 + \frac{a}{100}\right)\left(1 + \frac{x + y + z + b}{100}\right) \qquad (15)$$

a = tax, percent

b = contingency, percent

x = profit, percent

y = overheads and supervision, percent, and

z = interest, percent.

The bid costs are expressed in cost per unit of work to be performed, or as a lump sum cost. The contract unit costs were used in this research to calculate the total cost of construction of the highway. The unit costs that were input to the model included allowances for the contractor's overhead and supervisory cost. So, a value of zero percent for overheads and supervision was used in the present study.

Cost and Foreign Exchange.

In programming RTIM,, any cost may be expressed as a unit cost or in terms of quantities of work to be accomplished and productivity rates for labour, plant, etc. Costs may be specified in any monetary units. If no units are specified, pounds sterling are assumed by the model. Percentages of construction materials, labour, or plant costs which require foreign exchange may also be specified. The cost in foreign exchange will be calculated by the model from a specified exchange rate for the foreign currency unit. Foreign exchange costs can only be calculated correctly by the model if all job ratesare expressed in terms of productivities.

The construction period, which in RTIM may last a maximum of four years, may also be specified; the percentage of the construction cost spent in each year must be given. The construction cost can then be discounted to give its present (year zero) value using a specified discount rate. It is assumed by the model that all foreign exchange requirements for construction are spent at the outset, hence these are not discounted.

Whilst the preceding are useful aspects of the model the present study did not consider foreign exchange costs because this was not the main point of the analysis. The construction period for the Saraburi-Lomsak Highway was obtained from the progress schedule in the Final Report (DE LEUW CATHER, 1970).

CHAPTER V

ROAD USER COSTS

Road user costs were determined from estimates of the costs of fuel consumption, lubricating oil consumption, spare parts consumption, tyre wear, maintenance labour hours, depreciation, interest charges, crew hours, passenger time costs, and standing charges. The relationships which are incorporated into the model to estimate road user costs were mostly derived from the vehicle performance study and the road user survey conducted by the TRRL in Kenya. If the Kenya data are found to be unsatisfactory or incomplete, data from other sources may be incorporated into the model. The availability of accurate up - to - date information on vehicle operating costs is one of the most important prerequisites for evaluating the total road transportation cost, or for estimating future benefits in a feasibility study.

In the present study only a paved road was analysed. A comparison was made between the model - generated road user costs with those which were obtained for Thailand study by T.P.O' SULLIVAN & PARTNERS (1973). This comparison is presented in Chapter VII : Results and Discussion of Results.

Vehicle Types

Eight classes of vehicles may be used in the model. These are : (a) passenger cars ; (b) light commercial vehicles, up to 2.5- tonnes gross weight ;

(c) buses of 110-brake horsepower and 11-tonnes gross weight; and (d) five classes of heavy commercial vehicles. These eight classes may contain any groups of similar vehicles that the model user may require provided that he specifies the mean equivalence factor for each class. If the equivalence factors are not known, or not stated, the model will assume the vehicle types conform to those shown in Table 5. The configuration of axles for buses and heavy commercial vehicles, and the percentage of the gross vehicle weight on each axle, are shown in Table 5.

Table 5 Axle Weights for Buses and Heavy Vehicles Used in the Model

Class Number	Vehicle type	Type of axle and percentage of total weight on each axle					
		1	2	3	4	5	6
3	Bus	SW[1] 34	DW[2] 66				
4	2-axle truck	SW 34	DW 66				
5	3-axle truck	SW 25	tandem set 75				
6	4-axle truck	SW 22	DW 44	SW 17	DW 17		
7	5-axle truck	SW 14	DW 29	DW 19	tandem set 38		
8	6-axle truck	SW 12	tandem set 36		DW 18	tandem set 34	

[1]Single wheel at each end of axle

[2]Double wheel at each end of axle

In the present study, six classes of vehicles were used. These were classified as: (a)cars and taxis; (b) light trucks; (c) light buses; (d) heavy buses; (e) heavy trucks; and (f) truckswith more than two axles. Most of the trucks in the last category are three-axle trucks; therefore, the characteristics of three-axle trucks were used to represent all trucks having more than two axles. The axle-weight distributions for buses and heavy vehicles as used in the present study are shown in Table 6.

Table 6. Axle Weights for Buses and Heavy Vehicles Used in the Present Study[1]

Vehicle type	Type of axle and percentage of total weight on each axle		
	1	2	3
Bus and 2 - axle truck	SW[2] 35	DW[3] 65	
3 - axle truck	SW 20	Tandem set 41[4] and 39[5]	

[1] From Materials and Research Division; Thai Highway Department

[2] Single wheel at each end of axle

[3] Double wheel at each end of axle

[4] First axle of tandem set

[5] Second axle of tandem set

The amount of damage done to a road by a moving vehicle depends very strongly on the axle load of the vehicle. The relationship between the damage and the axle load is important for the proper design and evaluation of pavements. To help with this design, "equivalence factors" are used. The equivalence factor of a vehicle is defined as the number of passages of an axle carrying a standard 8200 - kg load which would do the same damage to a road as one passage of the vehicle in question. The equivalence factor for each class of vehicle can be calculated from the gross vehicle weight for each direction of travel using Liddle's formula[1]

[1] LIDDLE, W.J. (1963), Application of AASHO Road Test Results to the Design of Flexible Pavement Structures, University of Michigan.

$$EF = \left(\frac{L_i}{8,200}\right)^{4.5} \text{ for each single axle.} \tag{16}$$

where EF = unweighted equivalence factor ;

L_i = load in kilograms on a single axle.

In the RTIM model, the mean equivalence factor for each vehicle class is calculated from the mean vehicle weight of each class. This averaging procedure will seriously underestimate the road - damaging power of vehicles, particularly when they are very heavily laden. It is strongly recommended by TRRL that the mean equivalence factor for each direction of travel and for each heavy vehicle class be specified by the user whenever possible. Equivalence factors for passenger cars and light commercial vehicles are negligible in comparison with those of medium and heavy commercial vehicles and are therefore ignored in calculating the deterioration of paved roads.

For the present study, a field survey was conducted to determine the distribution of gross and axle weights of vehicles using the Saraburi - Lomsak Highway. These data were collected by recording the weight of vehicles stopping at the Highway Department weighting station at Hin Khang, south of Saraburi. From these data the mean vehicle weight for each class of heavy vehicle was calculated. Also, the mean equivalence factor was calculated from data for all the vehicles within each vehicle class. The resulting mean equivalence factor was supplied as input to the model, thus minimizing the underestimation of the road damaging power factor.

Observations at the Hin Khong weighting station were made by recording the scale reading as heavy vehicles stood on the weight-bridge. In addition, each driver was interviewed to determine the origin of his trip if southbound, or the destination of his trip if northbound. These observations were then classified for application to either the Saraburi - Ban Phu Khae segment (which comprises study Section Nos. 1 and 2), or the Ban Phu Khae - Lamnarai segment (which comprises study Section Nos. 3 and 4). The data and the corresponding calculated equivalence factors for heavy vehicles are shown in Appendix A, Tables A 1 through A 5.

Traffic Forecasts

All traffic forecasts used by the model must be applied by the user. These forecasts can be given in any of serveral ways. (1) The user may specify the average daily traffic (ADT) separately for each year of the analysis period. Thus, the user may study the effects of a sudden change in the traffic growth rate at any time during the design life of the road. Such sudden changes might be caused by diversion of traffic to another route, or by a change in government policy. (2) The user may specify a first - year ADT and the rate at which this traffic will be expected to grow thereafter. (3) The user may specify a first - year ADT and a constant increment (in vehicles/year) by which the traffic will increase in each subsequent year. Whichever of the three options is used, a different growth rate may be specified for each vehicle class.

The relationships used in the model to predict vehicle speeds and fuel consumption were developed under substantially free - flowing conditions where the only constraints on speed were the physical characteristics of the road and the characteristics of the vehicles. Hence, these relationships may not be valid when traffic interaction take place. It is assumed that the critical level of flow is about 3,000 passenger car units (pcu) per day, where cars and light vehicles are 1 passenger car unit, and medium and heavy vehicles are 3 pcu's. If these levels are exceeded, a warning massage will be printed but the model will proceed with the calculations.

In the present study the ADT volumes by type of vehicle were obtained from reports of the DEPARTMENT OF HIGHWAYS (1969 - 1974). An exponential curve was fitted by regression analysis to the observation of ADT for the years for which data were available for each of two highway segments : Saraburi - Ban Phu Khae, and Ban Phu Khae - Lamnarai. These calculations are shown in Appendix A, Table A 6 through A 9. The resulting curves are shown in Figs. 13 and 14. As the Saraburi - Lomsak Highway was opened to traffic in 1970, that year was taken as the base year for traffic studies. The 1970 ADT for each of the two segments was selected from the regression curve. In the subsequent analysis, traffic growth rates of 5, 10, and 15 percent per annum were applied to the base - year values to determine the sensitivity to variations in traffic of the total discounted construction, road maintenance, and vehicle operating cost.

Vehicle Speeds

Vehicle speeds are expected to be less for narrower widths of road. From the observations in Kenya, speeds were found to be significantly reduced

Fig. 13 ADT Observations and Regression Curve, Saraburi-Ban Phu Kae Segment

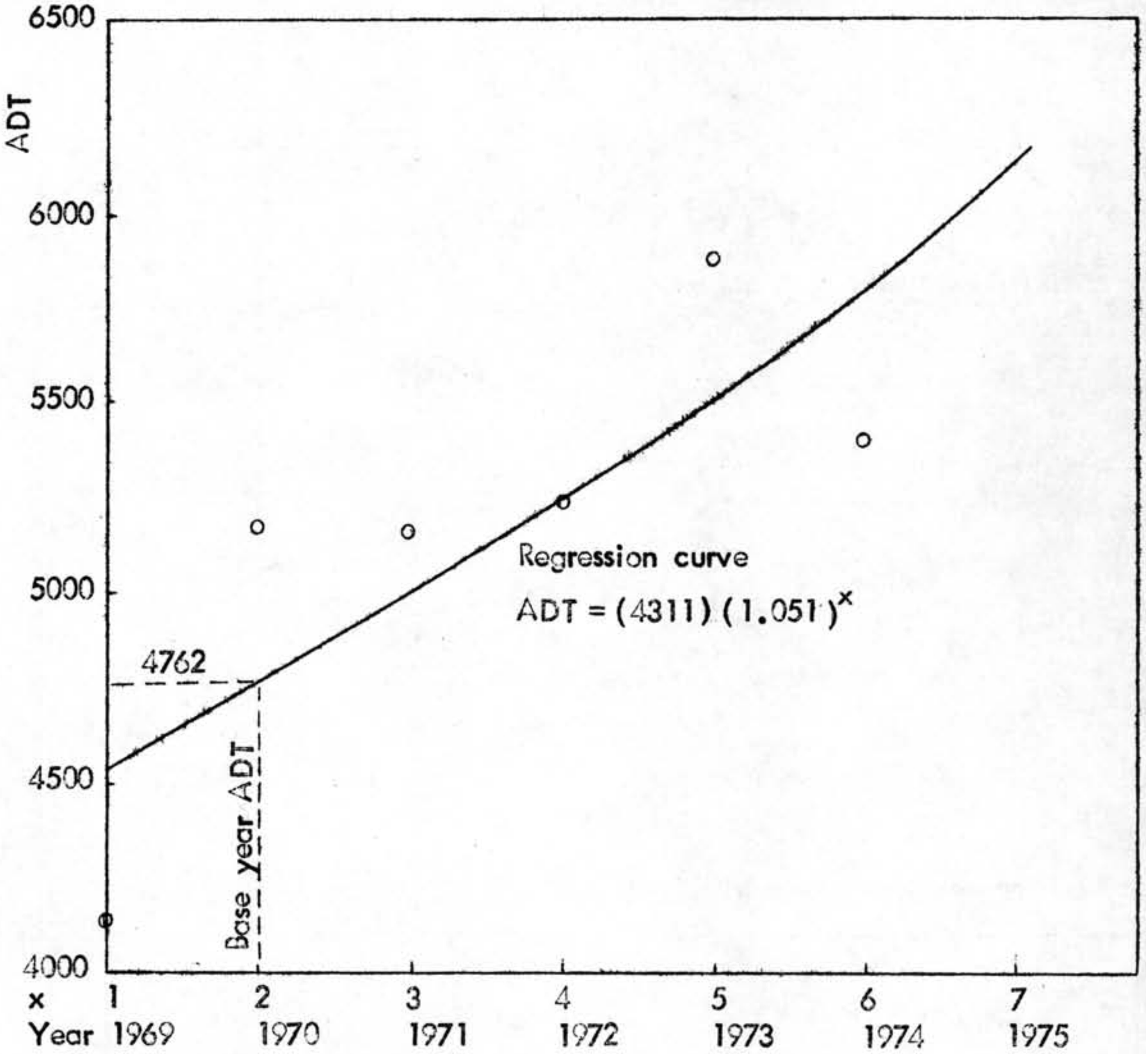
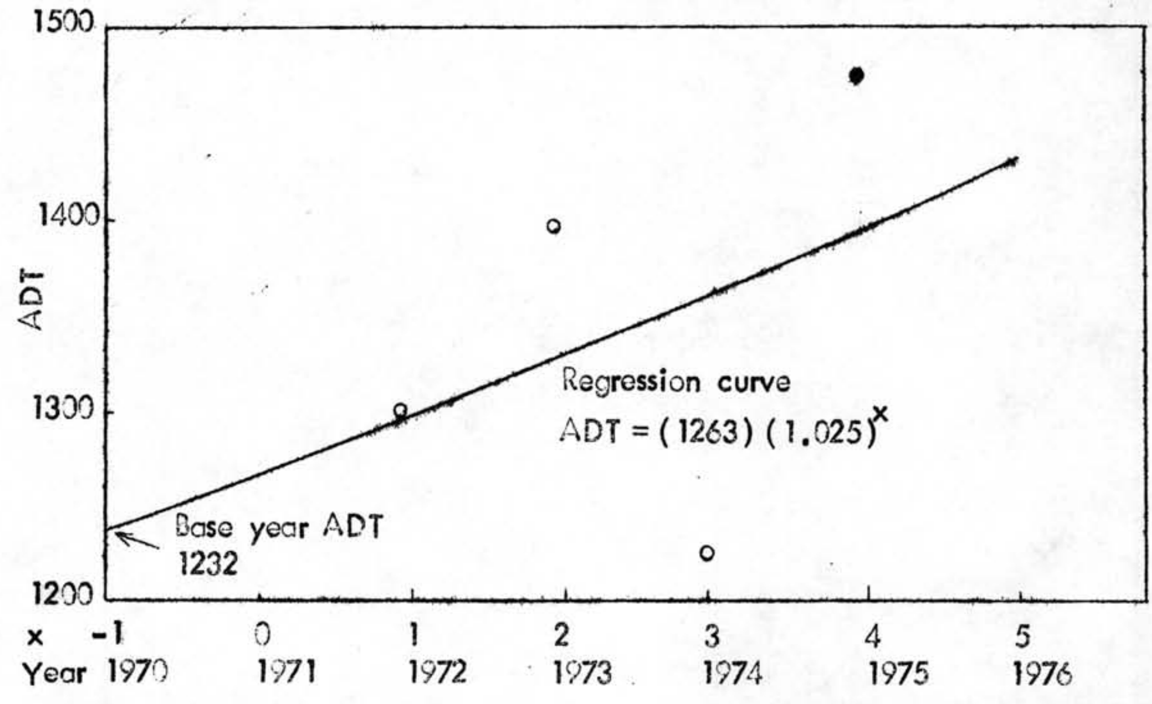


Fig. 14 ADT Observations and Regression Curve, Ban Phu Kae-Lamnarai Segment



on roads narrower than about 5 meters. The resulting relationships between road width and change in speed which are incorporated in the model are summarised below.

For paved roads

Cars and light goods vehicles

$$\triangle V = -(5.0 - W)(7.31) \tag{17}$$

where $\triangle V$ = reduction in speed when the road is less than 5.0 met wide, km/hr ;

W = width of the road, meters.

Medium and heavy goods vehicles and buses

$$\triangle V = -(5.0 - W)(3.29) \tag{18}$$

To determine vehicle speeds on paved roads, the following relationships are used by the model.

Passenger cars

$$V = 102.62 - 0.3724\ RS - 0.07589\ F - 0.1106\ C - 0.00491\ A \tag{19}$$

Light goods vehicles

$$V = 86.88 - 0.4181\ RS - 0.04962\ F - 0.07376\ C - 0.00278\ A \tag{20}$$

Medium and heavy goods vehicles

$$V = 68.06 - 0.5189\ RS + 0.02989\ F - 0.05807\ C - 0.00042\ A \tag{21}$$

Buses

$$V = 72.49 - 0.52558\ RS + 0.06663\ F - 0.06611\ C - 0.00417\ A \tag{22}$$

where V = vehicle speed, km/hr ;

RS = average rise of the road, m/km ;

F = average fall of the road, m/km ;

C = average horizontal curvature of the road, degree/km ;

A = altitude of the road, meters.

Table 7 indicates the allowable range of each variable. If these values are exceeded, the extrapolations are not considered to be reliable. Nevertheless, the results will be computed following the printing of a warning massage.

Table 7 Range of Variables for Vehicle Speeds on Paved Roads

Variable	Allowable range	Units
Rise, RS	0 - 85	m/km
Fall, F	0 - 85	m/km
Horizontal curvature, C	0 - 200	degree/km
Altitude, A	0 - 2500	meters

The speed for each type of vehicle calculated from these formulas using the data for the Saraburi - Lomsak Highway is shown in Table 8.

Table 8 Average Speed for Each Type of Vehicle by the Model Formulas

Vehicle type	Limits of average speed (km/hr)
Cars and taxis	98 - 102
Light trucks	83 - 87
Light buses	69 - 73
Heavy buses	64 - 68
2 - axle trucks	64 - 68
3 - axle trucks	64 - 68

The average speed depends on the rise and fall and horizontal curvature of each section. The values developed by the model are reasonable for a high-stantard primary highway such as the Saraburi – Lomsak Highway.

Fuel Consumption

The effects of speed change cycles were studied by HIDE, H., S.W. ABAYNAYAKA, I. SAYER and R.J. WYATT (1975) and it was concluded that the estimates of fuel consumption derived from the constant – speed experiments should be increased to obtain an estimate of the fuel consumption under normal operating conditions. For cars and light good vehicles, the increase is 8 percent of the constant – speed fuel consumption, and for medium and heavy goods vehicles and buses the increase is 13 percent. Engines and transmission systems of buses and heavy vehicles are very similar and therefore similar performances can be expected. However, buses make frequent stops and starts in normal operation and these can be expected to increase fuel consumption rates. On rural routes, where the frequency of stops and starts is must less than, those on urban routes, the excess fuel consumed is not expected to be greatly different from the excess resulting from the normal speed change cycles of other heavy vehicles.

Fuel Consumption on Paved Roads

Passenger cars

$$FL = (53.36 + \frac{498.67}{V} + 0.0058\,V^2 + 1.594\,RS - 0.8539\,F)(1.08) \qquad (23)$$

Light goods vehicles

$$FL = (74.70 + \frac{1150.51}{V} + 0.0131\,V^2 + 2.906\,RS - 1.277\,F)(1.08) \qquad (24)$$

Medium goods vehicles less than 8.5 tonnes gross vehicle weight

$$FL = \left(105.43 + \frac{902.53}{V} + 0.0143\,V^2 + 4.362\,RS - 1.834\,F - 2.396\,PW \right)(1.13) \qquad (25)$$

Heavy goods vehicles and buses

$$FL = \left(-48.57 + 69.2(GVW)^{1/2} + \frac{902.53}{V} + 0.0143\,V^2 + 1.834\,F - 2.396\,PW\right)(1.13) \qquad (26)$$

Where FL = fuel consumption, liters/1000 km ;

V = vehicle speed, km/hr;

RS = average rise of the road, m/km ;

F = average fall of the road, m/km ;

GVW = gross vehicle weight, tonnes ;

PW = power to weight ratio, brake horsepower/tonne.

The allowable range of each variable is shown in Table 9.

Table 9 Range of Variables for Vehicle Fuel Consumption on Paved Roads

Variable	Allowable Range	Units
Rise, RS	0.- 85	m/km
Fall, F	0 - 85	m/km
Speed, V		
- Cars	20 - 140	km/hr
- Light vehicles	10 - 110	km/hr
- Medium vehicles	5 - 100	km/hr
- Heavy vehicles and buses	5 - 100	km/hr
Gross vehicle weight	3.5 - 40	tonnes
Power/weight ratio	40:1 - 5:1	BHP/tonne

The economic cost of fuel used in the present study was based on the study of T.P.O'SULLIVAN & PARTNERS (1973) which gave petrol at 1.20 Baht per liter and diesel fuel at 0.90 Baht per liter.

In the present study, only the economic cost of fuel was used. It should be noted however, that provision is made in RTIM to input either or both of the economic cost and the market price of fuel. The model will produce results based on either or both of these inputs, deponding on what is provided by the user. The resulting average (economic) cost per kilometer for each class of vehicle obtained from the model is shown in Chapter VII.

Lubricating Oil Consumption

Average figures for total lubricating oil consumption on paved roads as used in the model are as follows.

Passenger cars	1.2 liters/1000 km
Light vehicles	1.8 liters/1000 km
Medium and heavy vehicles and buses	4.0 liters/1000 km

The preceding can be compared with the values which were reported by T.P.O'SULLIVAN & PARTNERS (1973) for operating costs in Thailand. The Thailand values are:

Passenger cars	1.0 liters/1000 km
Light vehicles	1.3 liters/1000 km
Heavy vehicles and buses	2.2 liters/1000 km

The lubricating oil performance figures for cars and small trucks published by T.P.O'SULLIVAN & PARTNERS (1973) are based on IBRD study (DE WEILLE, 1966), and for large trucks on the Thailand study by WILBUR SMITH - LYON ASSOCIATES (1970). In the present study, the economic cost of lubricating oil was taken as 7.10 Baht per liter, as assessed by T.P. O'SULLIVAN & PARTNERS (1973). In calculating the cost of lubricating oil consumption, the model uses the average oil consumption rates that are shown above. It is not practicable for the user to substitute other values.

Age Distribution of Vehicles

The age distribution of vehicles is used in the calculations of depreciation costs and to determine the average age of vehicles in each class. This information is needed to ascertain that parts and maintenance labour costs are not extrapolated beyond applicable limits.

In the model, the calculation of the age distribution of vehicles for each year of the analysis proceeds as follows. (1) The age distribution of vehicles in the first year of the analysis is determined from Eqs. (27) and (28). The analysis is separately performed for each class of vehicle. (2) The number of vehicles of each age lost during the current year is determined from the appropriate wastage equation (Eqs. 29, 30, 31). These losses are subtracted from the number of vehicles of each age which were in use in the previous year. (3) The total number of vehicles in use in the current year, excluding new vehicles is the sum of the previously calculated number of vehicles of each age. (4) The number of new vehicles is obtained by subtracting the total number of vehicles for each class of vehicle in use in the current year from the anticipated ADT value.

The age distribution of the vehicles in use in the first year of the analysis is assumed to be of the following form.

$$n(y) = n_o - \frac{n_o}{Q} y \quad , \quad y \leq Q$$
$$n(y) = 0 \quad , \quad y > Q \tag{27}$$

where $n(y)$ = number of vehicles y years old ;

n_o = number of vehicles which are one year old at the start of the study ;

Q = adjustment factor :

= 10 for cars and light goods vehicles

= 12 for medium and heavy vehicles and buses ,

$$\text{and } n_o = \frac{2N_o}{Q+1} \tag{28}$$

where N_o = total number of vehicles at the beginning of the analysis. In the present study, N_o was taken as the 1970 ADT value for each class of vehicle for the appropriate section of the highway.

Of the vehicles which are new in any one year, the percentage remaining in use in any subsequent year (y) can be calculated from the following equations.

Passenger cars and light vehicles

$$W = 99.5 + 0.37y - 1.2y^2 + 0.039y^3 \tag{29}$$

Medium and heavy vehicles

$$W = 99.7 - 0.11y - 0.93y^2 + 0.032y^3 \tag{30}$$

Buses

$$W = 100 - 0.44y - 0.56y^2 - 0.008y^3 \tag{31}$$

where W = percentage of those vehicles that were new in the base year that are still in use y years later.

The model will automatically calculate the age distribution of vehicles by using the above assumptions. It is not practicable for the user to substitute other age distribution functions.

It should be noted that implicit in this method seems to be the assumption that all annual traffic growth values are attributable to new vehicles coming into service. If correctly interpreted, this procedure thus ignores the effects of induced, generated, and diverted components of traffic which appear on the highway.

Spare Parts Consumption

Spare parts costs are calculated by the model from the following equations :

Passenger cars and light vehicles

$$PC = (0.0018R - 2.03)(VP)(K) \times 10^{-11} \tag{32}$$

Medium and heavy vehicles

$$PC = (0.00037R + 0.48)(VP)(K) \times 10^{-11} \tag{33}$$

Buses

$$PC = (0.0006R - 0.67)(VP)(K) \times 10^{-11} \tag{34}$$

where PC = cost of spare parts, Baht/km ;

R = roughness as measured by a bump integrator towed at 30 km/hr, mm/km ;

VP = price of a typical new vehicle in this class ;

K = distance traveled since new by an average vehicle in the class, km.

The range within which the relationships are valid are shown in Table 10. If any of these variables fall outside the specified range, a warning massage will be printed.

Table 10 Allowable Ranges of the Variables for Spare Parts Consumption

Vchicle type	Variable	
	Age, K, 10^3 km	Roughness, R,mm/km
Cars and light vehicles	0 - 160	0 - 7500
Medium and heavy vehicles	0 - 400	0 - 7500
Buses	0 - 1100	0 - 7500

Maintenance Labour Hours

The relationships used by the model relate labour hours to the cost of spare parts, the new vehicle price, and surface roughness, as shown in Eq.(35).

Passenger cars and light vehicles

$$\begin{aligned} LH &= (851 - 0.078\,R)\left(\frac{PC}{VP}\right), && R \leqslant 6{,}000 \\ &= (383)\left(\frac{PC}{VP}\right), && R > 6{,}000 \end{aligned} \tag{35}$$

Medium and heavy vehicles

$$LH = (2975 - 0.078\,R)\left(\frac{PC}{VP}\right) \quad , \quad R \leqslant 6{,}000$$
$$= (2507)\left(\frac{PC}{VP}\right) \quad , \quad R > 6{,}000 \qquad (36)$$

Buses

$$LH = (2640 - 0.078\,R)\left(\frac{PC}{VP}\right) \quad , \quad R \leqslant 6{,}000$$
$$= (2172)\left(\frac{PC}{VP}\right) \quad , \quad R > 6{,}000 \qquad (37)$$

where LH = number of maintenance labour hours needed per km ;

PC = cost of spare parts per km ;

VP = price of a typical new vehicle in this class.

Tyre Consumption

The relationships used in the model to estimate tyre wear are shown in Eqs. (38) and (39).

Passenger cars and light vehicles

$$TC = (-83 + 0.058\,R) \times 10^{6} \quad , \quad R \geqslant 2{,}000$$
$$= 3.0 \times 10^{-5} \quad , \quad R < 2{,}000 \qquad (38)$$

Medium and heavy vehicles and buses

$$TC = (83 + 0.0112\,R)(GVW) \times 10^{-7} \quad , \quad R \geqslant 1{,}500$$
$$= 1.0\,(GVW) \times 10^{-5} \quad , \quad R < 1{,}500 \qquad (39)$$

where TC = number of tyres consumed per km ;

GVW = gross vehicle weight, tonnes ;

R = roughness, mm/km .

The allowable of roughness for the tyre wear analysis is 0 to 7,500 mm/km.

Vehicle Usage

Table 11 gives average figures for annual vehicle kilometers run by four categories of vehicles in Kenya in 1972. These are used by the model if no information is provided by the model user.

Table 11 Average Anual Vehicle Kilometrage Used by the Model

Vehicle type	Annual kilometers of operation
Passenger cars	20,000
Light vehicles	45,000
Buses	90,000
Heavy vehicles	75,000

Vehicle usage in Thailand was studied by T. P. O'SULLIVAN & - PARTNERS (1973). The results of that study are shown in Table 12 and were used in the present application of RTIM to the Saraburi – Lomsak Highway.

Table 12 Average Annual Vehicle – Kilometers Used in the Present Study

Vehicle type	Annual kilometers of operation
Passenger cars	18,000
Light buses	35,000
Light trucks	25,000
Heavy buses	70,000
Heavy trucks	70,000

Depreciation

Depreciation per vehicle - kilometer can be expressed as a fraction of the new vehicle price for each year in the life of a vehicle. Thus, in the model, depreciation is calculated as follows.

Passenger cars and light vehicles

$$\begin{aligned} \text{Annual depreciation per km} &= \frac{0.22}{K_A}(VP) && \text{for vehicles 1 year old} \\ &= \frac{0.14}{K_A}(VP) && \text{for vehicles 2 years old} \\ &= \frac{0.08}{K_A}(VP) && \text{for vehicles 3 - 8 years old} \\ &= 0 && \text{for vehicles over 8 years old} \end{aligned} \tag{40}$$

Medium and heavy vehicles

$$\begin{aligned} \text{Annual depreciation per km} &= \frac{0.31}{K_A}(VP) && \text{for vehicles 1 year old} \\ &= \frac{0.625}{K_A}(VP)\left[(Y)^{\frac{1}{3}} - (Y-1)^{\frac{1}{3}}\right], && 1<y<8 \\ &= 0 && 8 \leqslant y \end{aligned} \tag{41}$$

where K_A = average annual kilometrage ;

VP = cost of an equivalent new vehicle ;

y = age of vehicle, years.

Commercial Vehicle Crew Hours

In the model, crew costs are determined from the crew hours worked, expressed in terms of hours per kilometer of operation.

Crew hours per km = (Average annual crew hours) / K_A ,

where K_A = average annual kilometrage.

If the model user has no estimate for the average anual crew hours, the model will use the values given in Table 13.

Table 13 Annual Crew Hours

Vehicle type	Average number of crew hours per year
Passenger cars	0
Light vehicles	2,000
Buses	6,000
2 - axle trucks	7,500
All other heavy vehicles	7,500

Interest Charges

If it is necessary for the model user to estimate the interest charge per vehicle kilometer, the following method is used ; it is based on the approach suggested by DE WEILLE (1966).

$$\text{Interest charge per km} = \frac{(\text{Rate of interest})(0.5 \times \text{New vehicle price})}{K_A} \qquad (42)$$

where K_A = average annual kilometrage.

If interest charges are not to be included in the vehicle operating cost, the interest rate should be set at zero.

It was suggested by T. P. O'SULLIVAN & PARTNERS (1973) that the interest charge be assessed on half the depreciable value[1] of the typical

[1] The depreciable value is taken as the price of a new vehicle.

vehicle. This reflects the age structure and hence the outstanding capital value of the vehicles in use. When the number of vehicles increases annually at as it has been in Thailand by 14 percent per annum, the age of the typical vehicle is less than half the average expected life. It was assumed by T. P. O'SULLIVAN & PARTNERS that the vehicle population would grow at about 8 percent per annum through the decade of the 1970's, so the average vehicle value would be 55 percent of the average depreciable value of each type of vehicle. This value is close to the typical vehicle value of 50 percent of new vehicle price used in the model.

A 12 percent per annum rate of interest is used in this study. This is the normal commercial bank interest to safe customers for vehicle purchases in Thailand, and is the rate used by the Highway Department for its economic feasibility studies.

Standing Charges

Standing charges include road tax, licence fees, insurance, etc. Unless the model user has better information available, it is assumed by the model that the standing charges for passenger cars are 10 percent of total running costs, and for all commercial vehicles they are 25 percent of total running costs. Standing charges are expressed in terms of cost per km as follows :

$$\text{Standing charges} = \frac{(\text{Running costs})(\text{Overhead coefficient})}{K_A} \qquad (43)$$

where K_A = average annual kilometrage.

Overhead coefficient = 0.10 for passenger cars,

= 0.25 for all goods vehicles and buses.

Passenger Time Costs

An hourly rate for the value of time may be specified for each vehicle class by the model user. The time taken for a journey over the road link in each direction is found from the average speed of the class of vehicle and length of the road segment. The average of these journey times is the time which is costed.

In the present study, the rate for the value of time was obtained from T.P. O'SULLIVAN & PARTNERS (1973).