Chapter 6. ENGINEERING SURVEYS AND ANALYSES

6.1 General

In order to obtain necessary technical data and information on the Study area, the following field surveys were carried out by the local consultants and engineers:

1) Topographic survey
2) Hydrological survey
3) Geological survey
4) Materials survey

6.2 Topographic Survey

For preliminary engineering design, the following topographic surveys were carried out:

1) Control points survey using Global Positioning System (GPS)
2) Ground survey using Total Station

6.2.1 Control Points Survey

Prior to the ground survey, 18 control points were established in the Project area by using Global Positioning System (GPS).

6.2.2 Ground Survey Using Total Station

Ground survey was carried out along the Project Road covering the area necessary for the alignment study and design as listed below.

- Detailed topographic survey along the N-55 between Kohat Toi to Dara Adam Khel excluding the middle part of the Kohat tunnel (1.5 km)
- Survey of the intersections at Kohat Toi and Dara Adam Khel and three interchanges on the N-55 between Kohat Toi to Dara Adam Khel
- Survey of the portal areas at both ends of the Kohat Tunnel including the control center at the south portal
- Topographic survey for the Alternative Route between Sta.17+500 and Sta.20+000 (Kohat Tunnel south portal, approximately 2.5 km) selected by the Engineer

Total Station was used for the survey to obtain the data in digital form.

The elevation and co-ordinates of survey spots for topographic mapping were measured by radiation method based on the traverse points established from the co-ordinates of the GPS stations. The obtained data include the following:

- Ground details including carriageways, shoulders, pavement edges, inner and outer shoulders, embankment and cut edges and trees, drains, slope protection work, guard-rail, toll plaza, irrigation channels, ROW, property lines, building lines, central reserves, and median barriers.
- Overhead and ground utilities including electrical, telephone poles and cables, storm water drainage, sewerage, water supply, gas pipelines, optical fibre cables, trees and all other means of services. The heights and depths of utility lines crossing the carriageway shall be determined.
- Buildings, bridges, tunnels, pipe and box culverts, underpasses and other developments.

The recorded data from total stations were downloaded onto computers and topographic maps with contours were prepared using AutoCAD system. These digital maps contain three-dimensional properties and can be used to produce three dimensional digital terrain modelling of the surveyed area.

6.3 Hydrological Study

6.3.1 General

A hydrological study was carried out to confirm or obtain the information on the design discharge, flood water level and scouring depth for the cross drainage structures, especially for river bridges (listed in Table 6.3.1), which had been used for the design of the 1st Kohat Tunnel and Access Road and their effectiveness for the design of the 2nd Kohat Tunnel Access Road.

The scope of work for the hydrological survey covered the follows activities:

- Field Survey
- Data Collection and Processing
- Hydrological Analysis
- Evaluation of Data and Analysis.

Table 6.3.1 List of River Bridges

Bridge No.	Station (at center)	Bridge Length (m)	Span	Remarks (Name of River)
1 R	2+736.245	120	4 - 30m Span	Kohat Toi (Jerma Minor)
2 R	4+735.415	50	2 - 25m Span	Chargai Algada
5 R	18+935.415	80	25m+30m+25m	Osti Khel Algad
6A R	21+260.525	180	6-30m Span	Osti Khel Algad & Panderi Algada
7 R	25+388.915	40	2-20m Span	Mullah Khel Algad

Another objective of the study is to evaluate the effects of creeks located on the right and left sides of the planned tunnel south portal.

The study was carried out by local engineers under the guidance and supervision of the JICA Study Team. The following methodology was adopted for the study:

- Rainfall data for the Kohat and Peshawar Stations are collected from the Pakistan Meteorological Department (PMD) for periods of 1951-2005 and 1950-2005 respectively. Other meteorological data on temperature, wind and humidity are also collected.
- Topographic maps produced by the Survey of Pakistan (SOP) on a scale of 1:50,000 and 1:250,000 are used for estimating catchment areas.
- Hydro-meteorological data are obtained from PMD. Discharge data are mostly available at the Irrigation Department of NWF Province and Surface Water Hydrology (SWH) are from the Water & Power Development Authority (WAPDA).
- River cross sections are obtained from topographic surveys and river profiles are obtained from 1:50,000 maps.
- Processing and analysis of the above data are conducted by appropriate methods.

6.3.2 Climate

The climate of the Project area is hot and arid and can be divided into the following four seasons based on temperature and rainfall:

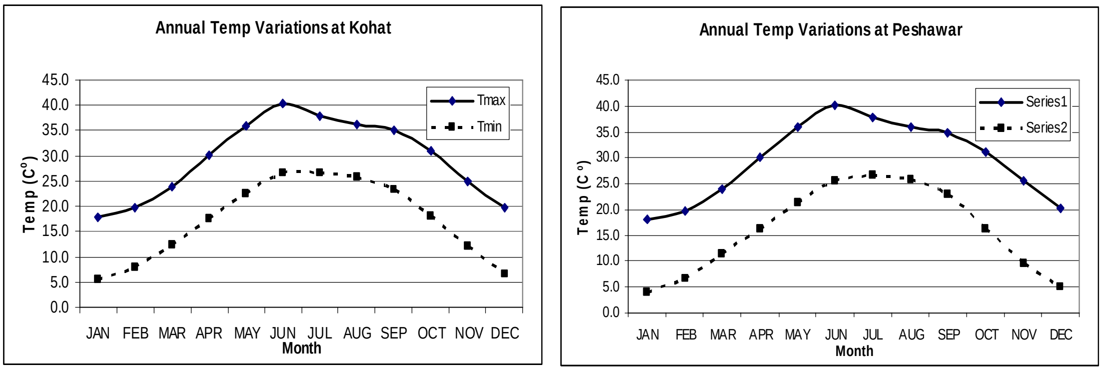
- Cold weather season: December - March
- Hot weather season: April - June
- Monsoon season: July - September
- Post-monsoon or Autumn season: October - November

(1) Temperature

The monthly mean of daily maximum and minimum temperatures at Kohat for the period of 1954 to 2005 and at Peshawar for the period of 1950 to 2005 is as follows:

	Kohat	Peshawar
Mean Max. Temperature (C^o):	40.3 (in June)	40.2 (in June)
Mean Min. Temperature (C^o):	5.5 (in January)	4.1 (in January)

The temperature variations in the Project area (Kohat and Peshawar) are shown in Figure 6.3.1. Temperature starts risings in January, reaches the maximum in June and then declines until December.



Source: JICA Study Team

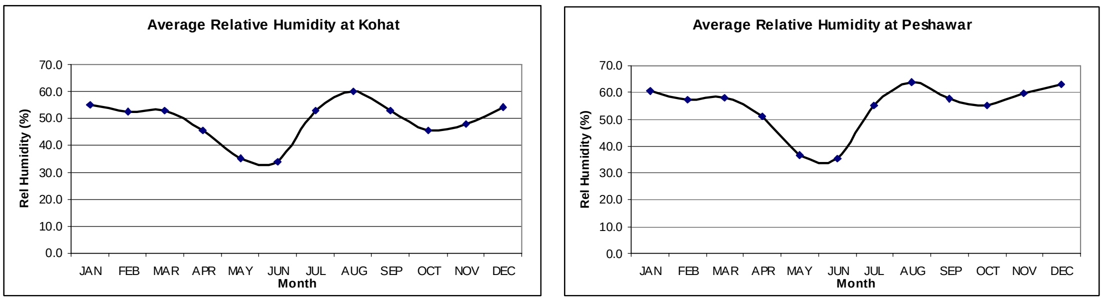
Figure 6.3.1 Annual Temperature Variations in the Project Area

(2) Relative Humidity

The average monthly relative humidity (RH) at Kohat (1954 - 2005) and Peshawar (1950 - 2005) is given below.

	Kohat	Peshawar
Mean Max. RH (%):	60 (in August)	64 (in August)
Mean Min. RH (%):	34 (in June)	35 (in June)

The relative humidity at Kohat and Peshawar, shown in Figure 6.3.2, is stable from January to March, decreases until June, increases sharply until August, decreases until October, then starts increasing again in November and December.



Source: JICA Study Team

Figure 6.3.2 Average Relative Humidity in the Project Area

6.3.3 Rainfall

The monthly rainfall data of the Kohat station are available for the period of 1954 to 2005 and those of the Peshawar station for the period of 1950 to 2005. The average annual rainfall at Kohat and Peshawar is 572 mm and 423 mm respectively. The minimum annual rainfall of 234 mm occurred in 1986 and the maximum annual rainfall of 1003 mm occurred in 2003 at Kohat. The minimum annual rainfall of about 174 mm occurred in 1952 and the maximum annual rainfall of 905 mm occurred in 2003 at Peshawar. The monthly average, monthly minimum and monthly maximum rainfalls at Kohat and Peshawar are summarised in Table 6.3.2 and graphed in Figure 6.3.3.

Table 6.3.2 Average Annual Rainfall (1950-2005) in the Project Area

Unit: mm

Month	Kohat			Peshawar		
	Average	Minimum	Maximum	Average	Minimum	Maximum
JAN	30.9	8.8	46.0	33.7	19.6	33.0
FEB	45.9	29.1	135.0	46.5	52.6	131.5
MAR	80.3	38.3	103.0	76.5	44.5	66.0
APR	54.6	18.7	85.0	52.0	19.0	129.0
MAY	37.4	3.9	13.0	24.8	5.1	23.0
JUN	24.9	21.1	55.0	11.3	0.0	10.0
JUL	79.0	25.4	210.0	44.4	0.3	156.0
AUG	107.6	40.5	182.0	57.1	0.5	114.0
SEP	49.5	6.4	108.0	24.6	20.8	111.0
OCT	27.2	8.2	10.0	17.7	0.5	70.0
NOV	11.8	18.2	31.0	13.7	0.0	42.0
DEC	22.3	15.1	25.0	20.6	10.9	19.0
Total	571.9	233.7	1,003.0	423.0	173.8	904.5

Source: Pakistan Meteorological Department (PMD)

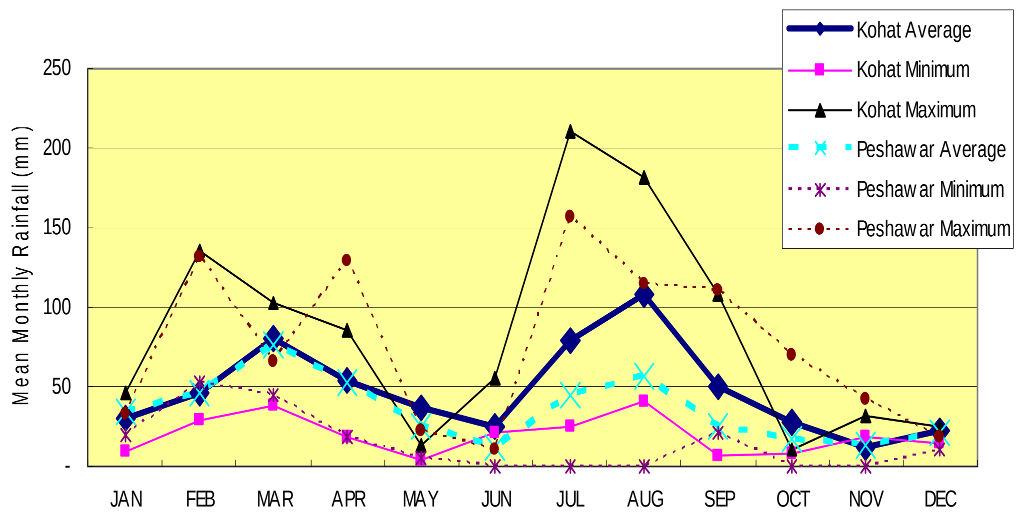


Figure 6.3.3 Average Monthly Rainfall in the Project Area

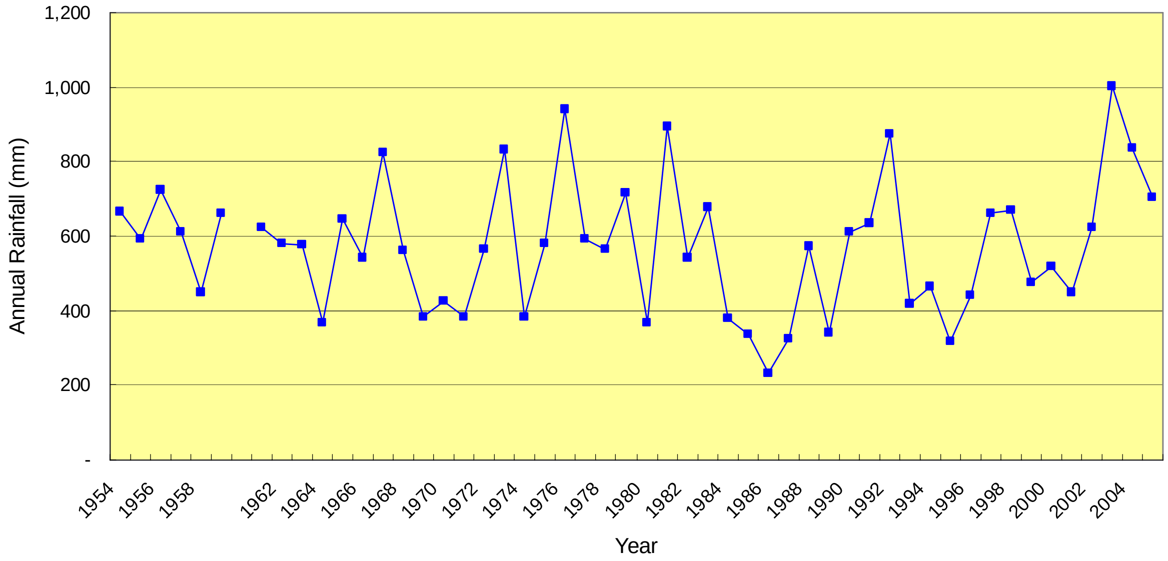
The annual rainfall at Kohat varies mostly from 400 mm to 900 mm (see Table 6.3.3). The highest monthly rainfall between 1950 and 2005 was 419.6 mm in August 1976. No significant increasing or decreasing trend was noticed in the last 50 years.

Table 6.3.3 Monthly Rainfall (1954-2005) at Kohat Station

Unit: mm

YEAR	JAN	FEB	MAR	APR	MAY	JUN	JUL	AUG	SEP	OCT	NOV	DEC	TOTAL
1954	107.4	103.6	41.4	17.3	48.8	56.6	134.3	62.7	45.7	40.4	7.9	0.0	666.1
1955	0.0	3.0	8.4	3.0	38.9	7.6	49.0	392.7	73.9	5.1	0.0	10.4	592.0
1956	2.3	26.9	178.6	31.5	0.0	59.2	246.9	51.1	99.3	25.1	1.3	1.3	723.5
1957	50.8	0.8	112.8	104.4	52.1	9.4	45.7	76.5	40.6	37.8	53.1	26.7	610.7
1958	18.0	7.1	67.6	18.8	5.1	7.9	85.3	59.2	59.2	1.5	4.6	115.6	449.9
1959	69.8	95.3	33.0	65.3	31.8	2.3	79.0	46.2	110.0	41.4	71.6	15.2	660.9
1960	15.0	1.0	114.6			18.0	109.7	68.0	80.2	1.3	0.0	44.7	
1961	110.0	26.2	16.3	94.7	28.2	46.7	73.7	68.3	41.1	84.3	33.0	2.5	625.0
1962	13.2	37.6	61.7	36.8	55.6	37.1	74.2	95.3	60.2	13.0	19.3	75.4	579.4
1963	0.0	38.9	130.0	114.3	64.5	9.1	25.1	28.2	50.8	57.9	23.4	32.8	575.0
1964	45.2	17.3	47.2	46.0	31.0	54.6	38.9	20.1	35.3	8.1	0.0	23.9	367.6
1965	21.1	76.7	48.5	201.2	80.8	28.7	56.1	46.7	38.6	13.0	28.2	5.1	644.7
1966	0.0	98.8	101.3	126.7	37.3	19.0	33.8	21.6	24.1	81.0	0.0	0.0	543.6
1967	0.0	50.5	239.5	38.6	18.0	15.2	119.9	92.5	11.9	61.0	22.1	154.7	823.9
1968	44.7	22.1	92.7	37.6	49.8	39.6	32.8	136.1	7.1	79.8	10.9	7.4	560.6
1969	0.0	52.8	38.1	36.1	43.7	18.3	59.7	58.9	30.0	46.0	0.0	0.0	383.6
1970	16.3	38.6	59.7	23.6	18.5	36.1	67.6	122.9	33.8	0.5	0.0	6.3	423.9
1971	4.8	29.5	21.1	84.1	18.0	38.1	106.2	55.9	19.8	3.6	0.3	2.5	383.9
1972	42.9	82.8	48.3	59.4	26.9	11.4	30.0	141.7	34.0	40.1	13.7	34.8	566.0
1973	7.4	66.0	115.3	14.0	66.3	6.6	115.3	308.1	109.2	9.1	0.0	15.2	832.5
1974	5.3	36.6	25.6	61.4	30.7	0.0	25.4	42.0	112.0	0.0	0.0	42.4	381.4
1975	4.9	59.0	88.4	65.2	73.7	14.5	134.0	116.8	21.1	0.0	0.0	3.3	580.9
1976	14.9	93.6	38.1	67.2	50.2	20.9	102.1	419.6	78.8	56.3	0.0	0.0	941.7
1977	56.3	14.2	8.0	70.3	65.6	4.9	81.9	103.5	91.1	70.0	12.8	13.5	592.1
1978	21.4	6.1	216.3	23.5	1.3	6.4	156.2	87.1	31.2	6.3	7.6	2.8	566.2
1979	41.3	41.9	34.4	14.5	69.8	8.7	94.0	356.2	14.8	13.2	17.7	11.2	717.7
1980	53.2	79.8	67.1	9.9	4.4	9.2	26.7	14.0	63.4	8.4	26.7	4.6	367.4
1981	29.7	35.4	364.9	36.8	21.2	0.8	100.8	249.9	19.9	30.2	5.8	0.0	895.4
1982	55.4	25.0	200.7	28.4	28.4	18.0	41.3	56.0	30.1	8.9	38.6	11.8	542.6
1983	36.3	46.9	91.5	195.2	42.9	14.0	19.5	146.8	36.5	42.8	3.0	1.0	676.4
1984	1.3	14.8	43.0	16.8	16.0	10.0	71.4	158.1	14.9	1.3	25.6	7.7	380.9
1985	34.5	1.5	31.7	51.2	29.0	24.2	17.7	76.9	5.1	5.8	10.5	48.8	336.9
1986	8.8	29.1	38.3	18.7	3.9	21.1	25.4	40.5	6.4	8.2	18.2	15.1	233.7
1987	0.0	23.5	64.1	5.2	39.2	36.3	67.0	57.0	23.0	7.0	0.0	1.0	323.3
1988	16.8	16.6	131.0	33.0	34.0	34.0	124.0	61.0	59.0	13.5	0.0	49.0	571.9
1989	16.3	8.0	42.1	10.0	9.2	19.2	61.0	90.0	52.0	1.0	2.0	30.0	340.8
1990	38.0	92.0	77.5	68.0	6.0	6.0	105.0	69.0	53.0	13.0	9.0	76.0	612.5
1991	20.0	77.0	125.0	128.2	117.0	35.0	51.0	55.0	17.4	7.0	0.0	1.0	633.6
1992	68.0	59.0	113.0	102.0	113.0	19.0	44.0	231.0	83.0	18.0	9.0	14.0	873.0
1993	11.0	7.0	139.5	34.0	15.0	18.0	134.0	22.0	17.0	7.0	12.5	0.0	417.0
1994	5.0	46.0	52.0	60.0	35.0	7.0	92.0	20.0	48.0	32.0	12.0	57.0	466.0
1995	8.0	32.0	49.0	61.0	5.0	14.0	52.0	41.0	24.0	18.0	5.0	9.0	318.0
1996	35.0	43.0	60.0	17.0	31.0	50.0	34.0	107.0	9.0	55.0	1.0	0.0	442.0
1997	7.0	18.0	19.0	129.0	92.0	24.0	88.0	102.0	31.0	100.0	9.0	42.0	661.0
1998	27.0	163.5	72.5	71.0	52.6	10.5	89.5	69.5	79.5	33.0	0.0	0.0	668.6
1999	122.0	27.0	67.0	4.0	30.5	14.0	84.0	66.0	29.0	8.0	24.0	0.0	475.5
2000	45.5	23.5	33.0	12.5	69.0	32.0	85.0	104.0	51.5	49.0	0.0	12.0	517.0
2001	0.0	0.0	36.5	64.5	19.6	66.2	98.0	71.5	61.5	29.0	0.0	3.5	450.3
2002	2.0	56.0	34.0	22.0	8.0	85.0	14.0	241.0	63.0	53.0	2.0	45.0	625.0
2003	46.0	135.0	103.0	85.0	13.0	55.0	210.0	182.0	108.0	10.0	31.0	25.0	1003.0
2004	76.5	59.0	18.0	51.0	0.0	82.0	154.0	140.0	117.0	47.0	37.0	56.0	837.5
2005	133.0	143.0	115.0	15.0	65.0	13.0	42.0	44.0	119.0	10.0	4.0	0.0	703.0
Average	30.9	45.9	80.3	54.6	37.4	24.9	79.0	107.6	49.5	27.2	11.8	22.3	571.9

Source: Pakistan Meteorological Department (PMD)



6.3.4 Estimate of Design Discharge

(1) Design Criteria and Methodology (US-SCS Method)

The design discharge with a 50-year return period was adopted in this study for the planned bridges and south portal creeks. Due to the absence of flood discharge data, the synthetic triangular hydrograph technique was applied. The U.S. Soil Conservation Services (US-SCS) approach was used to synthesize flood hydrograph by using the Curve Number method. The US-SCS Unit Hydrograph Method was used to estimate the peak discharge at the bridge locations. This method requires the following information about the catchments.

- Maximum 24-hour rainfall for the design return period
- Length of nullah (dry stream) measured along the longest path from the head to the site
- Slope of nullah (dry stream)
- Catchment area
- Antecedent soil moisture condition
- Soil group
- Cover complex classification.

The length and slope of streams and the catchment areas were determined from available G.T. Sheets on 1:50,000 scale. The Curve Number was determined from information gathered from the field visit and from G.T. Sheets.

The maximum discharge is calculated from the following formulae:

S	=	1000/CN - 10
Q	=	$(P-0.2S)^2/(P + 0.8S)$
tc	=	1/7700 x (L/S^0.5)^0.77
ΔD	=	0.133 x tc
tp	=	D/2 + 0.6 x tc
Qp	=	484 x A x Q/ tp

Where,

S	=	Potential maximum retention in inches
CN	=	Curve number
Q	=	Volume of runoff in inches
P	=	Maximum 24-hour rainfall in inches of required return period
L	=	Length of longest stream in feet

H	=	Difference of elevation in feet
tc	=	Time of concentration in hours as per Kirpich's equation
tp	=	Time to peak in hours
ΔD	=	Unit storm duration in hours
Qp	=	Peak rate of flow in cusecs
A	=	Catchment area in sq. miles

(2) Rational Method

For the two small creeks at the south portal, the rational method was used as their catchment areas are smaller than 10 sq. miles. The peak discharge was calculated by the following equation:

$$q = CiA$$

Where,

q = Discharge (ft^3/sec)
i = Rainfall intensity (in/hr)
A = Watershed area in acres
C = Runoff coefficient.

The rainfall intensity was estimated by using the Mononobe's formula. The estimated rainfall intensity for the west and east creeks of the tunnel south portal are 4.09 in/hr and 4.06 in/hr respectively. The "C" value adopted for the catchments is 0.8.

(3) 24-hour Annual Maximum Rainfall

To estimate the design discharges, the one day annual maximum rainfall at Kohat for the period of 1951 to 2005 (55 years) shown in Table 6.3.4 has been used.

Table 6.3.4 Maximum One Day Rainfall (1951-2005) in the Project Area

Year	One day (24-hr) Annual Maximum Rainfall (mm)	Year	One day (24-hr) Annual Maximum Rainfall (mm)	Year	One day (24-hr) Annual Maximum Rainfall (mm)
1951	45	1971	29	1991	56
1952	29	1972	72	1992	102
1953	69	1973	83	1993	52
1954	51	1974	45	1994	26
1955	93	1975	49	1995	21
1956	94	1976	118	1996	40
1957	74	1977	37	1997	52
1958	54	1978	100	1998	48
1959	58	1979	112	1999	38
1960	35	1980	37	2000	40
1961	55	**1981**	**120**	2001	51
1962	34	1982	40	2002	108
1963	27	1983	39	2003	66
1964	24	1984	39	2004	75
1965	37	1985	22	2005	75
1966	31	1986	18		
1967	**178**	1987	36		
1968	46	1988	42		
1969	49	1989	50		
1970	41	1990	35		

Source: Pakistan Meteorological Department (PMD)

The maximum one day rainfall of 178 mm occurred in March 1967. The next highest value of 120 mm was recorded in 1981. A frequency analysis of 55-year time series was carried out using the Weibull's plotting position formula and the Gumbel's extreme value type-I distribution. The plotted data and the related line are shown in Figure 6.3.4 and the results of frequency analysis are summarized in Table 6.3.5.

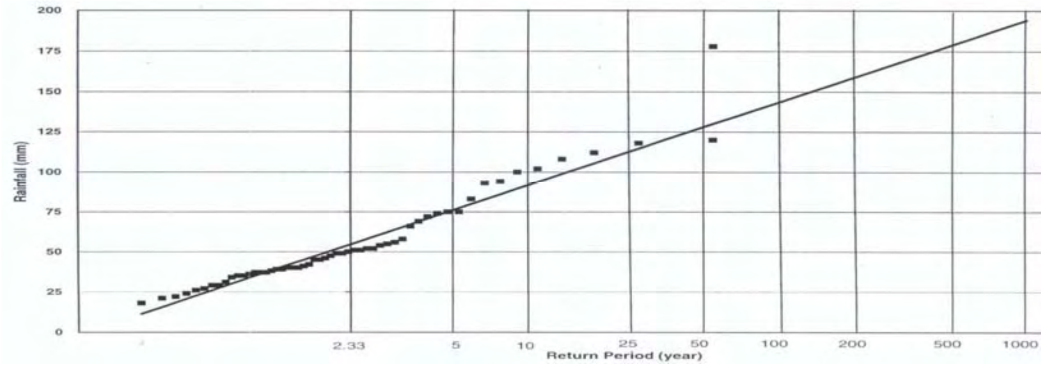


Figure 6.3.4 Frequency Analysis of One-Day Annual Maximum Rainfall

Table 6.3.5 Probable Rainfall by Return Period

Return Period Year	Probable Rainfall		Conversion One day to 24 hours Rainfall	Remarks (1st Kohat Design)
	Inches	mm		
10	3.6	91	103	107
25	4.4	112	127	128*
50	5.04	128	145	155
100	5.64	143	162	175

Note: * for 20 year return period

For converting the one day rainfall to 24-hour rainfall, a multiplication factor of 1.13 was used as found in various studies in Pakistan and in the Manual for Estimation of Probable Maximum Precipitation by the World Meteorological Organization.

No substantial difference is seen in the probable rainfall between the 1st Kohat Tunnel and Access Road and this survey.

(4) Catchment Characteristics

The catchment characteristics of the bridges under study and the two creeks on the south portal of the tunnel were derived from the Survey of Pakistan maps on 1:50,000 and 1:250,000 scale. The catchment characteristics including catchment area, length of stream, slope, time of concentration, hydrograph time step ΔD and lag time for all the locations are given in Table 6.3.6.

Table 6.3.6 Catchment Characteristics

Bridge No.	Stream Name	Catchment Area		Length 'L'	Slope 'S'	Sq. Root	Tc	d	Lag Time	Remarks*
		(sq km)	(sq mile)	ft		of 'S'	hr	hr	hr	Catchment Area (sq mile)
1	Kohat Toi (Jerma Minor)	1,500.0	579.2	354,600	0.01189	0.1090	13.42	1.79	8.05	378.0
2	Chargai Algad	99.0	38.2	65,620	0.01056	0.1028	3.83	0.51	2.30	30.5
5	Bosti Khel	29.0	11.2	45,280	0.01900	0.1378	2.30	0.31	1.38	10.6
6A	Bosti Khel Algad & Panderi Algada	99.0	38.2	88,600	0.01745	0.1321	3.98	0.53	2.39	-
7	Malla Khel Algada	116.0	44.8	108,770	0.01388	0.1178	5.09	0.68	3.05	46.5
South Portal Creek (West)		2.5	1.0	2,560	0.43030	0.6560	0.08	0.01	0.05	-
South Portal Creek (East)		2.7	1.0	2,800	0.54710	0.7397	0.07	0.01	0.04	-

Note: * from the design review report of the 1st Kohat Tunnel and Access Road

The catchment areas are almost same between the 1st Kohat Tunnel Access Road and this study except for the Kohat Toi River (as shown in the above table).

(5) Design Discharge

By using the estimated catchment parameters and rainfall, the flood hydrographs for the design return period (50 years) were synthesized through a worksheet computer program. A summary of the design discharge is shown in Table 6.3.7. There is no substantial difference between the 1st Kohat Tunnel and Access Road except Kohat Toi (Jerma Minor) in the design discharge for the rivers where bridges are constructed. The design discharge (55-57 m^3/sec) of creeks located at the tunnel south portal is relatively large against the catchment areas because of steep slopes. Sufficient drainage capacity should be provided both for water and debris flows. Protection against scouring is necessary for the lower part of embankments.

Table 6.3.7 Design Discharge

Site		Name of Main River / Stream	50 Year Peak Discharge (cumecs)	Remarks* 1st Kohat Design (cumecs)
Bridge No. 1	2+736.245	Kohat Toi (Jerma Minor)	5,030	1,600
Bridge No. 2	4+735.415	Chargai Algada	460	378
Bridge No. 5	18+920.415	Bosti Khel	158	243
Bridge No. 6A	21+260.525	Bosti Khel & Panderi Algada	520	-
Bridge No. 7	25+388.915	Malla Khel Algada	615	656
South Portal (West)		a creek	57	-
South Portal (East)		a creek	55	-

Note: * from the design review report of the 1st Kohat Tunnel and Access Road

It is noticed that there is a large difference between the discharge estimated by this study and that by the 1st Kohat Tunnel and Access Roads Project. This is originated from different catchment areas estimated by both studies (refer to Table 6.3.6). However, as a flood water storage function of the Tanda Dam, which is located at upstream of the Kohat Toi River is counted, the new bridge length will not be necessary to change from the existing bridge length.

6.3.5 Hydraulic Study

(1) Flood Water Level

To calculate the flood water levels, the cross sections and river slopes were obtained from the topographic survey data carried out under this FS Study. Table 6.3.8 shows the flood water levels computed on a mathematical model called HEC-RAS.

Table 6.3.8 Flood Water Level

Site		Name of River	River Bed Elevation (m)	FWL Elevation (m)	Remarks (FWL)* 1st Kohat Design (m)
Bridge No. 1	2+736.245	Kohat Toi (Jerma Minor)	444.0	456.7	449.8
Bridge No. 2	4+735.415	Chargai Algada	437.1	441.3	441.4
Bridge No. 5	18+920.415	Bosti Khel	691.8	693.5	692.8
Bridge No. 6A	21+260.525	Bosti Khel & Panderi Algada	672.9	674.9	-
Bridge No. 7	25+388.915	Malla Khel Algada	631.6	637.2	634.1

Note: * from the design review report of the 1st Kohat Tunnel and Access Road

(2) Scour Depth

The design scour depth of the river bed for the bridges on the 1st Kohat Tunnel Access Road Project is as follows (Table 6.3.9):

Table 6.3.9 Design Scour Depth of River Bridges on the 1st Kohat Tunnel Access Road

Site		Name of River	Scour Depth (m)	Scour Depth from HFL	
				Abutment (m)	Pier (m)
Bridge No. 1	2+736.245	Kohat Toi (Jerma Minor)	3.3	4.9	6.6
Bridge No. 2	4+735.415	Chargai Algada	2.7	4.1	5.4
Bridge No. 5	18+920.415	Bosti Khel	1.8	2.6	3.5
Bridge No. 6A*	21+260.525	Bosti Khel & Panderi Algada	-	-	-
Bridge No. 7	25+388.915	Malla Khel Algada	2.5	3.7	4.9

Notes: 1. From the design review report of the 1st Kohat Tunnel and Access Road
2. * No scour depth analysis for Bridge No.6A in the original design report

Since the scour depth for Bridge No.6A is not available in the design report of the 1st Kohat Tunnel and Access Roads (this bridge was not included in the original design list), its scour depth analysis was conducted in this study. The Lacey's method was used for score depth analysis. The discharge concentration 'q' was increased by 20% to cater for the curve in the nullah/stream and variation in discharge concentration over the cross section. The D50 required for the analysis was determined from the particle size distribution of the bed material. Two test pits were excavated at the bridge location to identify the particle size distribution at laboratory.

The formulae and computation results of scour depth computation are shown in Table 6.3.10 for piers and Table 6.3.11 for abutments of Bridge No.6A.

Table 6.3.10 Scour Depth Calculation for Piers of Bridge No.6A

Description	Formula	US Costmary Unit		Metric Unit	
Discharge	Q	18,364.00	cusecs	520.00	cumecs
Width of flow	B	251.00	ft	76.46	m
Discharge Concentration	q = Q/B	73.20	cusecs/ft	6.80	cumecs/m
20 % increase in Discharge Consent.	$q_{1.2}$	87.85	cusecs/ft	8.16	cumecs/m
Median grain size	D_{50}	0.07	mm	0.07	mm
Lacey's Silt Factor	$f = 1.76\,(D_{50})^{1/2}$	2.00		2.00	
Scour Depth	$R = [(q^2/f)]^{1/3}$	15.68	ft	4.78	m
F.O.S	F_S	1.75			
Scour Depth	$R = F_s \times [(q^2/f)]^{1/3}$	27.45	ft	8.37	m
Minimum Channel Elevation	MCE	2,207.59	ft	672.84	m
Water Surface Elevation	WSE	2,214.22	ft	674.86	m
Scour Extents	S=WSE-R	2,186.77	ft	666.49	m
Difference of Scour Elevation					
and Min. Channel Elevation	MCE-S	20.82	ft	**6.34**	**m**

Table 6.3.11 Scour Depth Calculation for Abutments of Bridge No.6A

Description	Formula	US Costmary Unit		Metric Unit	
Discharge	Q	18,364.00	cusecs	520.00	cumecs
Width of flow	B	250.90	ft	76.46	m
Discharge Concentration	q = Q/B	73.20	cusecs/ft	6.80	cumecs/m
20 % increase in Discharge Consent.	$q_{1.2}$	87.85	cusecs/ft	8.16	cumecs/m
Median grain size	D_{50}	40.00	mm	40.00	mm
Lacey's Silt Factor	$f = 1.76\,(D_{50})^{1/2}$	11.13		11.13	
Scour Depth	$R = [(q^2/f)]^{1/3}$	8.85	ft	2.70	m
F.O.S	F_S	1.75		1.75	
Scour Depth	$R = Fs \times [(q^2/f)]^{1/3}$	15.49	ft	4.72	m
Minimum Channel Elevation	MCE	2,210.08	ft	673.64	m
Water Surface Elevation	WSE	2,219.99	ft	676.66	m
Scour Extents	S=WSE-R	2,204.50	ft	671.94	m
Difference of Scour Elevation					
and Min. Channel Elevation	MCE-S	5.58	ft	**1.70**	**m**

As scouring to a considerable depth will occur, appropriate scouring protection works are necessary for all river bridges. It is necessary to reduce the current foundation elevation for the piers of Bridge No.1 as the actual scouring depth occurred is larger than that calculated in the original design.

6.4 Geological Survey

6.4.1 Boring Investigation

(1) Result of Drilling Work

Drilling work was carried out at the sites of the north portal, south portal, bridge No.4 and bridge No.1. A location map is shown in Figure 6.4.1 and the quantity of works in Table 6.4.1. Rotary boring machine was used for rock drilling at BH No.1 and BH No.2, and percussion machine was utilized for gravel drilling. Results of boring are shown in the form of drilling logs in Appendix.

Table 6.4.1 Quantity of Boring Work

BH No.	Elevation (m)	Location	Drilling Depth (m)	SPT (Times)	Type of Boring
1	726.20	North Portal	15.0	0	Rotary
2	672.00	South Portal	15.0	0	Rotary
3	597.00	Bridge No.4	16.0	4	Percussion
4	447.00	Bridge No.1	25.0	2	Percussion
Total			71.0	6	-

a) BH No.1 (North Portal, EL=726.20m)

The boring site is located at 30 m eastward from the existing road centreline which is aligned on a gentle monoclinal slope consisting of weathered shale and covered with thin talus deposit. On the slope, there are many big boulders of limestone having a diameter of 2 m to 3m. The geological cross section is shown in Figure 6.4.2 and the drilling results are as follows:

Depth (m)	Thickness (m)	Formation
0.00~1.00	1.00	Top Soil (Clayey Silt with Gravel)
1.00~3.50	2.50	Talus Deposit (Gravel with Silt)
3.50~5.50	2.00	Highly weathered Shale
5.50~15.00	9.50	Weathered Shale

b) BH No.2 (South Portal, EL=672.00m)

The boring site is located at the bottom of the narrow valley, at 70 m to the east of the existing road centreline. The valley faces steep slopes consisting of limestone and is filled with debris flow deposit. The geological cross section is shown in Figure 6.4.3 and the drilling results are as follows:

Depth (m)	Thickness (m)	Formation
0.00~2.00	2.00	Debris Flow Deposit (Gravel)
2.00~6.15	4.15	Weathered Fine Limestone
6.15~6.45	0.30	Fractured Zone (Rubble)
6.15~15.00	8.50	Weathered Fine Limestone

c) BH No.3 (Bridge No.4, EL=597.00m)

The boring site is located in the river bed, at 10m eastward from the edge of the Bridge No.4.

This site is on the surface of a small fan or alluvial cone formed by a by seasonal river. The river channel is obscure and shallow and alluvial gravel with a 10 - 20 cm diameter is found in the channel. The geological cross section is shown in Figure 6.4.4 and the drilling results are as follows:

Depth(m)	Thickness(m)	Formation
0.00~4.50	4.50	Fan Deposit (Gravel)
4.50~11.60	5.15	Highly Weathered Shale
11.60~11.80	0.20	Highly Weathered Sandstone
11.80~16.00	4.20	Weathered Shale

d) BH No.4 (Bridge No.1, EL=447.00m)

The boring site is located on the central part of the channel of the Kohat River which is the biggest seasonal river in the basin. Therefore, there is no running water in the channel except in the monsoon season. The geological cross section is shown in Figure 6.4.5 and the Drilling results are as follows:

Depth (m)	Thickness (m)	Formation
0.00~0.50	0.50	Alluvial Gravel
0.50~2.00	1.50	Alluvial Stiff Clay
2.00~8.50	5.40	Alluvial Gravel with Silt
8.50~9.50	1.00	Diluvial Hard Clay
9.50~1450	5.00	Diluvial Gravel with Clay
14.50~25.00	10.50	Dilivial Gravel

(2) Result of Standard Penetration Test (SPT)

The standard penetration test was carried out for BH No.3 and BH No.4. The results of SPT are as follows:

BH No.	Depth (m)	Formation	N Value (time/30cm)
3	0.00~4.50	Alluvial Gravel (AF)	(30~50)
3	6.15~6.45	Weathered Shale	60
3	8.15~8.45	Weathered Shale	81
3	9.15~9.45	Weathered Shale	R
3	11.00~11.30	Weathered Shale	R

BH No.	Depth (m)	Formation	N Value (time/30cm)
4	0.00~0.50	Alluvial Gravel (AG1)	(20~30)
4	1.15~1.45	Alluvial Stiff Clay (AC)	7
4	2.00~8.50	Alluvial Gravel (AG2)	(30~50)
4	8.50~9.50	Diluvial Hard Clay (DC2)	41
4	9.50~14.50	Diluvial Gravel (DG2)	(30~50)
4	14.50~25.00	Diluvial Gravel (DG3)	(50<)

Note : ( )= Estimated Value, R= Rebound

(3) Rock Quality Designation (RQD)

The RQD value of rock shows that the rate of total drilling core is 10 cm in length in one meter. This value is used as an indicator for evaluation of rock condition. Following is a comparison between the RQD value and rock classification:

Table 6.4.2 Rock Quality Designation (RQD)

RQD (%)	RQD (%)	Rock Classification
0 ~ 25	Very Poor	D ~ CⅡ
25 ~ 50	Poor	CⅡ ~ CⅠ
50 ~ 75	Fair	CⅠ ~ B
75 ~ 90	Good	B ~
90 ~ 100	Excellent	A ~ B

By Deere, 1967

As shown in the table below, the RQD value of BH No.2 ranges from Poor to Fair at various depths, but the RQD value at the depth between 7m and 9m is Very Poor.

Depth(m)	RQD(%)	Depth(m)	RQD(%)
2~3	59	7~8	21
3~4	49	8~9	14
4~5	64	9~10	74
5~6	31	10~11	43
6~7	54	11~12	37

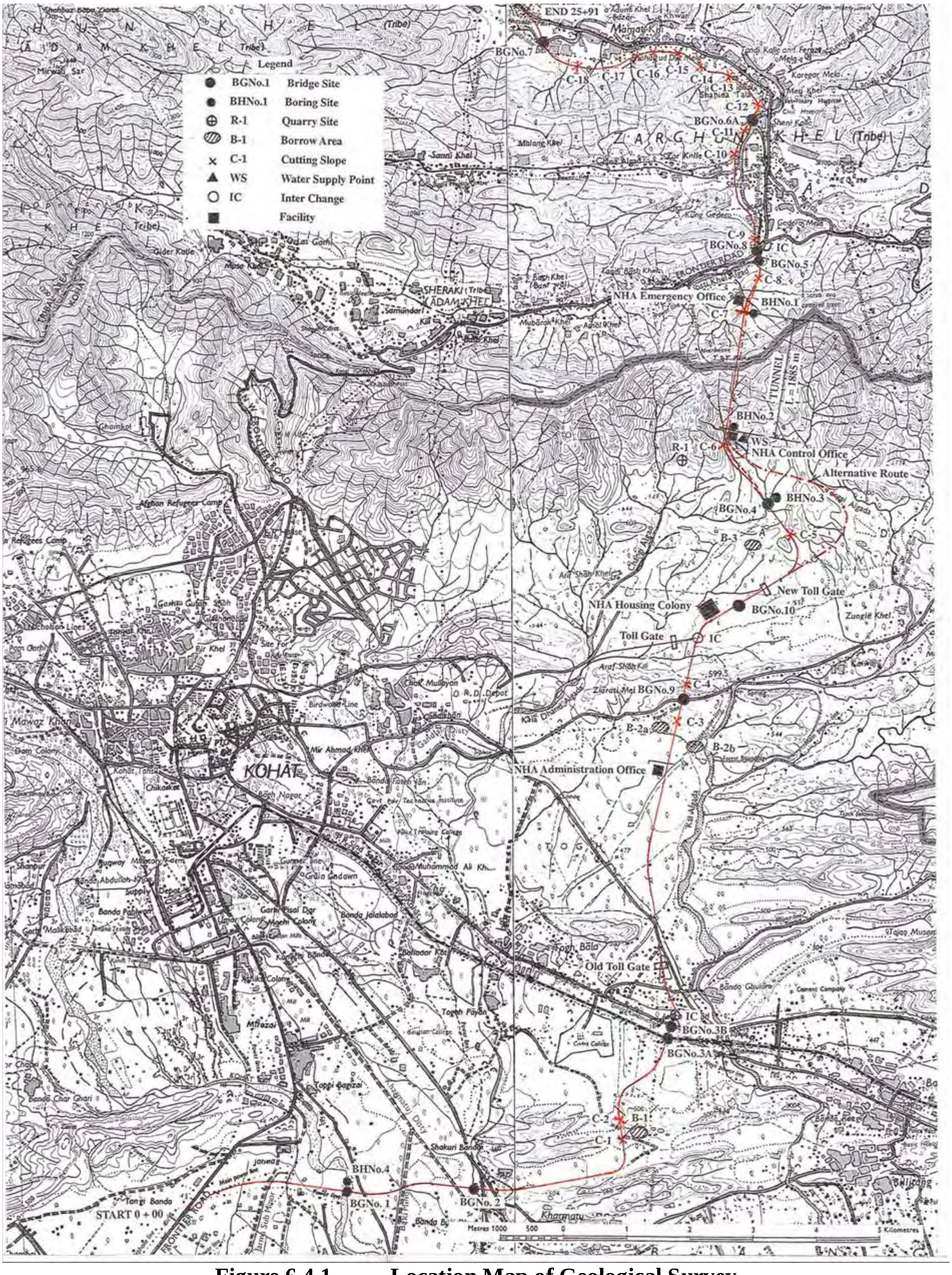


Figure 6.4.1 Location Map of Geological Survey

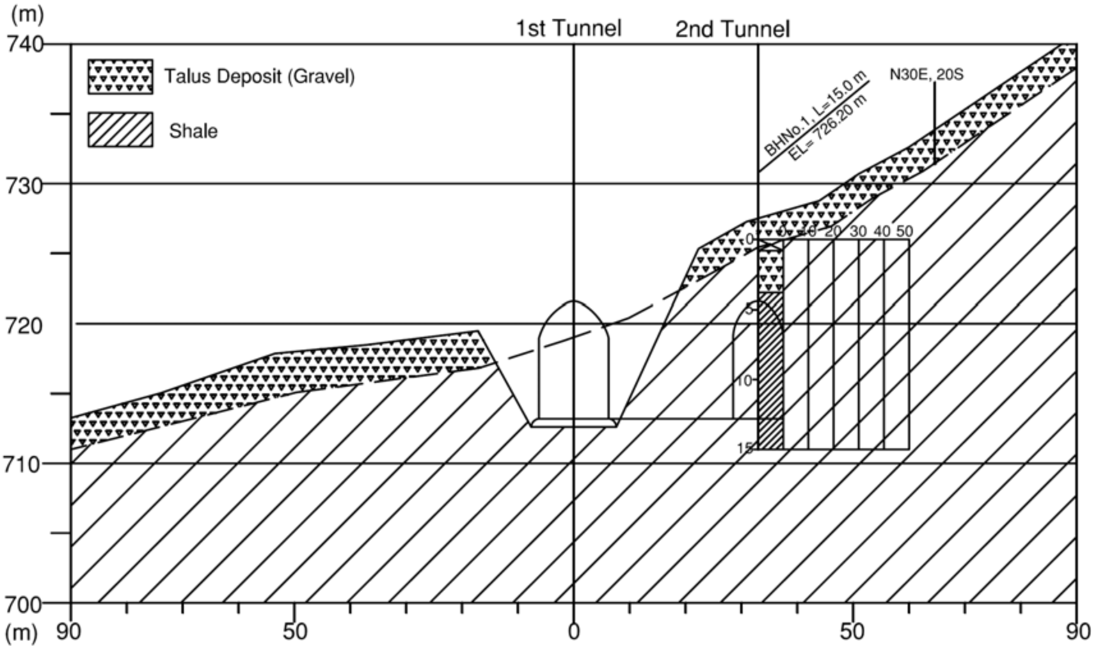


Figure 6.4.2 Geological Cross Section of North Portal

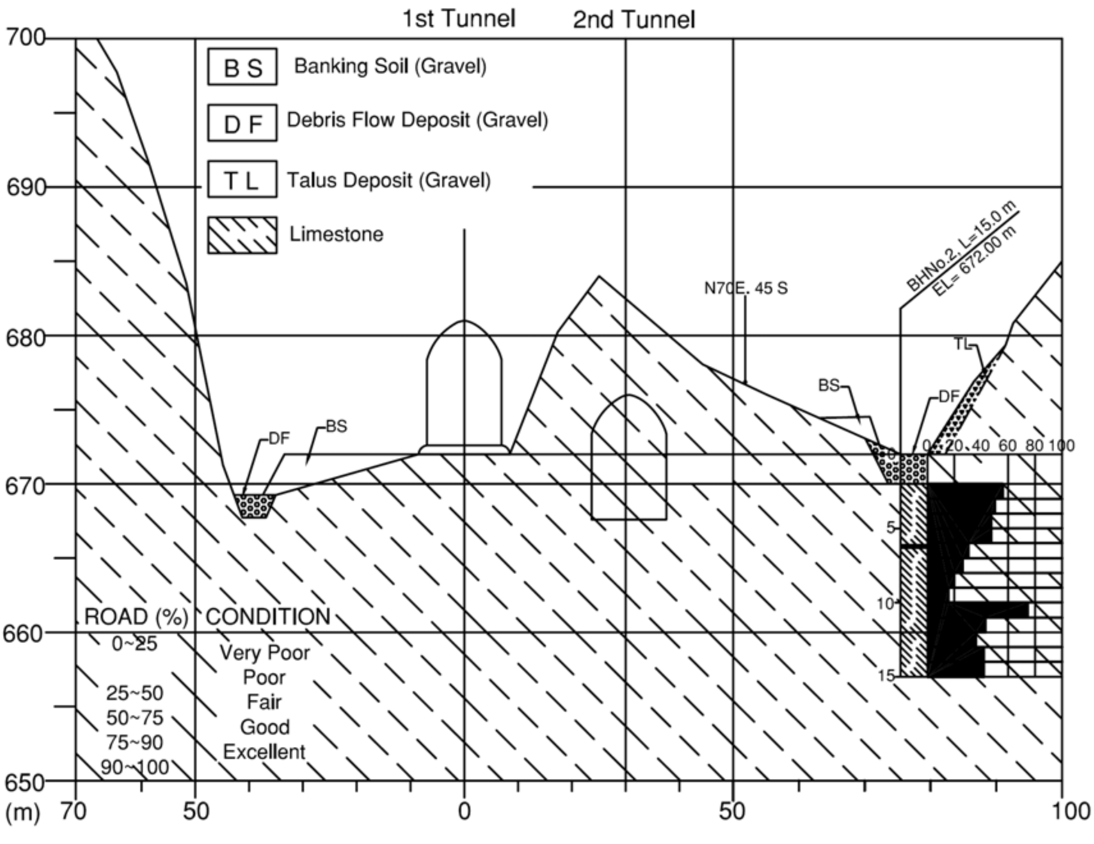


Figure 6.4.3 Geological Cross Section of South Portal

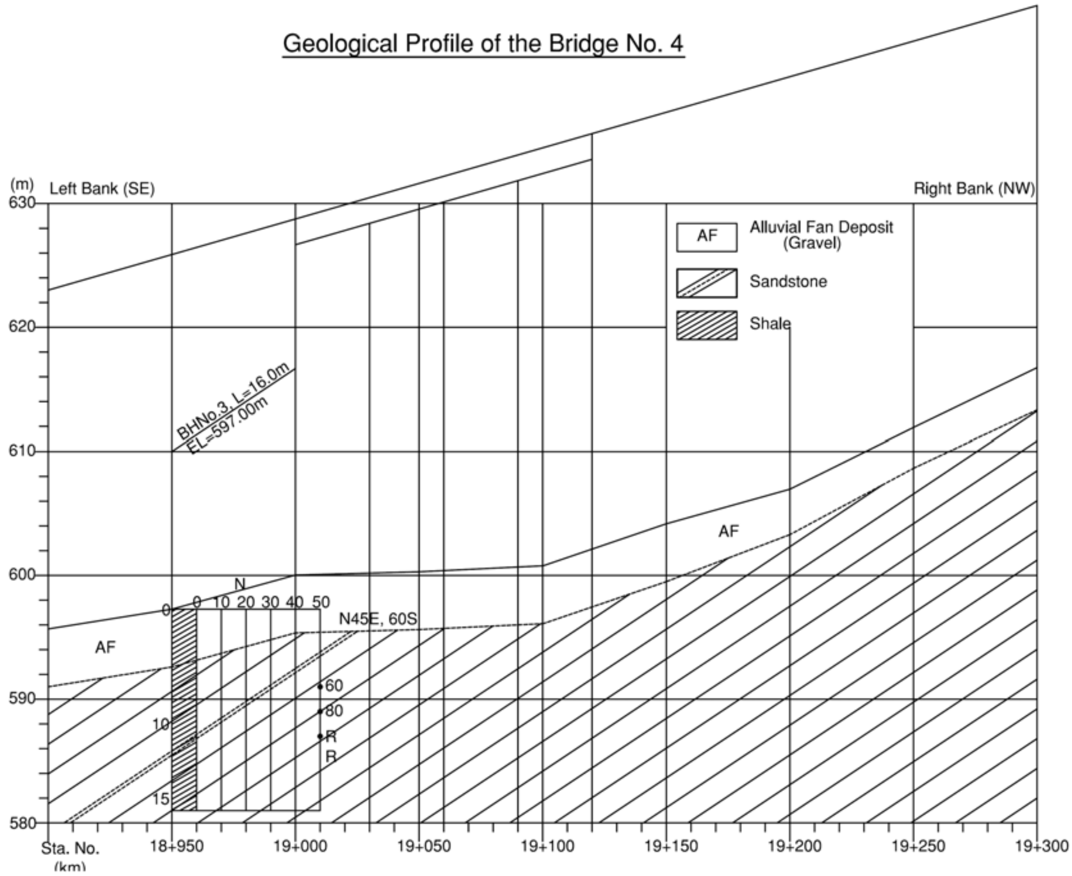


Figure 6.4.4 Geological Profile of Bridge No.4

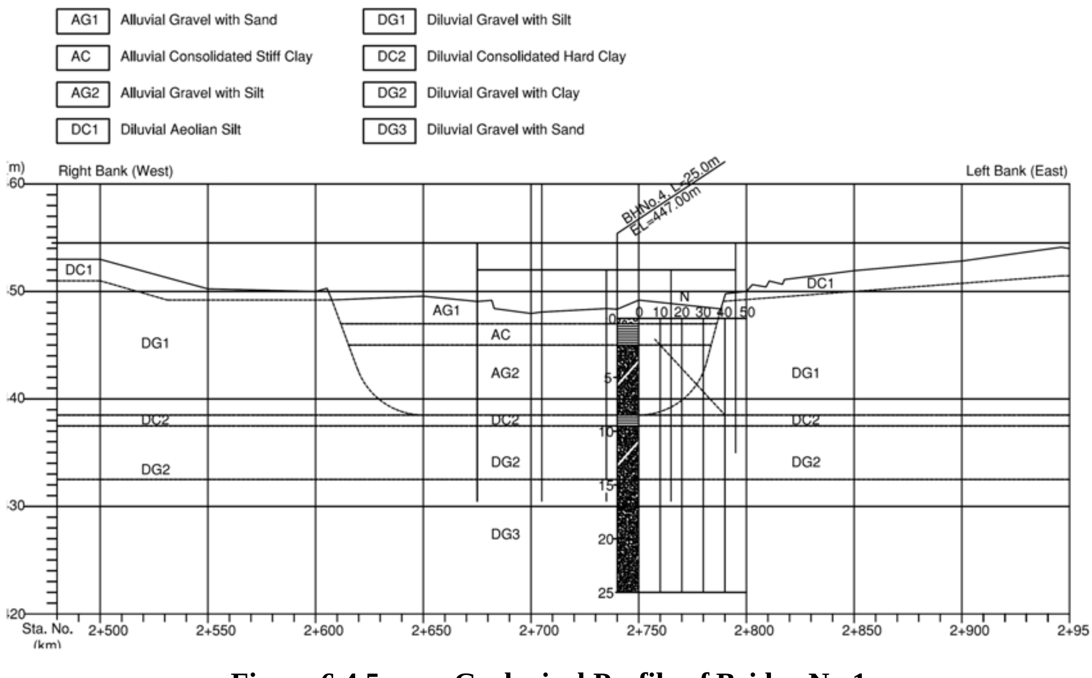


Figure 6.4.5 Geological Profile of Bridge No.1

6.4.2 Laboratory Test

Soil tests were carried out for samples taken from BH No.3 for the Bridge No.4 and from BH No.4 for the Bridge No.1. Rock test was carried out for the limestone taken from BH No.2 for the south portal. The items and quantities of laboratory tests are shown in Table 6.4.3.

Table 6.4.3 Quantities of Laboratory Tests

Soil				Rock			
BH No.	NMC	GS	LL	PL	Grain size	Bulk Density	Compression
No. 1	-	-	-	-	-	-	-
No. 2	-	-	-	-	-	2	2
No. 3	3	-	3	3	3	-	-
No. 4	2	2	9	9	11	-	-
Total	5	2	12	12	14	2	2

Note: NMC = Natural Moisture Content
GS = Specific Gravity
LL = Liquid Limit
PL = Plastic Limit

(1) Results of Soil Tests

As shown in Table 6.4.4, the weathered shale at BH No.3 with a N value between 60 and R is composed of consolidated hard clay. The mean value of liquid limit of all the three samples is LL=30%, and their mean value of plasticity index is SI=10. At BH No.4, a 1.5 m thick stiff alluvial clay (AC) layer is found at the depth from 0.50 m to 2.00 m, and a 1.0 m thick hard diluvial clay (DC2) layer is found at the depth from 8.50 m to 9.50 m. The values of liquid limits and plasticity index of stiff alluvial clay (AC) and hard diluvial clay (DC2) are as follows.

AC : LL=28.5%, PI=18.0

DC2 : LL=28.5%, PI=10.5

The materials have big strength in the dry condition and low permeability. On the other hand, some materials have low compressibility.

Table 6.4.4 Results of Laboratory Test for Soil

BH No.	Sample No.	Depth (m)	Soil Type	N Value (Times)	NMC (%)	GS	Atterberg Limit LL (%)	PL (%)	PI (%)	Grain Size Analysis Gravel (%)	Sand (%)	Silt (%)	Clay (%)
1	-	-	Rock	-	-	-	-	-	-	-	-	-	-
2	-	-	Rock	-	-	-	-	-	-	-	-	-	-
3	SPT1	6.15~6.45	Shale	60	8.2	-	32.6	21.1	11.5	-	8.4	40.4	51.2
3	SPT2	8.15~8.45	Shale	81	6.7	-	27.8	18.5	9.3	-	9.1	42.5	48.4
3	SPT3	9.15~9.45	Shale	R	6.6	-	30.0	19.9	10.1	-	10.3	43.0	46.7
4	SPT 1	1.15~1.45	Clay	7	18.5	-	28.5	10.5	18.0	-	8.6	35.0	55.0
4	DS 1	3.00~3.30	Gravel	-	-	-	28.2	17.0	11.2	56.4	12.1	10.0	21.5
4	DS 2	4.50~4.80	Gravel	-	-	-	23.2	14.0	9.2	56.7	14.9	14.0	14.4
4	DS 3	6.00~6.30	Gravel	-	-	-	23.2	13.1	10.1	52.6	16.2	15.0	16.2
4	DS 4	7.50~7.80	Gravel	-	-	-	21.5	12.9	8.6	59.9	13.9	16.0	10.2
4	DS 5	8.00~8.20	Gravel	-	-	2.673	21.7	14.0	7.7	66.0	20.4	9.0	4.6
4	SPT 2	8.30~8.60	Clay	41	19.2	-	28.5	18.0	10.5	-	-	36.0	64.0
4	DS 6	10.00~10.30	Gravel	-	-	-	25.6	16.0	9.6	50.7	17.7	14.2	17.4
4	DS 7	14.00~14.30	Gravel	-	-	2.676	21.9	14.7	7.2	70.0	18.0	5.8	6.2
4	DS 8	18.00~18.30	Gravel	-	-	-	-	-	-	99.7	0.3	-	-
4	DS 9	20.00~22.30	Gravel	-	-	-	-	-	-	99.8	0.2	-	-

Note: NMC = Natural Moisture Content , GS = Specific Gravity, LL = Liquid Limit , PL = Plastic Limit

Table 6.4.5 Results of Laboratory Test for Rock

BH No.	Sample No.	Depth (m)	Rock Type	Bulk Density (g/cm³)	Compressive Strength (Mpa)
2	C-1	2.69~2.86	Lime stone	2.691	53.03(540.75 kg f/ cm²)
2	C-2	7.59~7.70	Lime stone	2.795	36.89(376.17 kgf/cm²)

(2) Results of Rock Test

A compressive strength test was carried out for the limestone taken from BH No.2. The results of rock test shown in Table 6.4.5 indicate that the compressive strength of the limestone sample is 376 ~ 541 kgf/cm2. As reference, the general values of strength of rock are shown in Table 6.4.6.

Table 6.4.6 General Strength of Rock

Class	Rock Type	General Value
A	Fresh Hard Rock (Igneous Rock, Metamorphic Rock and Sedimentary Rock of Pre-Tertiary)	qu : > 800
		Vp : > 5000
B	Weathered Rock and Hard Tertiary Rock	qu : > 100~800
		Vp : > 3000~5000
C	Very Weathered Rock and Soft Tertiary Rock	qu : < 100
		Vp : ≒ 3000
D	Soil and Gravel	—

qu=Compressive Strength (kgf/cm^2)
Vp= Elastic Wave Velocity (m/s)
Rock Classification for Tunnel by Ministry of Construction of Japan

6.5 Analysis of Cutting Slope and Settlement of Banking

6.5.1 Analysis of Cutting Slope

Investigation of cutting slopes was carried out for gradients, rock type, stability and existing protection. The cutting slope sites of the existing road are shown in Figure 6.4.1 Location Map. The results of investigation for the existing road and Project road are shown in Tables 6.5.1 and 6.5.2, indicating the gradient and protection works of cutting slopes by rock type as follows:

Formation	Gradient	Protection
Rock (All types of rock)	1 : 0.5	Nothing, Partial Rock Net
Fractured and Weathered Zone	1 : 0.5~0.7	Rock Net
Unconsolidated Deposit	1 : 1.0~1.2	Grouted Riprap

According to the above results, the gradient of cutting slope for all kinds of hard rock is 1:0.5 and there are no slope protections for hard rock, except for partial rock nets installed for cracky rock. The slope gradient for weathered rock ranges between 1:0.5 and 1:0.7, but it seems that protections for fractured zones, weathered zones and cracky rock are not in so good condition. The slope gradient for unconsolidated deposits like talus deposit, terrace deposit or residual soil is 1:1.0 to 1:1.2, and protection works for these zones consists mostly of grouted riprap. At present, most cutting slopes appear to be stable because they have been constructed only 3 years before. Rock falls occurred at many places where no protections for rock are provided, but there is no scattering of rocks on the road owing to enough clearance between slope and road. The new route is planned to be constructed in parallel to the existing route, therefore there is no problem to adopt the same gradient and same protection for cutting slopes of the new route. However, rock nets should be installed to cover all the cracky rocks, fractured zones and weathered zones in order to prevent rock falls.

6.5.2 Settlement of Embankment

A 30 m high embankment construction is planned between Sta. 18+800 and Sta. 21+000. This section is located on a composite fan beside the mountain foot. According to the test results of BH No.3, this site consists of alluvial fan gravel with a thickness of 4.5 m and weathered shale. The gravel is very dense and its N value is estimated to be 30 to 50. The N value of weathered shale is between 60 and rebound. Considering this formation, it can be said that the possibility of settlement is very low at this site.

6.6 Materials Survey

Materials survey for borrow materials and aggregate was carried out with field reconnaissance and information from the contractor who constructed the 1st Kohat Tunnel Access Roads. The material survey as described below and no laboratory test were conducted for the materials surveyed.

(1) Borrow Materials for Embracement

Borrow pits for banking soil are shown in Figure 6.4.1 Location Map and among which the following borrow pits were selected:

B1: Sta.No.7+100 Alluvial Terrace Deposit

B2: Sta.No.14+000 Alluvial Terrace Deposit

B3: Sta.No.18+500 Talus Deposit

Alluvial deposit consists of gravel of limestone and sandstone with a 2 - 5 cm diameter and yellowish brown fine sand and silt. Talus deposit consists of rubble of limestone and sandstone with a 5 - 10 cm diameter and reddish brown fine sand and silt. These formations are widely distributed along the southern part of the projected route. So it is considered that their available quantity is enough for the construction works. From the geo-technical point of view, the CBR value of these deposits could be expected to be more than 20.

(2) Coarse Aggregate

The proposed quarry for coarse aggregate is shown in Figure 6.4.1 Location Map. This site is located at the mountain foot at 1 km westward from Sta. 19+100. This formation consists of highly weathered sandy limestone which is easily broken by strikes of hammer. Therefore, laboratory test is required to confirm its strength.

(3) Fine Aggregate

River sand as fine aggregate is not distributed around the existing road. River sand is found on the east bank of the Indus River which is located at 80 km east of Peshawar City along the Highway No.5. This site is a junction of the Indus River and the Kabul River. The Indus River forms several sand banks upstream of the junction by its several meanders. These sand banks consist of fine sand which can be used as fine aggregate for the construction works.

Table 6.5.1 Cutting Slope of Existing Road

No.	Sta.No. (km)	Length (m)	Mean Height (m)	Cutting Site	Rock Type	Gradient	Existing Protection	Stability
C-1	7+340~7+480	140	13	Both Sides	Grey, Weathered, Sandstone	1 : 0.5	Rock net	Medium
C-2	7+710~7+800	90	3	E.Side	Brown, Talus Deposit (Gravel)	1 : 1.0	Nothing	Bad
C-3	14+415~14+625	210	7	Both Sides	Reddish Brown, Conglomerate	1 : 1.2	Groutred Riprap	Good
C-4	15+110~15+420	310	26	Both Sides	Fractured sandstone and shale	1 : 0.7	Rock net	Medium
C-5	17+780~18+110	330	31	Both Sides	Red Sandstone(lower) , Conglomerate(upper)	1 : 1	Groutred Riprap	Good
C-6	19+740~19+960	220	26	W.Side	Gray, Sandy Limestone	1 : 0.5	Nothing	Medium
C-7	18+130~18+330	200	8	Both Sides	Brown, Talus Deposit (Gravel)	1 : 1	Groutred Riprap	Good
C-8	18+600~18+800	200	1	E.Side	Brown, Talus Deposit (Gravel)	1 : 1	Groutred Riprap	Good
C-9	19+180~19+220	40	4	W.Side	Gray, Fine Lime stone	1 : 0.5	Nothing	Medium
C-10	20+620~20+700	80	5	W.Side	Gray , Fine Limestone	1 : 0.5	Nothing	Medium
C-11	20+910~21+150	240	28	W.Side	Talus Deposit (upper), Fine Limes stone(lower)	1 : 0.5	Fence for Talus	Bad
C-12	21+360~21+690	330	30	Both Sides	Fractured Limestone with Fault	1 : 0.5	Nothing	Bad
C-13	32+280~22+400	120	10	Both Sides	Grey, Fine Lime stone	1 : 0.5	Nothing	Medium
C-14	23+030~23+080	50	3	Both Sides	Grey ,Fine Lime stone	1 : 0.5	Nothing	Medium
C-15	23+460~23+480	20	1	S.Side	Grey,Fine Lime stone	1 : 0.5	Nothing	Medium
C-16	23+550~23+630	80	16	S.Side	Grey , Fine Lime stone	1 : 0.5	Nothing	Medium
C-17	23+700~23+940	240	25	Both Sides	Fractured Fine Lime stone with Fault	1 : 0.5	Rock net (upper)	Bad
C-18	24+120~24+460	340	21	Both Sides	Fine and Sandy Lime stone	1 : 0.5	Rock net (upper)	Medium

Table 6.5.2 Cutting Slope of the Projected Road

No.	Sta. No. (km)	Length (m)	Height (m)	Cutting Site	Rock Type	Gradient	Proposed Protection
C-1	7+340~7+480	140	13	E. Side	Grey, weathered sandstone	1 : 0.5	Rock net with rock anchor (all over)
C-2	7+710~7+800	90	3	E. Side	Brown, talus deposit (gravel)	1 : 1.0	Grouted riprap (all over)
C-3	14+415~14+625	210	7	E. Side	Reddish brown conglomerate	1 : 1.2	Grouted riprap (all over)
C-4	15+110~15+420	310	26	E. Side	Fractured sandstone and shale	1 : 0.7	Rock net with rock anchor (all over)
C-5	17+780~18+110	330	31	E. Side	Red sandstone and conglomerate	1 : 1.0	Grouted riprap (lower), rock net (upper)
C-7	18+130~18+330	200	26	E. Side	Brown talus deposit (gravel)	1 : 1.0	Grouted riprap (all over)
C-8	18+600~18+800	200	3	E. Side	Brown talus deposit (gravel)	1 : 1.0	Grouted riprap (all over)
C-12	21+360~21+690	330	30	E. Side	Fractured limestone with fault	1 : 0.5	Rock net with rock anchor (all over)
C-13	22+280~22+400	120	6	E. Side	Grey, sandy limestone	1 : 0.5	Rock net with rock anchor (all over)
C-17	23+700~23+940	240	25	N. Side	Fractured limestone with fault	1 : 0.5	Rock net with rock anchor (all over)

6.7 Soil Characteristics along the Road Alignment

Table 6.7.1 shows a summary of the laboratory tests of soil along the Project route conducted by the 1st Kohat Tunnel and Access Roads Project during the design stage. The Study Team conducted site reconnaissance to confirm site conditions and soil type based on soil classification of AASHTO. The soil found at the site is not much diffidence from those in Table 6.7.1. The original ground has sufficient strength (the lowest CBR 5-6%) for embankment construction. No specific weak soil exists.

Table 6.7.1 Laboratory Test Results of Soil along the Project Road

Sample No	Location Station Km	Max Size	Analysis #10	(Pass %) # 40	# 200	Liquid Limit	P.I	Soil group with Group Index	Max. Dry Density gms/c.c	OMC %	CBR at 95 % Comp	Swell %
1	1+000	#4		100	85	43	21	A-7-6 (13)	1.93	11.0	6.4	0.5
2	3+000	#5		100	85	31	10	A-4 (8)	1.95	10.0	6.7	1.0
3	4+500	#6		100	88	27	8	A-4 (8)	1.92	11.0	5.4	0.5
4	6+000	#7		100	98	41	19	A-7-6 (123)	1.93	11.0	6.5	0.9
5	8+000	#8		100	95	29	9	A-4 (8)	1.92	11.0	4.9	0.4
6	10+000	6	20	13	10	N-P		A-1-a (0)	2.17	6.5	60.0	0.1
7	13+000	#4	100	98	80	29	9	A-4 (8)	1.97	10.0	8.0	0.3
8	14+000	3	37	31	28	28	8	A-2-4 (0)	2.16	6.2	30.0	0.2
9	19+500	6	22	20	20	N-P		A-1-b (0)	2.13	7.4	56.0	0.1
10	22+000	#4		100	90	32	10	A-4 (8)	1.89	11.0	6.9	0.4
11	24+000	6	32	22	15	N-P		A-1-a (0)	2.16	6.4	36.0	0.1
12	26+000	#4	100	98	75	29	10	A-4 (8)	1.96	10.0	5.9	0.1

Source: Design Review Report of the 1st Kohat Tunnel and Access Road Project

Chapter 7. Traffic Analysis

7.1 Present Traffic Condition

7.1.1 Available Data and Traffic Survey

(1) Toll Collection Data

NHA keeps data on tickets sold at two toll plazas near the tunnel, from which the traffic volume in the Kohat Tunnel can be calculated. Yearly and monthly data were provided by NHA to the JICA Study Team. In the ticket sales data, vehicles are classified into four types: 1) Car/Jeep, 2) Hi-ace/Coach, 3) Bus, 2&3-Axle Trucks, and 4) Articulated Trucks. Buses and 2&3-Axle Trucks belong to the same category because the same toll rate is applied for those vehicles.

(2) PTPS Traffic Survey

In the first phase of PTPS, traffic surveys were carried out on a nationwide scale, including several sites near the Kohat Tunnel.

(3) Traffic Survey, Phase II

In the second phase of the PTPS, supplemental traffic count surveys were carried out at four intersections along the access road of the Kohat Tunnel, as shown in Figure 7.1.1. The survey was designed as follows:

(4) Survey location and survey date

Survey Location

- IC-1: Kohat Toi Intersection (Start point of the Kohat Tunnel Access Road)
- IC-2: Kohat Pindi Interchange (with N-80: Kohat - Rawalpindi Road)
- IC-3: Kohat Link Road Interchange
- IC-4: Dara Adam Khel Intersection (End point of the Kohat Tunnel Access Road)

Survey Date and Time

- IC-1: 29-May (06:00 – 22:00, 16 hours)
- IC-2: 29-May (06:00 – 22:00, 16 hours)
- IC-3: 30-May -31 May (06:00 – 06:00, 24 hours)
- IC-4: 30 May (06:00 – 22:00, 16 hours)

(5) Vehicle Classification

Vehicles were classified into six types as follows, according to the vehicle types categorized for toll collection at the toll plazas of NHA.

1 Car, Jeep, Land Cruiser/Pajero, Suzuki Van/ Pickup
2 Wagon (up to 24 seats), Hilux (Single/Double Cabin), Milk Truck M-3000, Coaster & Mini Bus (up to 24 seats), Mini Truck/Tanker built on Mazda T-3500 chassis and equivalent
3 Buses greater than 25 seats
4 Rigid Trucks (2-Axle)
5 Rigid Trucks (3-Axle)
6 Articulated Trucks/Vehicles

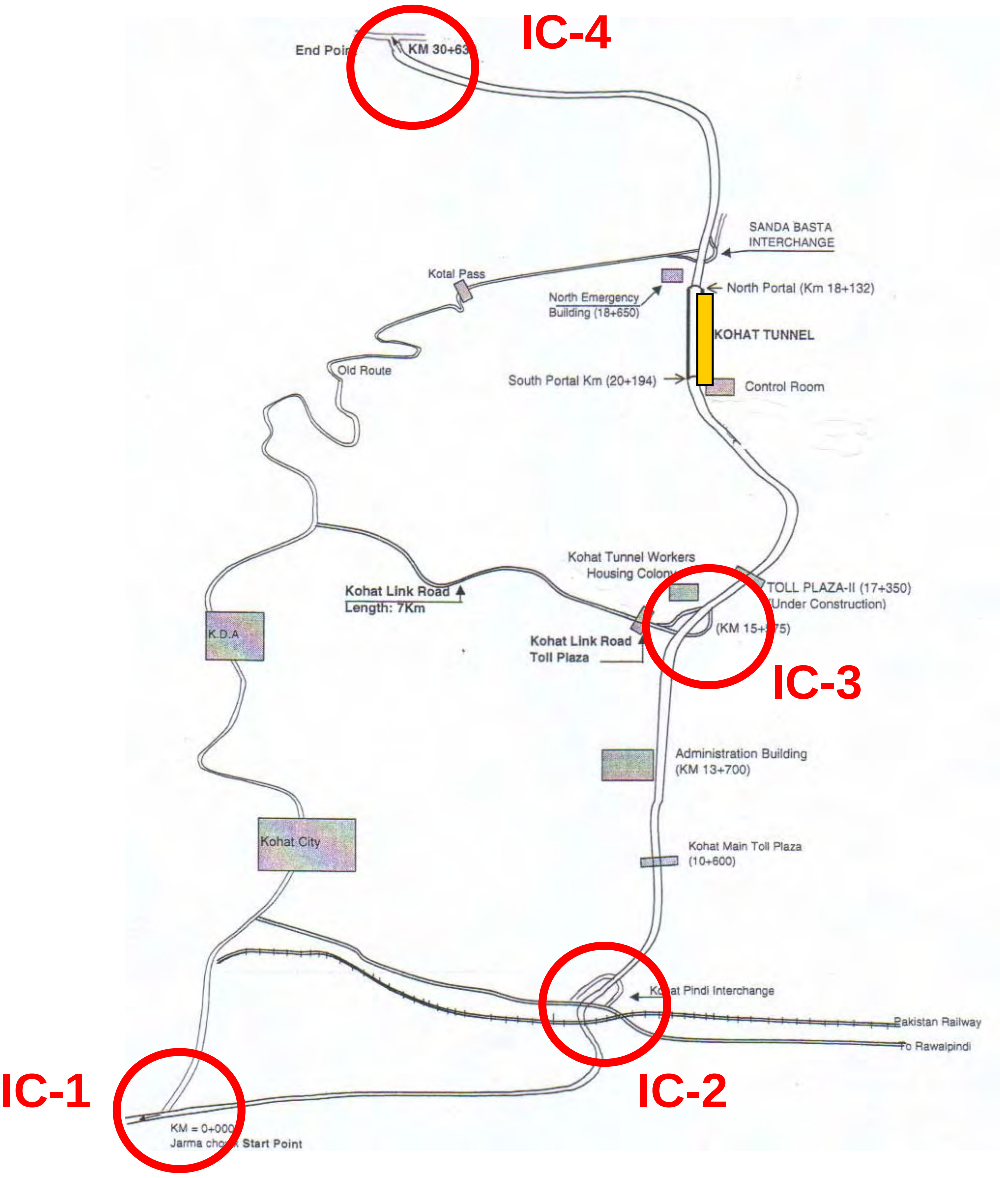
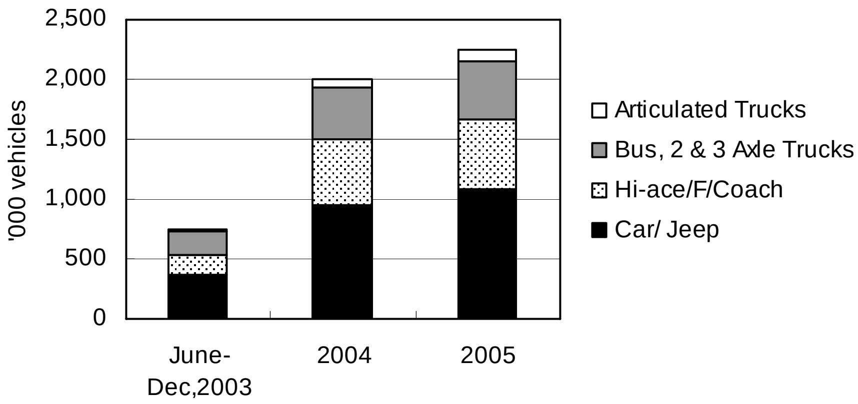


Figure 7.1.1 PTPS Traffic Survey Sites Near the Kohat Tunnel

7.1.2 Traffic Volume

(1) Yearly Traffic

Figure 7.1.2 illustrates the yearly traffic volume in Kohat Tunnel, which was 2.0 million vehicles in 2004 and 2.2 million in 2005, increased by 12.4%. These figures represent an annual average daily traffic of 5,487 vehicles/day in 2004 and 6,159 in 2005.

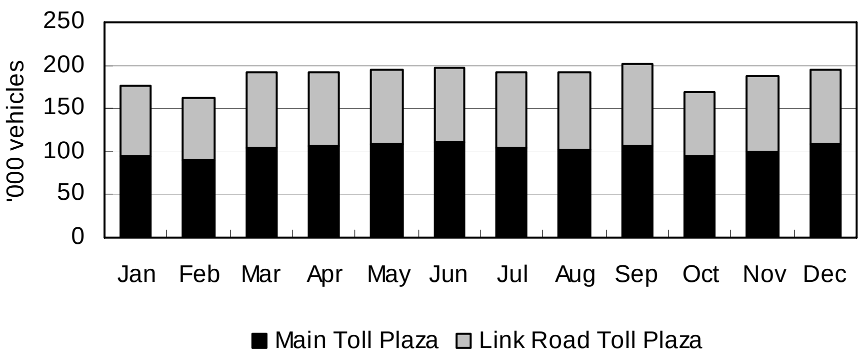


Source: NHA

Figure 7.1.2 Yearly Traffic Volume at Kohat Tunnel

(2) Monthly Traffic

Seasonal and monthly variations of the traffic volume are not significant as shown in Figure 7.1.3. The monthly traffic volume was highest in September at 1.09 times AADT, while it was lowest in October at 0.89 times AADT.

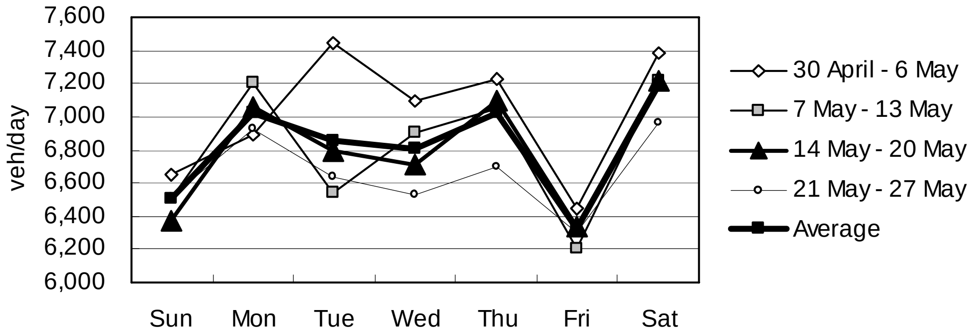


Source: NHA

Figure 7.1.3 Monthly Traffic Volume of Kohat Tunnel in 2005

(3) Daily Traffic

The daily traffic volume in the Kohat Tunnel varies from 6,300 to 7,200 vehicles per day (veh/day) according to the days of the week as shown in Figure 7.1.4. It is observed that the weekend traffic volume, on Friday and Sunday, is lower than that on weekdays. Peak traffic is observed on Saturday.

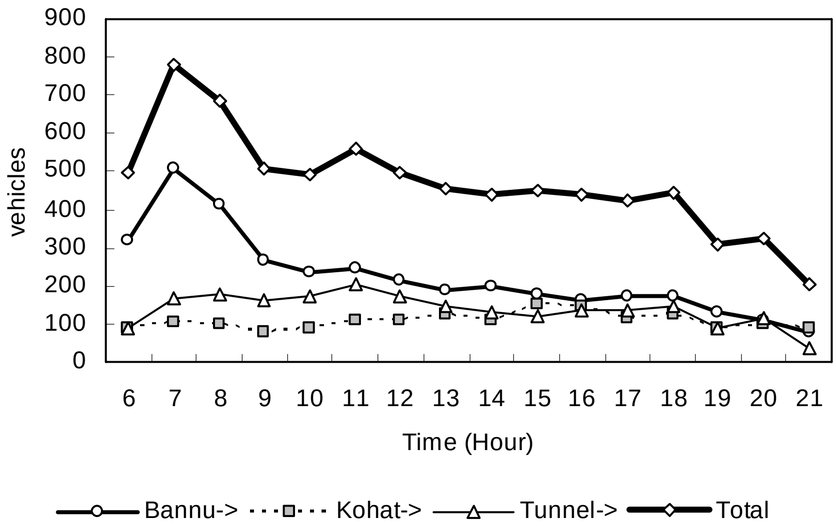


Source: NHA

Figure 7.1.4 Daily Traffic Volume of Kohat Tunnel in May 2006

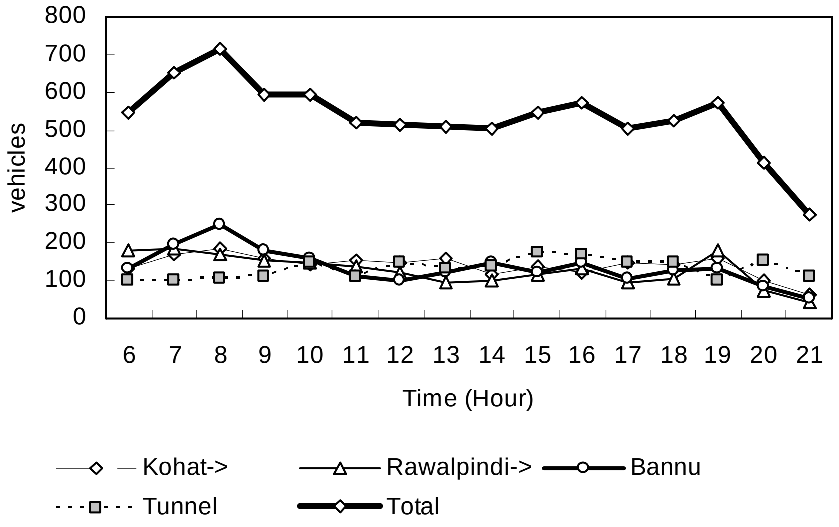
(4) Hourly Traffic

The hourly traffic volumes at the intersections on the Access Road are illustrated in Figures 7.1.5 to 7.1.8. The morning peak is observed at 8:00 - 9:00h and the afternoon peak at 16:00 - 17:00h. The ratio of the morning peak traffic to the daily traffic was 7.1%, while that of the afternoon peak traffic was 6.5%, as recorded on 29 May 2006. The morning peak is contributed mainly by the traffic from Kohat to Peshawar, while the afternoon peak is contributed mainly by the traffic from Peshawar to Kohat.



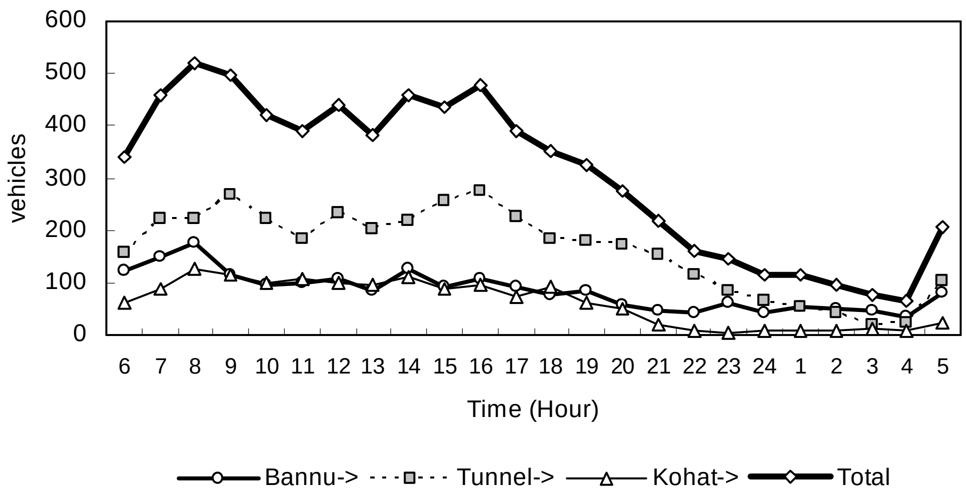
Source: PTPS Traffic Survey, Phase II (29-May-2006)

Figure 7.1.5 Hourly Traffic Volume at the Kohat University Intersection (IC-1)



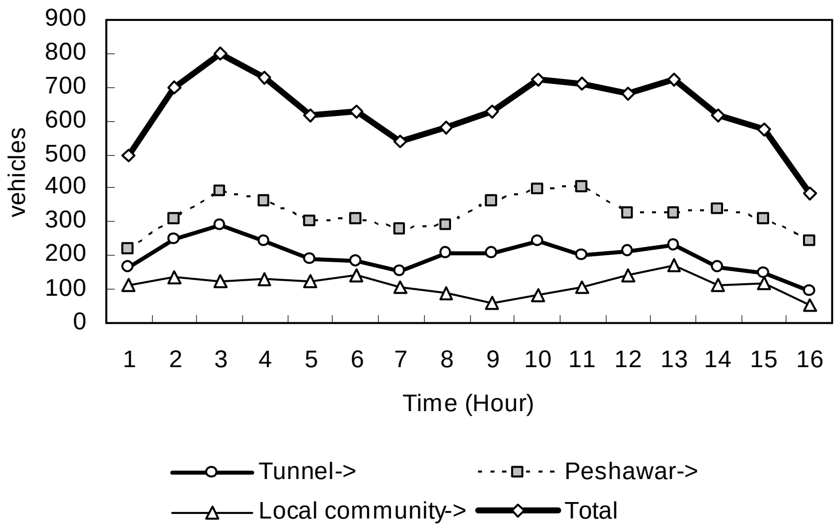
Source:
Traffic Survey, Phase II
(29-May-May-2006)

Figure 7.1.6 Hourly Traffic Volume at the Karim Abad Intersection (IC-2)



Source:
Traffic Survey, Phase II
(30-May, 31-May-2006)

Figure 7.1.7 Hourly Traffic Volume at IC-3

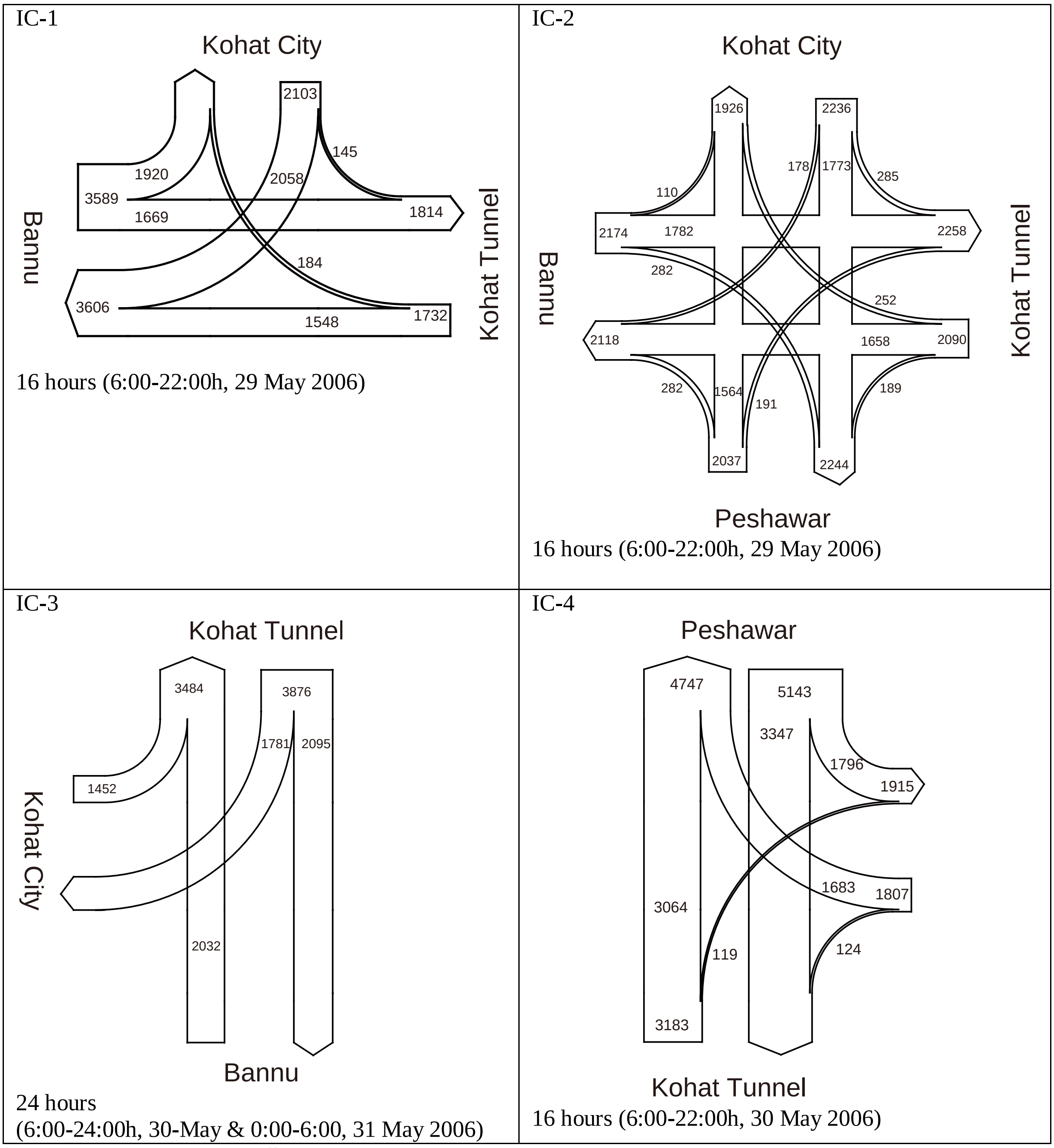


Source:
Traffic Survey, Phase II
(30-May-2006)

Figure 7.1.8 Hourly Traffic Volume at IC-4

(5) Traffic Volumes at Intersections

Figure 7.1.8 illustrates the traffic volumes at the four intersections where traffic count surveys were conducted. At the intersection IC-1, which is the start-point of the Kohat Tunnel Access Road, the major traffic directions are Bannu - Kohat City and Bannu - Kohat Tunnel. The traffic movement between Kohat City and the Kohat Tunnel is very small: it was only 329 vehicles from 6:00 to 22:00h on 29 May 2006. At the intersection IC-2 of the Kohat Tunnel Access Road and Kohat - Rawalpindi Road, the major traffic movements are through traffic and turning movements are relatively small. At the intersection IC-3 of the Kohat Tunnel Access Road and the Link Road, 44% of the traffic from/to the Kohat Tunnel uses the Link Road. At the intersection IC-4, which is the end-point of the Kohat Tunnel Access Road, four fifth of the traffic is between Peshawar and the Kohat Tunnel.



Source: PTPS Traffic Survey, Phase-II

Figure 7.1.9 Traffic Flow at the Selected Intersections

7.2 Traffic Demand Forecast

7.2.1 Forecast in the PTPS Master Plan

(1) Socioeconomic Scenario

In the Medium Term Development Framework (MTDF), the Government set the target GDP growth rates over the MTDF period at 7.0% in 2005/2006, gradually increasing to 8.2% in 2009/2010. Instead of the assumed growth rate of 7.0% in MTDF, the actual annual economic growth rate over 2005/06 was 6.6% due to rising energy prices and the earthquake of October 8, 2005. Nevertheless, as the investment and financial markets are expanding, it is possible that the target of MTDF may be attained. ADB prospects the mid-term outlook of Pakistan economy in "Asian Development Outlook 2006" as: "*The outlook for the economy is positive and the Government's growth target of 6–8% looks achievable.*" The outlook also analyzes that the economic growth is likely to be above 7% in 2006/07. However, such a high growth is hardly sustained for a long period. The JICA Study Team analyzed three scenarios for the macro economic framework in the Phase-I of the PTPS:

- **High Growth Scenario:** To maintain the average annual growth rate of 7.6% (MTDF target) for 20 years after MTDF.
- **Medium Growth Scenario:** The growth rate of 7% (the average of the last three years) will continue until 2010/11, and then started to decline by 0.5% every five years.
- **Low Growth Scenario:** The growth rate of 7.0% will decline by 0.5% every year until 2010/11, and will continue 4% up to the year 2025.

Table 7.2.1 shows annual growth rates by scenario while Table 7.2.2 shows the projection of GDP and GDP per capita by scenario. Since the high and low scenarios represent extreme sides of economic condition, the differences among scenarios are very large. This study applies the medium growth scenario for demand forecast and considers the high and low scenarios for sensitivity analysis

Table 7.2.1 Economic Growth Scenario

Year	Annual Growth Rate (%)		
	High	Medium	Low
2005/06	7.0	7.0	7.0
2006/07	7.3	7.0	6.5
2007/08	7.6	7.0	6.0
2008/09	7.9	7.0	5.5
2009/10	8.2	7.0	5.0
2010/11 – 2014/15	7.6	6.5	4.0
2015/16 – 2019/20	7.6	6.0	4.0
2020/21 – 2024/25	7.6	5.5	4.0
2024/25 – 2029/30	7.6	5.0	4.0

Source: PTPS Phase-I (Projection by the JICA Study Team)

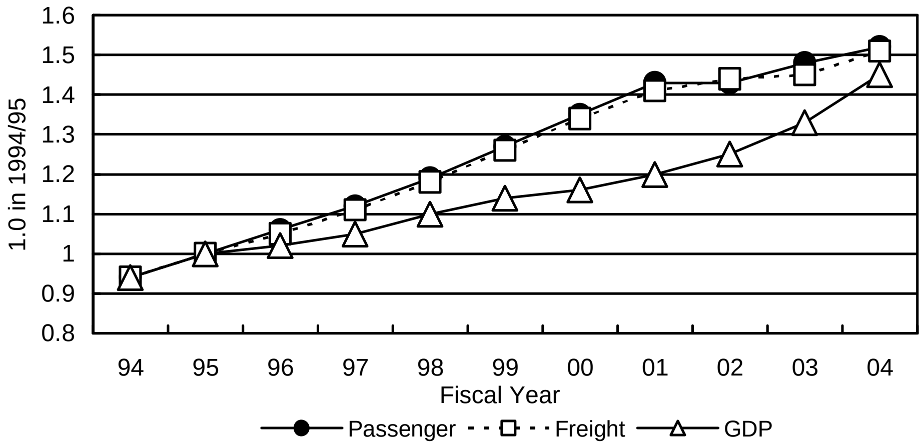
Table 7.2.2 Projection of GDP and GDP per Capita by Scenario

Year	Population (million)	High Case		Medium Case		Low Case	
		GDP (Rs. Billion)	GDP per Capita (Rs.)	GDP (Rs. Billion)	GDP per Capita (Rs.)	GDP (Rs. Billion)	GDP per Capita (Rs.)
2005/06	155.4	6,559	42,213	6,559	42,213	6,559	42,213
2010/11	169.2	9,513	56,214	9,199	54,361	8,531	50,408
2015/16	181.8	13,721	75,460	12,604	69,319	10,379	57,079
2020/21	193.4	19,790	102,304	16,867	87,195	12,627	65,277
2025/26	204.8	28,543	139,386	22,045	107,652	15,363	75,023
2030/31	216.2	41,168	190,381	28,135	130,110	18,691	86,438

Source: PTPS Phase-I (Projection by the JICA Study Team)

(2) Projection of Land Transport Demand

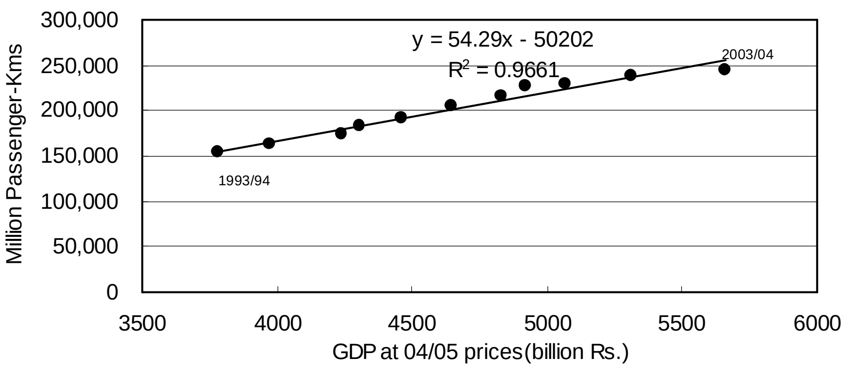
Passenger kilometres (passenger-km) and tonne kilometres (ton-km) have been estimated as important indicators of transport volumes for transport demand analysis and strategic targets for the transport sector in Pakistan. The CAGR[1] of passenger-km in the last 10 years (4.8%) was higher than that of ton-km (4.4%). The growth rates were higher than GDP growth rates prior to 2000/01, while they were lower over the recent four years as shown in Figure 7.2.1. The CAGRs of passenger-km and ton-km were 5.8% and 5.2%, respectively, from 1994/95 to 2000/01, while they were 3.4% and 3.2%, respectively, from 2000/01 to 2003/04.



Source: PTPS Phase-I (Elaborated by the JICA Study Team based on Pakistan Economic Survey, 2004-05)
Note: Fiscal Year 95: 1994-95 (ended June 30, 1995)

Figure 7.2.1 Increase in Land Transport and GDP (1.0 in 1994/95)

The linear regression model was used to estimate passenger-km. Figure 7.2.2 illustrates the result of the analysis. Since the formula computes passenger- km in 2003/04 higher than the actual volume, the linear with the same slope of the formula that intersects the point of 2003/04 was adopted as the projection model.

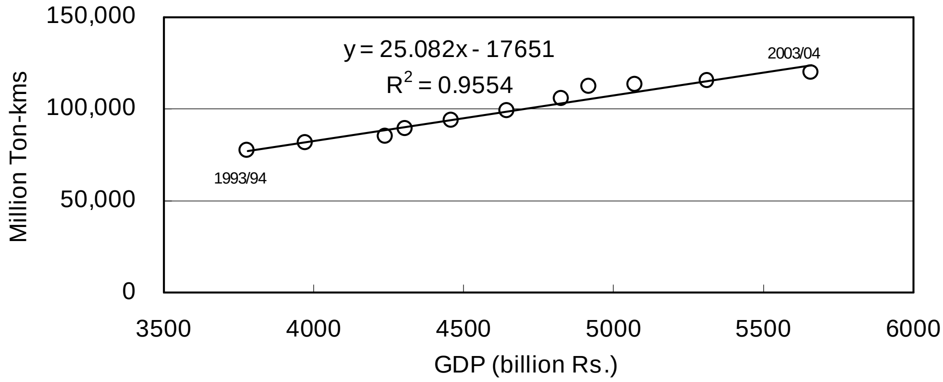


Source: PTPS Phase-I (Elaborated by the JICA Study Team based on Pakistan Economic Survey, 2004-05)

Figure 7.2.2 Regression Analysis for Passenger-km

The same approach as the projection of passenger-km was adopted for that of ton-km. Figure 7.2.3 illustrates the relationship between ton-km and GDP from 1993/94 to 2003/04 with the result of the regression analysis. The straight line of the same slope intersecting the point of 2003/04 was adopted for the projection of freight ton-km.

[1] Compound Annual Growth Rate



Source: PTPS Phase-I (Elaborated by the JICA Study Team based on Pakistan Economic Survey, 2004-05)

Figure 7.2.3 Regression Analysis for Freight ton-km

The projection models are summarized as:

- Passenger-km (million passenger-km) = 54.29 × (GDP – 5,657.2) + 245,821
- Freight ton-km (million ton-km) = 25.08 × (GDP – 5,657.2) + 119,040

The unit of GDP is Rs. billion at 04/05 prices.

From the models above, passenger-km and freight ton-km were projected by scenario as shown in Table 7.2.3.

Table 7.2.3 Projection of Passenger-km and Freight ton-km by Scenario

Year	Passenger-km (billion passenger km)			Freight ton-km (billion ton-km)		
	High	Medium	Low	High	Medium	Low
2005/06	295	295	295	142	142	142
2010/11	452	438	415	214	208	197
2015/16	679	623	518	319	293	245
2020/21	1,007	854	644	471	400	303
2025/26	1,480	1,135	756	689	530	355

Source: PTPS Phase-I (Projection by the JICA Study Team)

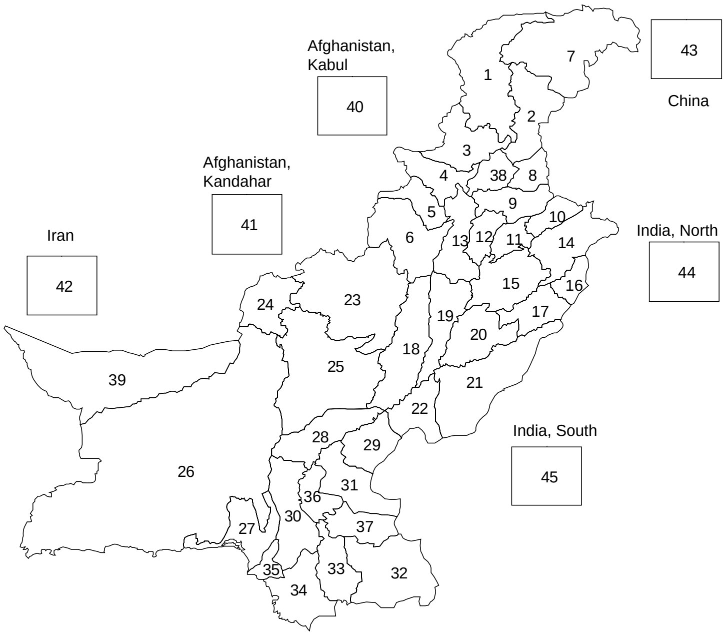
(3) Inter-zonal Traffic Demand

The projected passenger-km and freight ton-km in Table 7.2.3 are the overall land transport demand in Pakistan. This includes not only inter-district transport, but also inner district transport. Therefore, it is not proper to apply the result of the projection directly to traffic demand of national highways between major cities. On the other hand, the PTPS transport model was formulated based on traffic zones each of which consist of a few districts. Figure 7.2.4 illustrates the PTPS Traffic Zones. Inter-zonal transport is defined in the PTPS as the transport between traffic zones. The proportion of inter-zonal transport to the overall transport volume has declined as shown in Table 7.2.4. The sharp drop in the proportion between 1992/93 and 2005/06 is the result of an unnatural increase in freight ton-km in 1992/93.

Table 7.2.4 Share of Interzonal Transport

	Passenger-km			Ton-km		
	Road	Rail	Total	Road	Rail	Total
1980-81	0.554	0.912	0.626	0.907	0.984	0.930
1985-86	0.473	0.938	0.542	0.789	1.000	0.838
1992-93	0.526	0.967	0.576	0.774	0.979	0.803
2005-06	0.473	0.950*	0.524	0.659	0.950*	0.700

Source: PTPS Phase-I (NTPS JICA in 1983, 1988, and 1995, and PTPS)
Note: * Assumption



Source: PTPS Phase-I

Figure 7.2.4 PTPS Traffic Zones

It is expected that the large cities will continue to grow and the number of trips will increase within traffic zones at a higher rate than interzonal trips. On the other hand, trade and industrial trips will also grow at a rate that will be able to support economic growth. Therefore, the proportion of interzonal trips will decrease to some extents at a low pace.

The JICA Study Team assumes the future rates of interzonal transport to the overall land transport as shown in Table 7.2.5, and interzonal transport were projected from the rates as shown in Table 7.2.6. Differences in passenger-km and ton-km among scenarios were computed as shown in Table 7.2.7.

Table 7.2.5 Assumption of Rates of Interzonal Transport to Overall Land Transport

	2010/11	2015/16	2020/21 – 2030/31
Passenger-km	0.485	0.470	0.455
Freight ton-km	0.640	0.630	0.620

Source: PTPS Phase-I (Assumption by the JICA Study Team)

Table 7.2.6 Projection of Interzonal Transport by Scenario

Year	Passenger-km (billion passenger km)			Freight ton-km (billion ton-km)		
	High	Medium	Low	High	Medium	Low
2010/11	219	212	201	137	133	126
2015/16	319	293	244	201	185	154
2020/21	458	389	293	292	248	188
2025/26	673	517	344	427	329	220

Source: PTPS Phase-I (Projection by the JICA Study Team)

Table 7.2.7 Projection of Interzonal Transport by Scenario

Year	Passenger-km (billion passenger km)			Freight ton-km (billion ton-km)		
	High	Medium	Low	High	Medium	Low
2010/11	+3.2%	1	-5.2%	+3.1%	1	-5.1%
2015/16	+9.1%	1	-16.8%	+8.9%	1	-16.5%
2020/21	+17.9%	1	-24.6%	+17.6%	1	-24.3%
2025/26	+30.3%	1	-33.4%	+30%	1	-33.0%

Source: PTPS Phase-I (Projection by the JICA Study Team)

(4) Future O/D Table

The present OD tables for passengers and freight tons were prepared from the PTPS Traffic Survey carried out in 2005. The future trip generation & attraction at each traffic zone was estimated from the present OD table assuming that the volumes will increase at the same growth rate as Gross Regional Domestic Product (GRDP). Since GRDPs were not available, GDP was allocated to districts in proportion of population. For the future O/D tables, tentative O/D tables were computed by the fratar method from the present O/D and the future trip generation & attraction. Since passenger-km and ton-km calculated from the tentative O/D tables do not necessarily equal to the projected interzonal transport, the O/D tables were adjusted so that the O/D tables could correspond to the projection of interzonal transport.

When converting passenger volumes and freight volumes to the number of vehicle, it was assumed that the present vehicle composition, passenger occupancy rates, the average loading of a truck will be the same in the future.

(5) Modal Share

The PTPS Phase-I proposed ambitious investment plan on railway system so that Pakistan Railways would be able to play an important role in freight transport because it would be desirable for Pakistan economy. As far as transport distance concerned, railway transport between major cities like Karachi, Lahore, Islamabad and Peshawar is competitive against road transport. PTPS Phase-I prepared two scenarios for modal share: (1) modal sift case and (2) without modal shift case. In the Phase-II, “without modal shift case” was applied for the traffic forecast, considering the present situation of railway projects.

(6) Traffic Assignment

Future O/D matrices were produced by scenario based on the estimated increase in inter-zonal traffic of passenger and freight, and O/D matrices were input to the road network model using JICA STRADA. In the traffic assignment, the traffic volume in the Kohat Tunnel (N-55 between Peshawar and Kohat) was calculated as shown in Tables 7.2.8.

Table 7.2.8 PTPS Traffic Demand Forecast for the Kohat Tunnel

		2005/06	2010/11	2015/16	2020/21	2025/26
High Growth Scenario	PCU	13,780	17,555	25,105	35,235	48,881
	Increase Ratio	1.00	1.43	1.82	2.56	3.55
	CAGR	-	7.0%	7.4%	7.0%	6.8%
	GDP Growth Rate	-	7 – 8.2%	7.6%	7.6%	7.6%
Medium Growth Scenario	PCU	13,780	17,344	23,387	31,115	40,742
	Increase Ratio	1.00	1.26	1.70	2.26	2.96
	CAGR	-	4.7%	6.2%	5.9%	5.5%
	GDP Growth Rate	-	7.0%	6.5%	6.0%	5.5%
Low Growth Scenario	PCU	13,780	16,055	19,331	23,118	27,292
	Increase Ratio	1.00	1.17	1.40	1.68	1.98
	CAGR	-	3.1%	3.8%	3.6%	3.4%
	GDP Growth Rate	-	5 - 7%	4.0%	4.0%	3.0%

Note: Projection by the JICA Study Team

As seen in the results of traffic assignment, the annual increase rates of the traffic volume in the Kohat Tunnel were estimated to be similar to that of GDP in the medium growth scenario. Differences of the projection among the scenarios were calculated as follows:

Table 7.2.9 Differences in the Projection to Medium Growth Case

	2010/11	2015/16	2020/21	2025/26
High	+1.2%	+7.3%	+13.2%	+20.0%
Low	-7.4%	-17.3%	-25.7%	-33.0%

Note: Projection by the JICA Study Team

7.2.2 Impact of Transport Projects on Demand Forecast

(1) Impact of the Khushalgarh Bridge Project

Construction of a new bridge at Khushalgarh in place of the existing narrow bridge is included in MTDF, as well as its access road Kohat - Jand - Fatehjang. Articulated trucks like container trucks cannot pass through the existing bridge because there are cranks at both ends of the bridge where large trucks cannot make a turn. If the new Khushalgarh Bridge is constructed, a part of the traffic between Kohat and Rawalpindi will be diverted from the Kohat Tunnel (N-55) to the Khushalgarh Bridge (N-80). On the other hand, a part of the traffic between Peshawar and Rawalpindi will be diverted from N-5 or M-1 to the Kohat Tunnel - Khushalgarh Bridge. The former shift will decrease the traffic the tunnel while the latter will increase. Using PTPS Traffic Model, traffic assignments were carried out to analyze this. The results indicated that the difference of traffic volume would be small as shown in Table 7.2.10. From this, the impact of the Khushalgarh Bridge Project was not considered in the traffic demand forecast for the 2nd Kohat Tunnel Project.

Table 7.2.10 Changes in Traffic Volume

2010	2015	2020	2025
0%	6%	4%	-3%

Note: % : (With K. Bridge Case – Without K. Bridge Case)/Without K. Bridge Case
:Projection by the JICA Study Team

(2) Impact of the 2nd Kohat Tunnel

The increase of traffic handling capacity of the Kohat Tunnel and Access Road will induce route diversion to some extent. This will occur when other alternative routes become congested. To evaluate the impact of the 2nd Kohat Tunnel and dualization of the Access Road, traffic assignments of PTPS Traffic Model were carried out. Without the Kushalgarh Bridge Project, the traffic increase in the Kohat Tunnel would not be significant at 3% in 2020 and 5% in 2025. If the new Kushalgarh Bridge is open, a part of the traffic using the existing Kohat Tunnel route would choose N-80, which would reduce the traffic volume (especially that of trucks) in the Kohat Tunnel. The 2nd Kohat Tunnel will be able to regain the diverted traffic. In addition, the Kohat Tunnel - Khushargarh Bridge route would attract other traffics. It was computed that the 2nd Kohat Tunnel Project will increase the traffic volume along the Access Road by 14% in 2020, and 4% in 2025.

Table 7.2.11 Changes in Traffic Volume (PCU) by Kohat Tunnel

	2010	2015	2020	2025
Without Kushalgarh Bridge	1 %	1 %	3 %	5 %
With Kushalgarh Bridge	4 %	14 %	14 %	4 %

Note: (With 2nd Tunnel Case – Without 2nd Tunnel Case)/Without 2nd Tunnel Case
: Projection by the JICA Study Team

For the demand forecast of the 2nd Kohat Tunnel in the Phase-II, this kind of induced traffic was not separated from the total traffic because the projection model would become complex if such classification of demand was introduced. Instead, a simple growth model which assumed annual growth rates of traffic in the Kohat Tunnel was applied for the projection.

7.2.3 Traffic Growth Rate

(1) Review of Past Traffic Projection for Kohat Tunnel

The projection for the Kohat Tunnel before its opening was different from the actual traffic due to the following two reasons: 1) the relationship between ADT and AADT was not proper in the projection; and 2) GDP growth rate was 3.3% from 1995-96 to 2002-03 which was lower than the assumed growth rate of 5%. Although traffic data before the opening is not enough, traffic volume might have increased at the similar growth rate judging from the limited data.

(2) Trend Analysis after the Opening of the Kohat Tunnel

As mentioned above, the traffic volume in the Kohat Tunnel increased by 12.4% from 2004 to 2005 (calendar year). The growth rates based on fiscal years are:

- From 2003/04 to 2004/05: 25.3% (GDP growth rate = 8.3%)
- From 2004/05 to 2005/06: 10.6% (GDP growth rate = 6.6%)

The traffic growth rate in the Kohat Tunnel has been higher than the GDP growth rate from its opening in 2003. The rapid traffic increase after the opening of the tunnel showed a drastic route diversion to the tunnel and emerging of induced traffic as a result of the impact of the Kohat Tunnel. The route diversion from the old route (Kohat Pass) to the Kohat Tunnel has already almost completed, judging from traffic data below.

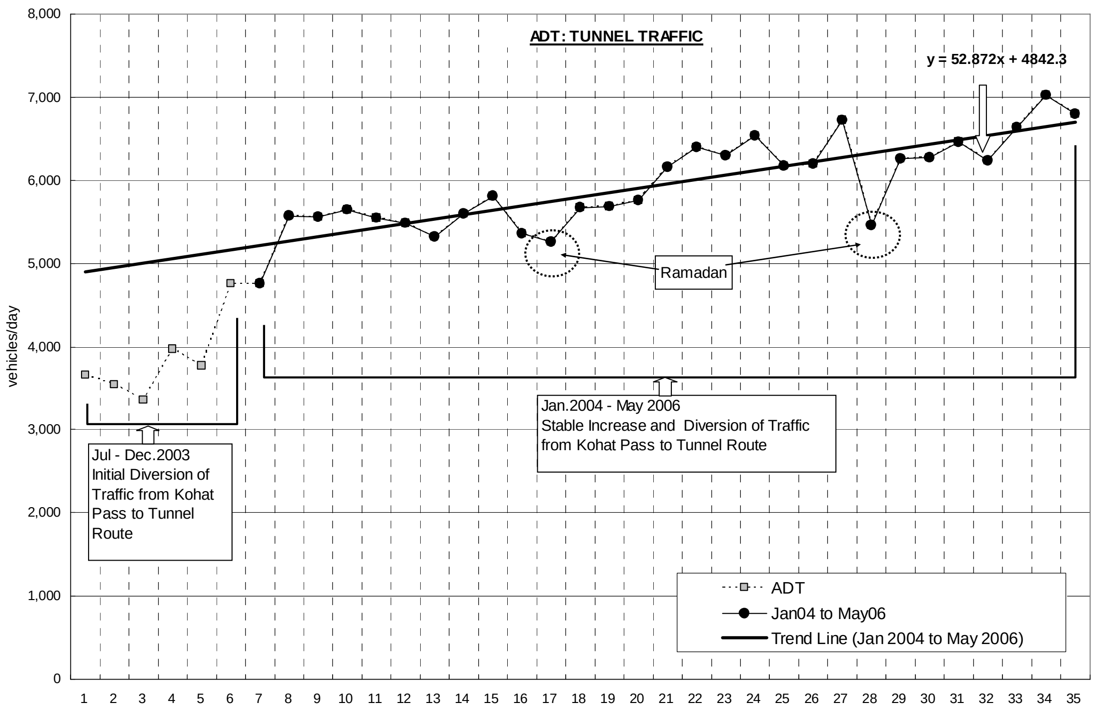
Table 7.2.12 Traffic Volume of Kohat Tunnel and Kohat Pass

	2004	2005
Kohat Tunnel	5,463 (87.5%)	6,149 (89.5%)
Kohat Pass	781 (12.5%)	720 (10.5%)
Total	6,244	6,869

Note: Traffic data of the Kohat Tunnel was taken from Toll & Traffic Data of NHA

On the other hand, the result of the traffic assignment gives higher traffic volume by 14.5% at the Kohat Tunnel compared to the result of the PTPS Traffic Survey in 2005[1]. This implies that diversion from other routes to N-55 via Kohat Tunnel still remains, because the traffic assignment assumes that drivers choose the cost-minimum path.

In addition, the high traffic growth in the last year implies that new traffic demand (induced traffic) has been generated after the opening of the Kohat Tunnel and the emerging of induced traffic will continue for the next several years. To estimate traffic growth rates for the next several years, a trend analysis was carried out as shown in Figure 7.2.4.



Note: Projection by the JICA Study Team based on data from NHA

Figure 7.2.4 Trend Analysis of Traffic Volume at the Kohat Tunnel

[1] Traffic Count= 12,033 PCU; Traffic Assignment = 13,780 PCU

Table 7.2.13 shows the result of the trend analysis. If the past trend continues, the annual growth rate will gradually decrease from about 10% to 7% for the next five years. Although it is uncertain whether the trend in the last two years can explain the traffic in the future, the trend line in Figure 7.2.4 suggests that the growth will continue for several years.

Table 7.2.13 Traffic Projection of Kohat Tunnel by Trend Analysis.

Year (at Dec.)	ADT	Annual Growth Rate
2003	5,160	
2004	5,794	12.3%
2005	6,428	11.0%
2006	7,063	9.9%
2007	7,697	9.0%
2008	8,332	8.2%
2009	8,966	7.6%
2010	9,601	7.1%
2011	10,235	6.6%

Note: ADT = 52.872 x + 4,842.3; x: The number of months from July 2003;
Projection by the JICA Study Team

A land development area is observed along the Link Road in the north of Kohat City. It is expected that population of Kohat City will increase higher than the projection in the Phase-I of the PTPS in which such land development was not considered because the socioeconomic framework applied in the Phase-I was estimated based on nationwide analysis. Taking these analyses into consideration, the traffic growth rates in the Kohat Tunnel were assumed as follows:

Table 7.2.14 Assumption of Traffic Growth Rates

Fiscal Year	GDP Growth Rate	Annual Traffic Growth Rate %	Remark
2006/07	7.3*1	10.0	
2007/08	7.0	9.0	
2008/09	7.0	8.0	
2009/10	7.0	7.5	
2010/11	6.5	7.0	
2011/12 – 2014/15	6.5	6.5	From the results of the Traffic Assignment of PTPS Traffic Model for the Medium Growth Scenario
2015/16 – 2019/20	6.0	6.0	
2020/21 – 2024/25	5.5	5.5	
2025/26 –	5.0	5.0	

Note: *1/ Asian Development Outlook, 2006
Projection by the JICA Study Team

7.2.4 Traffic Demand Forecast for the Kohat Tunnel

(1) Traffic Volume in the Kohat Tunnel

From the traffic survey results, some basic data were calculated. The Average Daily Traffic (ADT) of the Kohat Tunnel for the base year (2006) was calculated at 7,366 veh/day, or 10,872 Passenger Car Units (PCUs). This is the 24-hour traffic volume at IC-3 on May 30, 2006. The peak hour traffic volume was calculated at 520 veh/h or 690 PCUs/h for both directions. Directional split at the peak hours was 58% for the direction from Peshawar to Kohat. The Peak Hour Factor was 0.87. The percentage of buses and trucks in ADT was adopted as the same in the peak hour traffic (buses: 2%; trucks: 26.5%).

Table 7.2.15 24-Hour Traffic Volume of the Base Year (2006)

Vehicle Type	Car	Wagon	Large Bus	2-Axle Truck	3-Axle Truck	Articulated Truck	Total
veh/day	3,264	2,004	147	1,055	512	384	7,366
PCU/day	3,264	2,004	294	2,110	1,280	1,920	10,872
PCU	1.0	1.0	2.0	2.0	2.5	5.0	-

Source: Projection by the JICA Study Team based on the PTPS Traffic Survey Phase-II (May 30, 2006 at IC-3)

Applying the revised growth rate, the future traffic volume in the Kohat Tunnel in the "without Khushalgarh Bridge Project" scenario was estimated as shown in Table 7.2.10. Since the impact of the bridge project is assumed to be neutral for the traffic demand forecast for the Kohat Tunnel, this result can be used for the cases of "with" and "without" the Kushalgarh Bridge Project. The future traffic volume indicated in Table 7.2.16 was applied for the north section (Kohat Tunnel – Link Road).

Table 7.2.16 Future Traffic Volume in the Kohat Tunnel

Year	Growth Rate (%)	ADT (veh/day)	ADT (PCU/day)	Peak Hour Traffic Volume	Vehicular flow rate for peak-15 min
2006	-	7,366	10,872	520	598
2007	10.0	8,103	11,959	572	658
2008	9.0	8,832	13,036	624	717
2009	8.0	9,538	14,078	673	774
2010	7.5	10,254	15,134	724	832
2011	7.0	10,972	16,194	775	890
2012	6.5	11,685	17,246	825	948
2013	6.5	12,444	18,367	879	1,010
2014	6.5	13,253	19,561	936	1,075
2015	6.0	14,048	20,735	992	1,140
2016	6.0	14,891	21,979	1,051	1,208
2017	6.0	15,785	23,298	1,114	1,281
2018	6.0	16,732	24,695	1,181	1,358
2019	6.0	17,736	26,177	1,252	1,439
2020	5.5	18,711	27,617	1,321	1,518
2021	5.5	19,740	29,136	1,394	1,602
2022	5.5	20,826	30,738	1,470	1,690
2023	5.5	21,971	32,429	1,551	1,783
2024	5.5	23,180	34,213	1,636	1,881
2025	5.0	24,339	35,923	1,718	1,975

Note: Projection by the JICA Study Team

(2) Traffic Volume on the South Section of Access Road (Starting Point - N80 - Link Road)

The traffic volume on the Access Road between the Link Road and N80 is about 60% of that of the Kohat Tunnel, while the Link Road accounts for about 40%. The base year traffic volume on this section in ADT was calculated to be 4,127 veh/ day from the traffic survey as shown in Table 7.2.17. The traffic volume between N80 and the start point of the Access Road is almost the same.

Table 7.2.17 24-hour Traffic Volume of the Base Year (2006)

Vehicle Type	Car	Wagon	Large Bus	2-Axle Truck	3-Axle Truck	Articulated Truck	Total
veh/day	1,479	931	123	778	462	354	4,127
PCU/day	1,479	931	246	1,556	1,155	1,770	7,173
PCU	1.0	1.0	2.0	2.0	2.5	5.0	-

Source: Projection by the JICA Study Team based on the PTPS Traffic Survey, Phase II (May 29, 2006 at IC-2)

Applying the growth rates to the base year traffic, the average daily traffic was calculated at 7,871 veh/day in 2015/2016, 10,483 veh/day in 2020/2021, and 13,636 veh/day in 2025/2026.

There are an entering ramp and an exiting ramp at the junction IC-3 between the Kohat Access Road and the Link Road. The entering ramp provides a connection from the Link Road to the Kohat Tunnel, while the exiting ramp provides a connection from the Kohat Tunnel to the Link Road. Ramps for the connection between the Link Road and the South section of the Kohat Access Road are not provided at present. It is proposed to provide such ramps so that the Kohat Access Road can function as a bypass road of Kohat City.

If the new ramps are provided, a part of the traffic to the east and to the south of Kohat City will divert from the existing route, where congestion is very heavy due to economic activities along the route, to the Access Road as a bypass. From the northern part of Kohat City to Bannu, the travel time will be reduced by 5 - 10 minutes. The travel time from the northern part of Kohat City towards Khushalgarh will be reduced by about 15 minutes. Toll payment will not be required for the bypass route.

To estimate the diversion traffic, the following percentages were used:

- Percentage of traffic whose origin or destination is in northern Kohat
- Percentage of traffic which will divert from the existing route to the bypass

From the above percentages, the diverted traffic volume was computed to be 1,760 veh/ day in the base year as shown in Table 7.2.18. It was assumed that most of buses would not change their route because they cannot catch passengers along the bypass. For this reason, the traffic volumes on the new ramps are computed as shown in Table 7.2.19.

Table 7.2.18 Estimated Diverted Traffic to the Access Road via Link Road

Original Route	Traffic volume of the base year (veh/day) *	Percentage of traffic whose origin or destination is in northern Kohat (%)	Percentage of diverted traffic to the Access Road (%)	Diverted traffic to the Access Road of the base year (veh/day)
Kohat City ←→ Khushargarh (via IC-2)	3,870	40	80	1,240
Kohat City ←→ Bannu (via IC-1)	3,460	30	50	520
Original Route				1,760

Note*: Buses are excluded from traffic volumes
: Projection by the JICA Study Team

Table 7.2.19 Base Year Traffic between Link Road and Access Road (IC2 – IC3)

Vehicle Type	Car	Wagon	Large Bus	2-Axle Truck	3-Axle Truck	Articulated Truck	Total
East	758	357	15	53	28	29	1,240
South	367	68	6	64	13	2	520
Total	1,125	425	21	117	41	31	1,760

Note: Projection by the JCIA Study Team

The future traffic volume on the Access Road for the section of SP- N80 - Link Road was computed based on the base year traffic and the growth rates as shown in Table 7.2.13. If the new ramps are provided, the traffic between N80 and Link Road would reach 14,900 veh/day in 2020/2021. The traffic on the new ramps would be about 5,800 veh/day, or 2,650 veh/day in each direction in 2025/2026.

Table 7.2.20 Future Traffic Volume on the Access Road (SP- N80 – Link Road)

Year	Without New Ramp	With New Ramp		
	SP* - Link Road	N80 – Link Road	SP – N80	Ramp
2010-11	5,745	8,195	6,469	2,450
2015-16	7,871	11,228	8,863	3,357
2020-21	10,483	14,954	11,804	4,471
2025-26	13,630	19,452	15,355	5,815

Note: Both directions
: Projection by the JICA Study Team

7.3 Capacity Analysis

7.3.1 Tunnel

(1) Capacity of the Kohat Tunnel

The Kohat Tunnel is a two-lane road. The Highway Capacity Manual (HCM) 2000, published by the Transportation Research Board (TRB), USA, states that the ideal capacity of a two-lane highway is 1,700 pc/h in each direction or 3,200 pc/h in both directions. On the other hand, the ideal capacity of 2,800 pc/h in both directions, which is applied in HCM 1985, has been commonly used for two-lane highways. Since HCM 2000 does not give a methodology to adjust the ideal capacity to the actual capacity, and the ideal capacity of 3,200 pc/h seems to be high, the HCM 1985 was applied in calculating the capacity of the Kohat Tunnel. It is noteworthy that the ideal capacity of 2,500 pc/h is commonly used in Japan.

In HCM 1985, the capacity of a two-lane road is calculated as a service flow rate for the level of service (LOS) F as follows:

$$SF_i = 2800\,(v/c)_i\, f_d\, f_w\, f_{HV}$$

Where,
- SF_i = service flow rate for LOS i, veh/h in both directions
- $(v/c)_i$ = maximum permissible v/c ratio for LOS i
- f_d = adjustment factor for directional distribution
- f_w = adjustment factor for narrow lanes and/or shoulders
- f_{HV} = adjustment factor for heavy vehicles

In the detailed design (D/D) of the Kohat Tunnel Project, the capacity of the tunnel was calculated at 1,861veh/h using the above formula.

Table 7.3.1 Capacity of Tunnel Section in D/D

LOS	SF_i	Ideal Capacity	$(v/c)_i$	f_d	f_w	f_{HV}
A	61	2,800	0.04	0.97	0.76	0.744
B	233	2,800	0.16	0.97	0.76	0.706
C	466	2,800	0.32	0.97	0.76	0.706
D	886	2,800	0.57	0.97	0.76	0.753
E	1,861	2,800	1.00	0.97	0.91	0.753

Source: Kohat Tunnel D/D Report

The results of traffic survey showed that there are minor differences in the conditions of traffic flow which affects the adjustment factor, but the changes are not significant. Instead, it is necessary to review the calculation for the following reasons:

Influences of a tunnel section were not considered in the D/D to compute the capacity. Narrow lanes and/or shoulders were considered as f_w, but this is not enough to reflect tunnel conditions such as low visibility, feeling of pressure, sudden change in environment at portals, and so on. Some traffic data in Japan shows that the traffic capacity of a tunnel is about 80% of its access roads. The adjustment factors in the table above contribute only 88% ($f_d \times f_w = 0.97 \times 0.91$). In order to consider tunnel conditions, PTPS adopted a factor of 0.70 for f_w when LOS is from A to D. In addition, the adjustment factor for directional distribution was also revised to 0.94 based on the result of traffic survey.

Since HCM 1985 was developed based on traffic conditions in USA, it is necessary to consider traffic conditions in Pakistan, especially for trucks. It is observed in Pakistan that trucks run at a crawl even in flat sections. The 2%-gradient in the Kohat Tunnel is a hard condition for Pakistani trucks. This means that passenger-car equivalents in HCM 1985 should be adjusted. For this, PTPS adopted the following values to compute f_{HV}.

Table 7.3.2 Passenger-Car Equivalents (PCE) for Trucks for the Kohat Tunnel

LOS	D/D	PTPS
A	2.0	2.5
B & C	2.2	2.7
D & E	2.0	2.5

Source: Kohat Tunnel D/D Report (PCE for D/D)

From these reviewed parameters, the capacity of the Kohat Tunnel is computed to be 1,642 veh/h in both directions as shown in the table below.

Table 7.3.3 Revised Capacity of the Kohat Tunnel

LOS	SF_i	Ideal Capacity	$(v/c)_i$	f_d	f_w	f_{HV}
A	52	2,800	0.04	0.94	0.70	0.707
B	200	2,800	0.16	0.94	0.70	0.680
C	401	2,800	0.32	0.94	0.70	0.680
D	745	2,800	0.57	0.94	0.70	0.709
E	1,642	2,800	1.00	0.94	0.88	0.709

Note: Projection by the JICA Study Team

(2) Demand and Capacity Analysis

The table below shows the results of estimate of the v/c ratio and LOS for the Kohat Tunnel in the future. According to this, LOS has already become D at peak hour this year. The Kohat tunnel will experience LOS of E for a decade from 2009/2010 to 2021/2022, and will reach the capacity in 2022/2023. It should be noted that a peak hour rate of 7.1%, taken from the traffic count survey, was applied in computing the hourly volume for the analysis. The relatively low rate of 7.1% means that the peak is moderate and therefore similar traffic volumes are observed during daytime. In other words, the situation that LOS is E will continue in daytime in the near future.

Table 7.3.4 Traffic Demand & Capacity of the Kohat Tunnel

Year	Growth Rate (%)	ADT (veh/day)	Hourly Volume (veh/h)	Vehicle Flow Rate (veh/h)	v/c	LOS
2006-07		7,366	520	598	0.4	D
2007-08	10.0	8,103	572	658	0.4	D
2008-09	9.0	8,832	624	717	0.5	D
2009-10	8.0	9,538	673	774	0.5	E
2010-11	7.5	10,254	724	832	0.6	E
2011-12	7.0	10,972	775	890	0.6	E
2012-13	6.5	11,685	825	948	0.6	E
2013-14	6.5	12,444	879	1,010	0.7	E
2014-15	6.5	13,253	936	1,075	0.7	E
2015-16	6.0	14,048	992	1,140	0.8	E
2016-17	6.0	14,891	1,051	1,208	0.8	E
2017-18	6.0	15,785	1,114	1,281	0.9	E
2018-19	6.0	16,732	1,181	1,358	0.9	E
2019-20	6.0	17,736	1,252	1,439	1.0	E
2020-21	5.5	18,711	1,321	1,518	1.0	E
2021-22	5.5	19,740	1,394	1,602	1.1	E
2022-23	5.5	20,826	1,470	1,690	1.1	F
2023-24	5.5	21,971	1,551	1,783	1.2	F
2024-25	5.5	23,180	1,636	1,881	1.3	F
2025-26	5.0	24,339	1,718	1,975	1.3	F

Note: Projection by the JICA Study Team

7.3.2 Access Road

(1) Capacity of Access Road

The capacity of general sections of the Access Road is higher than that of the tunnel section. The same methodology for two-way segments applied in item 7.3.1 can be also applied for the general sections. The difference in conditions between these general sections and the tunnel section are:

- Passing is possible.
- Shoulder space is large enough.

The former affects the v/c value while the latter affects the fw value. The same values can be applied for other parameters. Accordingly, the capacity of these sections was computed to be 1,866 veh/h as shown in the table below.

Table 7.3.5 Service Flow Rates on General Sections of Access Road

LOS	SF_i	Ideal Capacity	$(v/c)_i$	f_d	f_w	f_{HV}	E_T	E_B
A	279	2,800	0.15	0.97	1.00	0.707	2.5	1.8
B	483	2,800	0.27	0.97	1.00	0.680	2.7	2.0
C	770	2,800	0.43	0.97	1.00	0.680	2.7	2.0
D	1,194	2,800	0.64	0.97	1.00	0.709	2.5	1.6
E	1,866	2,800	1.00	0.97	1.00	0.709	2.5	1.6

Note: Projection by the JICA Study Team

(2) Demand and Capacity Analysis

a) Starting Point - N80 - Link Road

The volume to capacity ratios (v/c) were computed from flow rates at the peak hour and the capacity of 1,866 veh/h. The flow rates were calculated as follows:

Flow Rate (veh/h) = AADT × K × PHF

Where, K = the ratio of designed hourly traffic volume to AADT (0.0706)
PHF = peak hour factor (0.92)

LOSs were determined from the service flow rates shown in Table 7.3.5 and the flow rates.

Table 7.3.6 shows v/c and LOS of the Access Road from the Starting Point to the Link Road. The table indicates that the traffic volume will not exceed the capacity. However, the v/c ratio between N80 and Link Road will reach 0.8 in 2023/2024. LOS of this section will become E in 2020/2021.

b) Link Road - Ending Point

Table 7.3.6 shows the results of the demand-capacity analysis for the section between the Link Road and Ending Point of the Access Road. LOS will become E in 2016/2017. The v/c ratio will reach 0.8 in 2019/2020, and reach 0.9 in 2021/2022. The traffic will reach the capacity in 2024/2025.

It should be noted that the traffic stream near the tunnel tends to be disturbed because vehicle speeds often change at the portal of the tunnel. Therefore, even if the v/c ratio is less than 1.0, traffic congestion will occur when the v/c ratio is high, because the capacity will decrease once disturbed traffic flow causes congestion.

Table 7.3.6 Future v/c and LOS of the Access Road (SP – N80 – Link Road)

K-Factor = 0.0706

	SP - N80				N80 - Link Road			
Year	AADT (veh/day)	Flow Rate (veh/h)	v/c	LOS	AADT (veh/day)	Flow Rate (veh/h)	v/c	LOS
2006-07	4,647	377	0.2	B	5,887	478	0.3	B
2007-08	5,112	415	0.2	B	6,476	525	0.3	C
2008-09	5,572	452	0.2	B	7,059	573	0.3	C
2009-10	6,017	488	0.3	C	7,623	619	0.3	C
2010-11	6,469	525	0.3	C	8,195	665	0.4	C
2011-12	6,922	562	0.3	C	8,769	712	0.4	C
2012-13	7,372	598	0.3	C	9,339	758	0.4	C
2013-14	7,851	637	0.3	C	9,946	807	0.4	D
2014-15	8,361	678	0.4	C	10,592	860	0.5	D
2015-16	8,863	719	0.4	C	11,228	911	0.5	D
2016-17	9,394	762	0.4	C	11,901	966	0.5	D
2017-18	9,958	808	0.4	D	12,615	1,024	0.5	D
2018-19	10,556	857	0.5	D	13,372	1,085	0.6	D
2019-20	11,189	908	0.5	D	14,174	1,150	0.6	D
2020-21	11,804	958	0.5	D	14,954	1,214	0.6	E
2021-22	12,453	1,011	0.5	D	15,777	1,280	0.7	E
2022-23	13,138	1,066	0.6	D	16,644	1,351	0.7	E
2023-24	13,861	1,125	0.6	D	17,560	1,425	0.8	E
2024-25	14,623	1,187	0.6	D	18,525	1,503	0.8	E
2025-26	15,355	1,246	0.7	E	19,452	1,578	0.8	E

Note: v/c = vehicle flow rate/1866
: Projection by the JICA Study Team

Table 7.3.7 Future LOS of the Access Road (South and North of the Tunnel)

K-Factor = 0.0706

Year	Growth Rate (%)	ADT (veh/day)	Hourly Volume (veh/h)	Vehicle Flow Rate (veh/h)	v/c	LOS
2006-07		7,366	520	598	0.3	C
2007-08	10.0	8,103	572	658	0.4	C
2008-09	9.0	8,832	624	717	0.4	C
2009-10	8.0	9,538	673	774	0.4	D
2010-11	7.5	10,254	724	832	0.4	D
2011-12	7.0	10,972	775	890	0.5	D
2012-13	6.5	11,685	825	948	0.5	D
2013-14	6.5	12,444	879	1,010	0.5	D
2014-15	6.5	13,253	936	1,075	0.6	D
2015-16	6.0	14,048	992	1,140	0.6	D
2016-17	6.0	14,891	1,051	1,208	0.6	E
2017-18	6.0	15,785	1,114	1,281	0.7	E
2018-19	6.0	16,732	1,181	1,358	0.7	E
2019-20	6.0	17,736	1,252	1,439	0.8	E
2020-21	5.5	18,711	1,321	1,518	0.8	E
2021-22	5.5	19,740	1,394	1,602	0.9	E
2022-23	5.5	20,826	1,470	1,690	0.9	E
2023-24	5.5	21,971	1,551	1,783	1.0	E
2024-25	5.5	23,180	1,636	1,881	1.0	F
2025-26	5.0	24,339	1,718	1,975	1.1	F

Note: v/c = vehicle flow rate/1866
: Projection by the JICA Study Team

7.3.3 Intersection

Currently, there is no signalized intersection along the Kohat Tunnel Access Road. A separate left-turn lane with a triangle island and unsignalized T-intersections compose an intersection at the start point of the access road (IC-1) and at the end point (IC-4). Ramps connect the access road and minor roads without signals. To analyze unsignalized intersections and ramps, HCM 2000 was referred to. Gap acceptance theory is the underlying concept for computing the capacity of minor roads at unsignalized intersections. The model is expressed as:

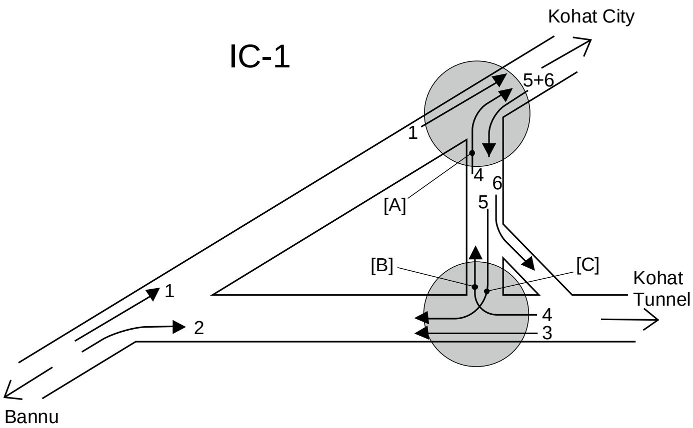
$$c_p = v_c \frac{\exp(-v_c t_c / 3600)}{1 - \exp(-v_c t_f / 3600)}$$

where: c_p = potential capacity of minor movement (veh/h)
v_c = conflicting flow rate (veh/h)
t_c = critical gap (s)
t_f = follow-up time (s)

(1) Present Performance

a) IC-1

For IC-1, the start point of the Kohat Tunnel Access Road, the following three movements at the peak hour were analyzed:



[A] right-turn from the minor road to the major road

[B] right turn from the major road to the minor road

[C] right-turn from the minor road to the major road

Source: PTPS Traffic Survey, Phase-II

Other movements are given priority at the intersection and the capacity is enough at present. For the selected three movements, v/c, queue length, and delay were estimated as shown in the table below. No problem was identified at IC-1 from the analysis.

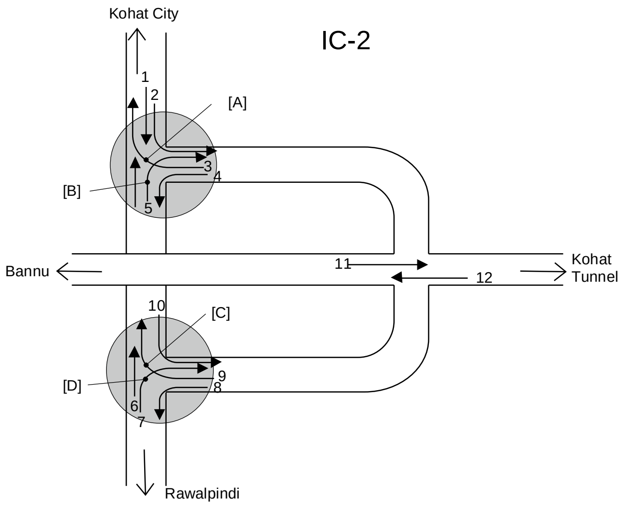
Table 7.3.1 Intersection Analysis for the Selected Movement at IC-1

	Peak flow rate (veh/h)	Conflicting flow rate (veh/h)	Movement capacity (veh/h)	v/c	Q95 (veh)	Delay (s)	LOS
[A]	20	384	602	0.0	0.1	11	B
[B]	175	377	608	0.3	1.2	13	B
[C]	20	255	1,259	0.0	0.0	8	A

Note: Projection by the JICA Study Team

b) IC-2

For IC-2, the interchange between the Access Road and N-80, the following four movements at the peak hour were analyzed. Currently, traffic volumes of these movements are very small.



Source: PTPS Traffic Survey, Phase-II

[A] right-turn from the direction to Bannu via the ramp to N-80

[B] right-turn from N-80 to the direction to the Kohat Tunnel via the ramp

[C] right-turn from the direction to the Kohat Tunnel via the ramp to N-80

[D] right-turn from N-80 to the direction to Bannu via the ramp

Other movements are given priority at the intersection and the capacity is enough at present. For the selected four movements, v/c, queue length, and delay were estimated as shown in the table below. No problem was identified at IC-2 from the analysis.

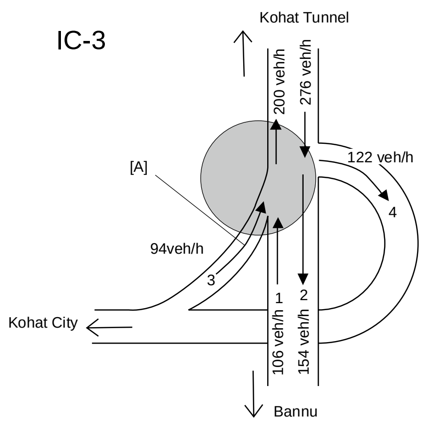
Table 7.3.2 Intersection Analysis for the Selected Movement at IC-1

	Peak flow rate (veh/h)	Conflicting flow rate (veh/h)	Movement capacity (veh/h)	v/c	Q95 (veh)	Delay (s)	LOS
[A]	12	394	548	0.0	0.1	12	B
[B]	12	211	1,318	0.0	0.0	8	A
[C]	13	404	462	0.0	0.1	13	B
[D]	7	197	1298	0.0	0.0	8	A

Note: Projection by the JICA Study Team

c) IC-3

The capacity of the interchange between the Access Road and the Link Road is enough, compared to the traffic volume at present.



Source: PTPS Traffic Survey, Phase-II

Since vehicles of the traffic movement of [A] have to stop at the end of the ramp, the intersection between the ramp and the Access Road was analyzed as unsignalized T-intersection. No problem was identified from the analysis.

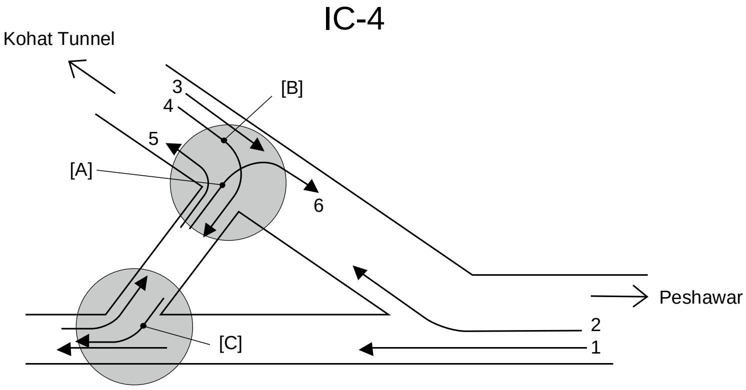
Table 7.3.3 Intersection Analysis for the Selected Movement at IC-3

	Peak flow rate (veh/h)	Conflicting flow rate (veh/h)	Movement capacity (veh/h)	v/c	Q95 (veh)	Delay (s)	LOS
[A]	102	115	906	0.1	0.4	9	B

Note: Projection by the JCIA Study Team

d) IC-4

For IC-4, the interchange at the end of the Access Road, the following three movements were analyzed.



[A] right-turn from the major road to the Access Road (to Peshawar)

[B] right-turn from the Access Road to the minor road

[C] the second right-turn after the right-turn of [B]

Source: PTPS Traffic Survey, Phase-II

The results of analysis show that v/c is 0.4 and LOS is C for the traffic movement of [A]. This means that the movement [A] will be the first that reach the capacity under the current conditions. No problem was identified for other movements.

Table 7.3.4 Intersection Analysis for the Selected Movement at IC-3

	Peak flow rate (veh/h)	Conflicting flow rate (veh/h)	Movement capacity (veh/h)	v/c	Q95 (veh)	Delay (s)	LOS
[A]	142	624	385	0.4	1.7	20	C
[B]	12	264	1,312	0.0	0.0	8	A
[C]	14	244	696	0.0	0.1	10	B

Note: Projection by the JICA Study Team

(2) Future Performance

The future values of movement capacity, v/c, queue length, and delay were estimated based on the estimated traffic growth rate. The traffic volume will exceed the capacity for some traffic movements. At IC-1, v/c of movement [C] will exceed 1.0 in 2017. At IC-2, v/c of movement [C] will exceed 1.0 in 2021. At IC-4, v/c of movement [A] will soon exceed 1.0 in 2012. These intersections should be signalized before the year when v/c exceeds 1.0.

Table 7.3.5 IC-1 Intersection Analysis

Year	Annual Growth Rate %	IC-1-[A] Peak Flow Rate	Capacity	v/c	Q95	d	LOS	IC-1-[B] Peak Flow Rate	Capacity	v/c	Q95	d	LOS
2006		20	651	0.0	0.1	11	B	20	1,259	0.0	0.0	8	A
2007	10.0	22	624	0.0	0.1	11	B	22	1,232	0.0	0.1	8	A
2008	9.0	24	597	0.0	0.1	11	B	24	1,206	0.0	0.1	8	A
2009	8.0	25	573	0.0	0.1	12	B	25	1,180	0.0	0.1	8	A
2010	7.5	27	549	0.0	0.2	12	B	27	1,156	0.0	0.1	8	A
2011	7.0	29	526	0.1	0.2	12	B	29	1,131	0.0	0.1	8	A
2012	6.5	31	504	0.1	0.2	13	B	31	1,107	0.0	0.1	8	A
2013	6.5	33	482	0.1	0.2	13	B	33	1,082	0.0	0.1	8	A
2014	6.5	35	459	0.1	0.2	13	B	35	1,057	0.0	0.1	9	A
2015	6.0	37	438	0.1	0.3	14	B	37	1,032	0.0	0.1	9	A
2016	6.0	40	416	0.1	0.3	15	B	40	1,006	0.0	0.1	9	A
2017	6.0	42	395	0.1	0.4	15	C	42	979	0.0	0.1	9	A
2018	6.0	45	373	0.1	0.4	16	C	45	952	0.0	0.1	9	A
2019	6.0	47	351	0.1	0.5	17	C	47	924	0.1	0.2	9	A
2020	5.5	50	331	0.2	0.5	18	C	50	897	0.1	0.2	9	A
2021	5.5	53	311	0.2	0.6	19	C	53	869	0.1	0.2	9	A
2022	5.5	55	291	0.2	0.7	20	C	55	841	0.1	0.2	10	A
2023	5.5	58	271	0.2	0.8	22	C	58	813	0.1	0.2	10	A
2024	5.5	62	252	0.2	0.9	24	C	62	784	0.1	0.3	10	A
2025	5.0	65	235	0.3	1.1	26	D	65	756	0.1	0.3	10	B

Year	Annual Growth Rate %	IC-1-[C] Peak Flow Rate	Capacity	v/c	Q95	d	LOS
2006		148	608	0.2	1.0	13	B
2007	10.0	162	578	0.3	1.2	14	B
2008	9.0	177	549	0.3	1.4	15	B
2009	8.0	191	523	0.4	1.7	16	C
2010	7.5	205	498	0.4	2.1	17	C
2011	7.0	220	474	0.5	2.5	19	C
2012	6.5	234	451	0.5	3.1	22	C
2013	6.5	249	428	0.6	3.8	25	D
2014	6.5	266	405	0.7	4.9	31	D
2015	6.0	281	383	0.8	6.2	39	E
2016	6.0	298	361	0.9	8.1	53	F
2017	6.0	316	339	1.0	10.6	79	F
2018	6.0	335	317	1.1	13.9	120	F
2019	6.0	355	295	1.3	17.8	180	F
2020	5.5	375	276	1.4	21.9	252	F
2021	5.5	396	256	1.6	26.5	341	F
2022	5.5	417	237	1.9	31.5	448	F
2023	5.5	440	219	2.2	36.8	577	F
2024	5.5	464	200	2.5	42.4	734	F
2025	5.0	488	184	2.9	47.8	905	F

Note: The new ramps at IC-3 are taken into consideration
: Projection by the JICA Study Team

Table 7.5.6 Intersection Analysis at IC-2

Year	Annual Growth Rate %	IC-2-[A] Peak Flow Rate	Capacity	v/c	Q95	d	LOS	IC-2-[B] Peak Flow Rate	Capacity	v/c	Q95	d	LOS
2006		12	569	0.0	0.1	11	B	68	1,294	0.1	0.2	8	A
2007	10.0	14	486	0.0	0.1	12	B	75	1,276	0.1	0.2	8	A
2008	9.0	15	455	0.0	0.1	13	B	81	1,258	0.1	0.2	8	A
2009	8.0	16	426	0.0	0.1	13	B	88	1,241	0.1	0.2	8	A
2010	7.5	17	398	0.0	0.1	14	B	94	1,224	0.1	0.2	8	A
2011	7.0	18	372	0.0	0.2	14	B	101	1,207	0.1	0.3	8	A
2012	6.5	20	348	0.1	0.2	15	B	107	1,191	0.1	0.3	8	A
2013	6.5	21	324	0.1	0.2	16	C	114	1,174	0.1	0.3	8	A
2014	6.5	22	300	0.1	0.2	17	C	122	1,155	0.1	0.4	8	A
2015	6.0	24	278	0.1	0.3	18	C	129	1,138	0.1	0.4	9	A
2016	6.0	25	256	0.1	0.3	19	C	137	1,120	0.1	0.4	9	A
2017	6.0	26	235	0.1	0.4	20	C	145	1,100	0.1	0.5	9	A
2018	6.0	28	214	0.1	0.4	22	C	154	1,080	0.1	0.5	9	A
2019	6.0	30	194	0.2	0.5	24	C	163	1,060	0.2	0.5	9	A
2020	5.5	31	176	0.2	0.6	26	D	172	1,040	0.2	0.6	9	A
2021	5.5	33	159	0.2	0.8	29	D	182	1,019	0.2	0.6	9	A
2022	5.5	35	142	0.2	0.9	32	D	191	998	0.2	0.7	9	A
2023	5.5	37	126	0.3	1.1	37	E	202	976	0.2	0.8	10	A
2024	5.5	39	111	0.3	1.4	43	E	213	954	0.2	0.9	10	A
2025	5.0	41	99	0.4	1.8	51	F	224	932	0.2	0.9	10	B

Year	Annual Growth Rate %	IC-2-[C] Peak Flow Rate	Capacity	v/c	Q95	d	LOS	IC-2-[D] Peak Flow Rate	Capacity	v/c	Q95	d	LOS
2006		13	488	0.0	0.1	12	B	74	1,373	0.1	0.2	8	A
2007	10.0	14	415	0.0	0.1	13	B	82	1,357	0.1	0.2	8	A
2008	9.0	16	386	0.0	0.1	14	B	89	1,340	0.1	0.2	8	A
2009	8.0	17	360	0.0	0.1	15	B	96	1,325	0.1	0.2	8	A
2010	7.5	18	336	0.1	0.2	15	C	103	1,309	0.1	0.3	8	A
2011	7.0	19	313	0.1	0.2	16	C	111	1,294	0.1	0.3	8	A
2012	6.5	21	291	0.1	0.2	17	C	118	1,279	0.1	0.3	8	A
2013	6.5	22	270	0.1	0.3	18	C	125	1,263	0.1	0.3	8	A
2014	6.5	23	249	0.1	0.3	19	C	134	1,246	0.1	0.4	8	A
2015	6.0	25	230	0.1	0.4	21	C	142	1,230	0.1	0.4	8	A
2016	6.0	26	211	0.1	0.4	22	C	150	1,213	0.1	0.4	8	A
2017	6.0	28	192	0.1	0.5	24	C	159	1,195	0.1	0.5	8	A
2018	6.0	30	174	0.2	0.6	26	D	169	1,177	0.1	0.5	9	A
2019	6.0	31	157	0.2	0.7	29	D	179	1,157	0.2	0.5	9	A
2020	5.5	33	142	0.2	0.9	33	D	189	1,139	0.2	0.6	9	A
2021	5.5	35	127	0.3	1.1	37	E	199	1,120	0.2	0.6	9	A
2022	5.5	37	113	0.3	1.3	43	E	210	1,100	0.2	0.7	9	A
2023	5.5	39	100	0.4	1.6	51	F	221	1,079	0.2	0.8	9	A
2024	5.5	41	88	0.5	2.1	63	F	234	1,057	0.2	0.8	9	A
2025	5.0	43	77	0.6	2.6	79	F	245	1,037	0.2	0.9	10	A

Note: The new ramps at IC-3 are taken into consideration
: Projection by the JICA Study Team

Table 7.3.7 Intersection Analysis at IC-3 and IC-4

Year	Annual Growth Rate %	IC-3-[A] Peak Flow Rate	Capacity	v/c	Q95	d	LOS	IC-4-[A] Peak Flow Rate	Capacity	v/c	Q95	d	LOS
2006		102	906	0.1	0.4	9	A	142	385	0.4	1.7	20	C
2007	10.0	112	893	0.1	0.4	10	A	156	349	0.4	2.2	23	C
2008	9.0	123	879	0.1	0.5	10	A	170	317	0.5	3.0	29	D
2009	8.0	132	867	0.2	0.5	10	A	184	288	0.6	4.1	37	E
2010	7.5	142	854	0.2	0.6	10	B	198	262	0.8	5.5	51	F
2011	7.0	152	842	0.2	0.7	10	B	212	238	0.9	7.5	77	F
2012	6.5	162	830	0.2	0.7	10	B	226	216	1.0	9.9	120	F
2013	6.5	173	817	0.2	0.8	11	B	240	194	1.2	12.9	190	F
2014	6.5	184	804	0.2	0.9	11	B	256	174	1.5	16.4	287	F
2015	6.0	195	791	0.2	1.0	11	B	271	156	1.7	19.9	406	F
2016	6.0	207	778	0.3	1.1	11	B	287	139	2.1	23.7	555	F
2017	6.0	219	763	0.3	1.2	12	B	305	123	2.5	27.6	744	F
2018	6.0	232	749	0.3	1.3	12	B	323	108	3.0	31.6	982	F
2019	6.0	246	734	0.3	1.5	12	B	342	94	3.7	35.8	1286	F
2020	5.5	260	719	0.4	1.6	13	B	361	82	4.4	39.6	1640	F
2021	5.5	274	704	0.4	1.8	13	B	381	71	5.4	43.6	2088	F
2022	5.5	289	689	0.4	2.1	14	B	402	61	6.6	47.7	2662	F
2023	5.5	305	673	0.5	2.4	15	B	424	51	8.2	51.9	3402	F
2024	5.5	322	656	0.5	2.7	16	C	447	43	10.3	56.1	4368	F
2025	5.0	338	641	0.5	3.1	17	C	470	37	12.8	60.1	5513	F

Year	Annual Growth Rate %	IC-4-[B] Peak Flow Rate	Capacity	v/c	Q95	d	LOS	IC-4-[C] Peak Flow Rate	Capacity	v/c	Q95	d	LOS
2006		12	1,312	0.0	0.0	8	A	14	696	0.0	0.1	10	B
2007	10.0	13	1,283	0.0	0.0	8	A	16	671	0.0	0.1	10	B
2008	9.0	14	1,255	0.0	0.0	8	A	17	647	0.0	0.1	11	B
2009	8.0	16	1,229	0.0	0.0	8	A	18	624	0.0	0.1	11	B
2010	7.5	17	1,202	0.0	0.0	8	A	20	602	0.0	0.1	11	B
2011	7.0	18	1,177	0.0	0.0	8	A	21	580	0.0	0.1	11	B
2012	6.5	19	1,151	0.0	0.1	8	A	23	560	0.0	0.1	12	B
2013	6.5	20	1,125	0.0	0.1	8	A	24	538	0.0	0.1	12	B
2014	6.5	22	1,098	0.0	0.1	8	A	26	517	0.0	0.2	12	B
2015	6.0	23	1,072	0.0	0.1	8	A	27	496	0.1	0.2	13	B
2016	6.0	24	1,045	0.0	0.1	9	A	29	475	0.1	0.2	13	B
2017	6.0	26	1,016	0.0	0.1	9	A	31	454	0.1	0.2	13	B
2018	6.0	27	988	0.0	0.1	9	A	32	433	0.1	0.2	14	B
2019	6.0	29	958	0.0	0.1	9	A	34	411	0.1	0.3	15	B
2020	5.5	31	930	0.0	0.1	9	A	36	391	0.1	0.3	15	C
2021	5.5	32	901	0.0	0.1	9	A	38	371	0.1	0.3	16	C
2022	5.5	34	871	0.0	0.1	9	A	40	351	0.1	0.4	17	C
2023	5.5	36	841	0.0	0.1	9	A	43	330	0.1	0.4	18	C
2024	5.5	38	810	0.0	0.1	10	A	45	310	0.1	0.5	19	C
2025	5.0	40	782	0.1	0.2	10	A	47	292	0.2	0.6	20	C

Source: Projection by the JICA Study Team

Chapter 8. DESIGN STANDARDS

8.1 General

According to the "Standards for Roads in Pakistan" published by NHA, roads shall be designed to meet the following basic criteria:

- Provide road users with a comfortable and stress-free driving environment
- Accommodate existing and future traffic needs
- Provide the highest practical and feasible level of safety to highway users
- Match the surrounding weather, terrain and soil conditions
- Meet international highway design standards
- Minimize future maintenance requirements
- Minimize adverse community and environmental impacts.

Structural, drainage and pavement design standards shall follow the American Association of State Highway and Transport Design Guides with required modifications where necessary to suit local conditions.

The Project aims at providing a dual carriageway by constructing two additional lanes on the east side (right-hand side towards Peshawar) land in parallel to the existing road. The design standards applied for the Project road are as follows:

- Roadway:
  - Standards for Roads in Pakistan, NHA, February 1992
  - Typical and General Drawings (Highways), NHA, April 2004
  - Policy on Geometric Design of Highways and Streets, AASHTO, 2001
  - Highway Drainage Guidelines, AASHTO, 1992
- Pavement:
  - AASHTO Guide for Design of Pavement Structures, 1993
- Bridges:
  - Standardization of Bridge Superstructures, NHA, March 2005
  - Standard Specifications for Highway Bridges, AASHTO, 17th Edition
  - Draft Seismic Design Standard, NHA, August 2006
    (Note: New PGA established after the earthquake in October 2005)
- Tunnel:
  - Technical Standards for Road Tunnels, Japan Road Association, 2001 and 2003
  - Design Manual Volume III (Tunnel), Japan Highway Public Corporation, 1997
  - Standards of Tunnels in Mountainous Terrain, Japan Society of Civil Engineers, 1996
- Materials:
  - Standard Construction Specifications, NHA, 1998

In addition to the above, the design standards used for the 1st Kohat Tunnel and Access Roads were referred to since the 2nd Tunnel and Access Roads are planned to be constructed in parallel to the 1st Kohat and Access Roads.

8.2 Classification of the Project Road

The Kohat Tunnel and Access Roads are a part of the National Highway N-55 (Indus Highway) administrated by NHA. The Project road (length: 30 km) starts at the Kohat Toi Intersection (south of Kohat Town), crosses over the Kohat Pindi (N-80) IC at Sta.9+646, the Kohat Link Road IC at Sta.15+575, the Sanda Basta (N.W.F. Road) IC at Sta.19+088 and ends at Dara Adam Khel Intersection, 30km south of Peshawar.

8.3 Highway Design Standards

8.3.1 Design Speed and Design Vehicles

The geometric design standards applied for the Project road are NHA Standards and AASHTO Geometric Design Guide (2001) according to which the design speed and design vehicle govern the major elements of road geometry. The geometric design standards of the Project road are also governed by the existing 1st Kohat Tunnel and Access Roads as it is constructed at 13.3 m and 10.8 m away and in parallel to the existing road.

The design vehicle applicable for the Project road is truck-trailers, Vehicle Type WB-12 (H:4.1m, W:2.4m, L:13.9m), WB-15 (H:4.1m, W:2.6m, L:16.8m) and WB-19 (H:4.1m, W:2.6m, L:20.9m), referring to AASHTO specifications.

The design speeds applied for the geometric design are as shown in Table 8.3.1. The same design speeds (90 km/h for the tunnel south section and 80 km/h for the tunnel north section) were used for the 2nd Kohat Tunnel and Access Roads as it constitutes a widened part of the existing road.

Table 8.3.1 Design Speed of Throughway

Road Section	Terrain	Area	Design Speed (km/h)*		Remarks
			1st Kohat Road	**2nd Kohat Road**	NHA 1992**
Kohat Toi - Changai Algada (Sta.18+000)	Flat to Rolling	Sub-urban to Rural	90	**90**	100
Changai Algada (Sta.18+000) - N.W.F.Road IC. (Sta.19+000)	Mountain	Rural	80	**80**	80
N.W.F.Road IC. - Dara Adam Khel (End of Project)	Flat to Rolling	Rural	80	**80**	100
Kohat Tunnel			60	**60**	-

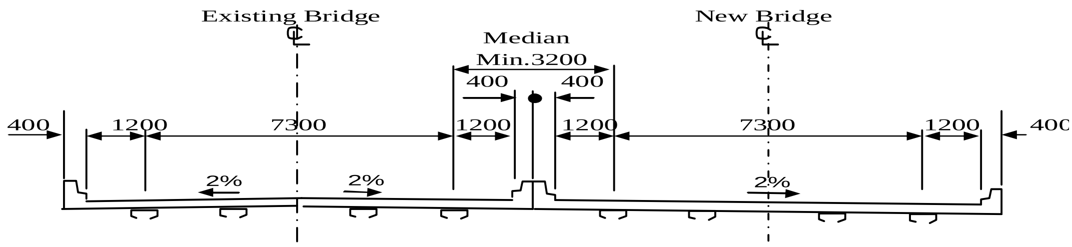
Notes: * The design speed is based on A Policy on Geometric Design of Highways and Streets, 2001, AASHTO
** Standards for Roads in Pakistan, NHA, 1992

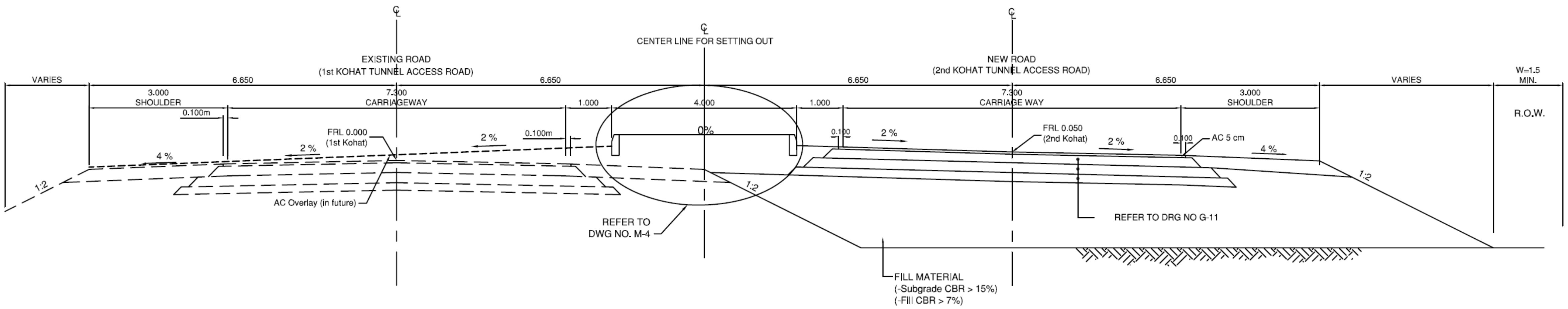
8.3.2 Geometric Design Standards

The geometric design standards applied for the Project road are as summarised in Table 8.3.2.

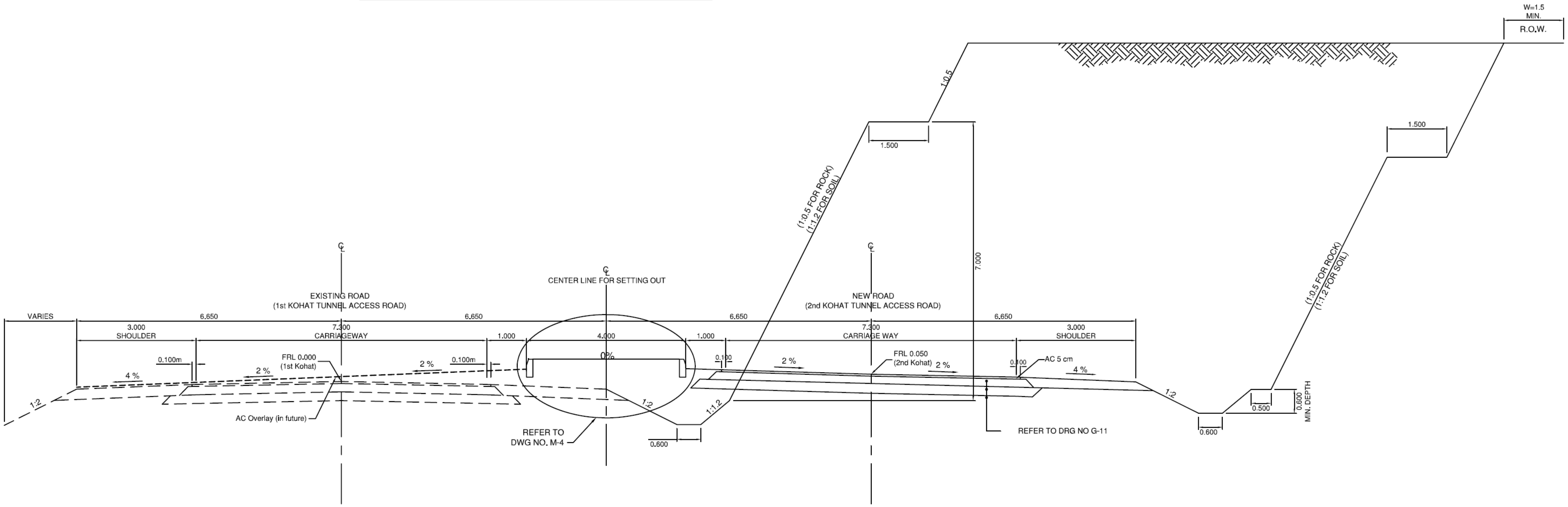
(1) Cross Section Elements

Figures 8.3.1 and 8.3.2 show the typical cross sections for the north and south sections respectively. The two new lanes of the tunnel south section are located at 13.3 m from the existing road centreline providing a 6m-wide median as originally planned in the 1st Kohat Tunnel and Access Road Project, and those of the north section are located at 10.8 m from the existing road, providing a 3.5m-wide median. The original median was 3.0 m wide for the north section but it was reviewed in this FS to provide a minimum median width of 3.2 m at four bridges (see the following illustration) and to avoid the centreline deviation in a short distance.





TYPICAL CROSS SECTION IN EMBANKMENT (SOUTH)



TYPICAL CROSS SECTION IN EMBANKMENT AND CUT (SOUTH)

Figure 8.3.1 Typical Cross Sections for South Section

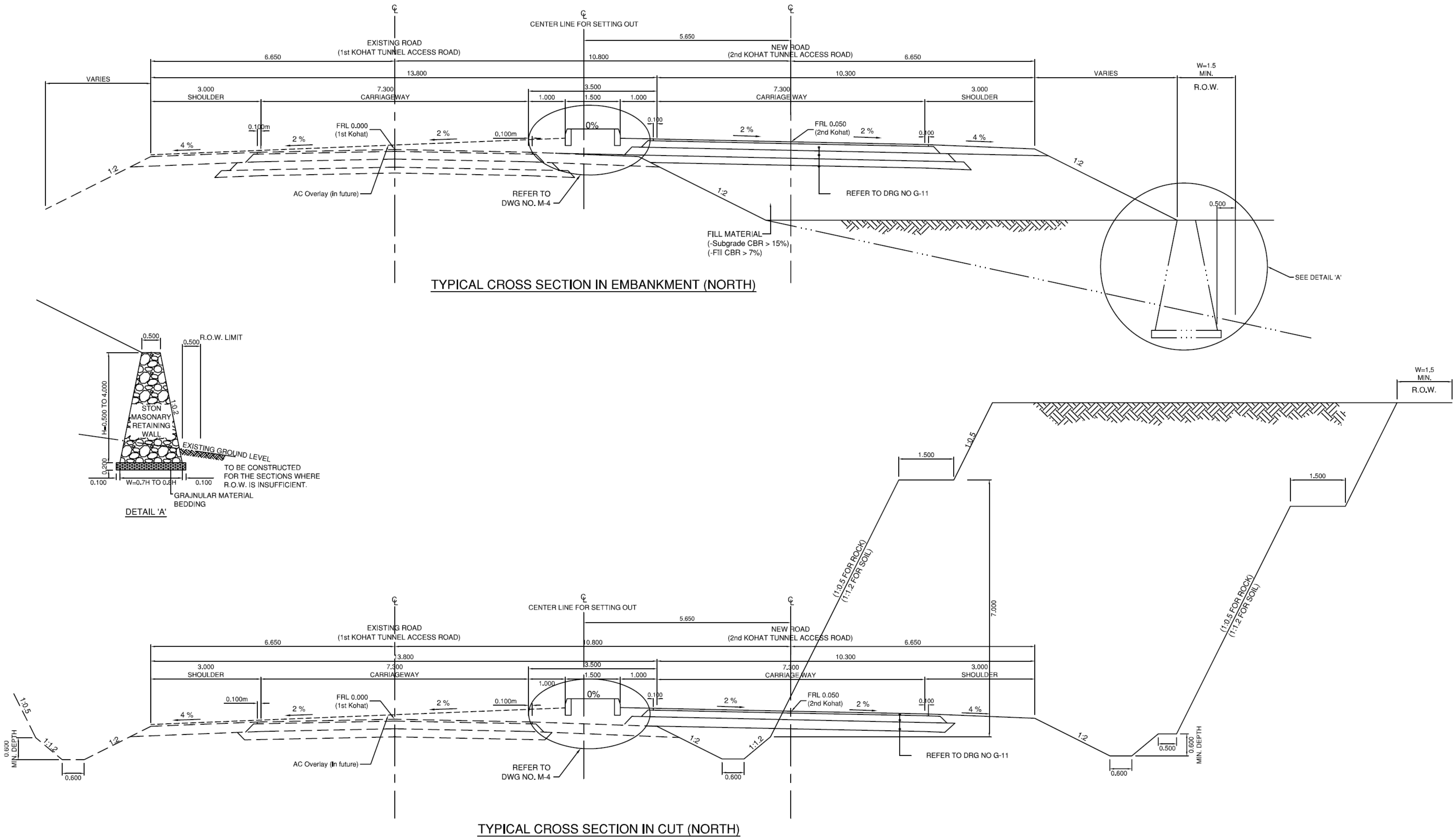


Figure 8.3.2 Typical Cross Sections for North Section

Table 8.3.2 Geometric Design Standards

Item	Unit	Design Standards			
		1st Kohat Acesss Road		**2nd Kohat Acesss Road**	
Section		South	North	South	North
Design Section	Km/h	90	80	90	80
Cross Section Elements:					
- Lane width	m	3.65	3.65	3.65	3.65
- Outer Shoulder Width	m	3.00	3.00	3.00	3.00
- Outer Shoulder Width for climbing lane	m	1.00	1.00	-	-
- Inner Shoulder Width	m	1.00	1.00	1.00	1.00
		(Future 4-lanes)			
- Median Width	m	6.00	3.00	6.00	3.50
		(Future 4-lanes)			
- Climbing Lane Width	m	3.00	-	-	-
- Crossfall of Travelled Way	%	2	2	2	2
- Crossfall of Shoulder	%	4	4	4	4
- Vertical Clearance	m	5.03	5.03	5.03	5.03
- Railway Vertical Clearance	m	6.71	6.71	6.71	6.71
- Stopping Sight Distance	m	137	120	160	130
- Passing Sight Distance	m	600	550	615	540
Horizontal Alignment:					
Circular Curve:					
- Min. Radius	m	270	220	275	210
- Min. Superelevation Runoff Length	m	50	46	115	108
		(one lane rotated)		(two lane rotated)	
- Max. Superelevation Rate	%	10	10	10	10
- Tangent Runout	m	16	15	23	22
Transition Curve:					
- Type of transition curve		-	-	Spiral Curve (Clothoid)	-
- Min. Transition Curve Length	m	-	-	50	-
- Max. Radius for Use of a Spiral Curve Transition *	m	-	-	480 (1200)	-
Vertical Alignment:					
- Max. Grade	%	7	7	4	5
Crest Curve					
- Stopping Sight Distance	m	-	-	160	130
- Passing Sight Distance	m	600	550	615	540
Sag Curve					
- Stopping Sight Distance	m	-	-	160	130

Note: * recommended max. radius for use of a transition curve if site condition allows

The carriageway width is 7.3 m (3.65 m x 2 lanes), the outer shoulder width is 3.0 m and the inner shoulder width is 1.0 m. The standard cross fall of the travelway is 2.0% and that for shoulders is 4.0%.

Slopes of cuts are 1 (V) : 1.2 (H) for common soil, 1 (V) : 0.7 (H) for semi-rock, and 1 (V) : 0.5 (H) for rock. Intermediate 1.5m-wide berms will be provided for the cuts higher than 7m. Embankment slopes are 1 (V): 2 (H) without intermediate berms irrespective of heights. Retaining walls will be constructed at the fill and cut sections where the existing Right of Way (ROW) is insufficient in the tunnel north section to avoid additional land acquisition.

The maximum superelevation of 10%, which is same as the 1st Kohat Tunnel and Access Roads Design, is used for the Project road. The superelevation (SE) and SE run-off are as shown in Table 8.3.3 (AASHTO Geometric Design Standards).

Table 8.3.3 Superelevation and Minimum Length of SE Runoff for e_{max} = 10%

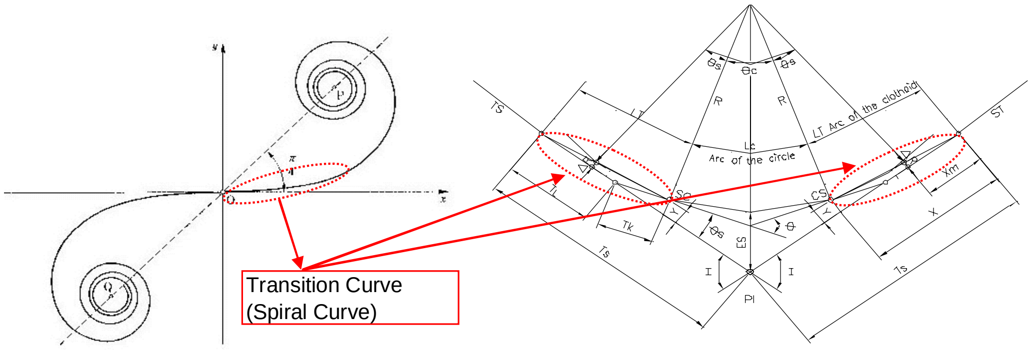
<table>
<tr><th rowspan="3">R
(m)</th><th colspan="3">V = 80 Km/h</th><th colspan="3">V = 90 Km/h</th></tr>
<tr><th rowspan="2">e
(%)</th><th colspan="2">L (m)</th><th rowspan="2">e
(%)</th><th colspan="2">L (m)</th></tr>
<tr><th>2-lanes</th><th>4-lanes</th><th>2-lanes</th><th>4-lanes</th></tr>
<tr><td>7000</td><td>NC</td><td>0</td><td>0</td><td>NC</td><td>0</td><td>0</td></tr>
<tr><td>5000</td><td>NC</td><td>0</td><td>0</td><td>NC</td><td>0</td><td>0</td></tr>
<tr><td>3000</td><td>NC</td><td>0</td><td>0</td><td>NC</td><td>0</td><td>0</td></tr>
<tr><td>2500</td><td>NC</td><td>0</td><td>0</td><td>RC</td><td>15</td><td>23</td></tr>
<tr><td>2000</td><td>RC</td><td>14</td><td>22</td><td>2.2</td><td>17</td><td>25</td></tr>
<tr><td>1500</td><td>2.4</td><td>17</td><td>26</td><td>2.9</td><td>22</td><td>33</td></tr>
<tr><td>1400</td><td>2.6</td><td>19</td><td>28</td><td>3.1</td><td>24</td><td>36</td></tr>
<tr><td>1300</td><td>2.8</td><td>20</td><td>30</td><td>3.3</td><td>25</td><td>38</td></tr>
<tr><td>1200</td><td>3.0</td><td>22</td><td>32</td><td>3.6</td><td>28</td><td>41</td></tr>
<tr><td>1000</td><td>3.5</td><td>25</td><td>38</td><td>4.2</td><td>32</td><td>48</td></tr>
<tr><td>900</td><td>3.9</td><td>28</td><td>42</td><td>4.6</td><td>35</td><td>53</td></tr>
<tr><td>800</td><td>4.3</td><td>31</td><td>46</td><td>5.1</td><td>39</td><td>59</td></tr>
<tr><td>700</td><td>4.8</td><td>35</td><td>52</td><td>5.8</td><td>44</td><td>67</td></tr>
<tr><td>600</td><td>5.5</td><td>40</td><td>59</td><td>6.5</td><td>50</td><td>75</td></tr>
<tr><td>500</td><td>6.4</td><td>48</td><td>69</td><td>7.6</td><td>58</td><td>87</td></tr>
<tr><td>400</td><td>7.5</td><td>54</td><td>81</td><td>8.8</td><td>67</td><td>101</td></tr>
<tr><td>300</td><td>9.0</td><td>65</td><td>97</td><td>9.9</td><td>76</td><td>114</td></tr>
<tr><td>250</td><td>9.7</td><td>70</td><td>105</td><td colspan="3">R_{min} = 275 m</td></tr>
<tr><td></td><td colspan="3">R_{min} = 210 m</td><td colspan="3"></td></tr>
</table>

Source: A Policy on Geometric Design of Highways and Streets, AASHTO, 2001

where,
R: Radius of Curve
Vd: Assumed Design Speed
e: Rate of Superelevation
L: Minimum Length of Runoff (not including tangent run out)
NC: Normal Crown Section
RC: Remove adverse crown, superelevation at normal crown slope

(2) Horizontal Alignment

The NHA design standards do not specify the necessity of transition curves. The road geometry used for the 1st Kohat Tunnel and Access Roads was a combination of straight lines and simple curves. However, it is common to insert a transition curve between the straight section and the circular curve taking safety and convenience of driving into consideration. Clothoid or spiral curves (see the following illustration), which show a similar trail of vehicle movement after turning handle, are used as transition curves by most of the world road agencies.



Calculation and site set out of spiral curves were very complicated before personal computers are available at an acceptable price. However, it is easy now as computer programs give all necessary data for drawings preparation and site set out.

Providing safe and user friendly road facilities is the mission statement of road administrators and agencies, not only in Pakistan but also throughout the world. The Study Team designed the 2nd Kohat Tunnel by both methods: one without transition curves and the other with transition curves and compared them for application.

The minimum curve radius applied for the tunnel south section is 300m and that for the north section is 260m and both meet the geometric design standards shown in Table 8.3.2 except at the junctions. The radius of curves in the tunnel south section is relatively larger because this section is located in a flat/rolling terrain. However, the curves in the tunnel north section have a smaller radius with short straight sections due to the mountainous terrain at this site.

Travelway widening is needed for certain horizontal curves because the design vehicles occupy a greater width on curves or drivers may not run on the lane centre. The design widening values are shown in Table 8.3.4.

Table 8.3.4 Travelway Widening for Two-Lane Highways (One-way or Two-way)

R(m)	Design Vehicle WB-12 (Semi-trailer)					Design Vehicle WB-15 (Semi-trailer)					Design Vehicle WB-19 (Semi-trailer)				
	Design Speed (Km/h)					Design Speed (Km/h)					Design Speed (Km/h)				
	50	60	70	80	90	50	60	70	80	90	50	60	70	80	90
2000	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-
1500	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-
1000	-	-	-	-	-	-	-	-	0.1	0.1	0.1	0.1	0.1	0.2	0.2
900	-	-	-	-	-	-	-	0.1	0.1	0.1	0.1	0.1	0.2	0.2	0.2
800	-	-	-	-	-	-	0.1	0.1	0.1	0.1	0.1	0.2	0.2	0.2	0.2
700	-	-	-	-	-	0.1	0.1	0.1	0.2	0.2	0.2	0.2	0.2	0.3	0.3
600	-	-	-	-	-	0.1	0.2	0.2	0.2	0.3	0.2	0.3	0.3	0.3	0.4
500	-	-	-	-	-	0.2	0.2	0.3	0.3	0.4	0.3	0.3	0.4	0.4	0.5
400	-	-	-	-	-	0.3	0.3	0.4	0.4	0.5	0.5	0.5	0.6	0.6	0.7
300	-	-	-	0.1	0.2	0.4	0.5	0.5	0.6	0.7	0.6	0.7	0.7	0.8	0.9
250	0.1	0.1	0.2	0.2	0.3	0.5	0.6	0.7	0.7	0.8	0.7	0.8	0.9	0.9	1.0
200	0.1	0.2	0.3	0.3		0.7	0.8	0.9	0.9		0.8	1.1	1.2	1.2	

Notes: 1. The above are adjusted values for the roadway width from 7.20 m in USA to 7.30 m in Pakistan.
2. Values less than 0.6m may be disregarded.
Source: A Policy on Geometric Design of Highways and Streets, AASHTO, 2001

However, as not so many truck-trailers of Type WB-15 (or WB-12 and WB-19) use the Project road and they run at a speed lower than the design speed, and as the widening cost is high, travelway widening should be considered carefully. As travelway widening was not applied for the 1st Kohat Tunnel and Access Roads, the same policy of design without widening should be adopted for the 2nd Kohat Tunnel and Access Roads.

(3) Vertical Alignment

Two new lanes will be provided for the south-bound traffic from Peshawar to Kohat/Bannu. The road climbs up from Dara Adam Khel to the tunnel north portal and then descends to Kohat Toi. The maximum grade is limited to less than 4.8% for the access roads. The grade of the new tunnel, which is to be used for the down-grade traffic, is 2.4% compared to 2.2% of the existing tunnel. Vertical curve alignment is designed to meet the stopping site distance and taking visual comfort of driving into consideration.

The 1st Kohat Tunnel and Access Roads have a climbing lane at Sta.17+800 - Sta.19+700 (L=2.3 km) where the grade of vertical alignment is 3.9%. The 2nd Kohat Tunnel and Access Roads will not be provided with a climbing lane as this section will be used for the down-grade traffic only.

8.3.3 Drainage Facilities

Drainage structures for the Project road consist of river bridges/box culverts, roadway cross drainages (pipe culverts), road side drainages and tunnel drainages. The flood with a return period of 50 years is used for bridge design and 25 years for culvert design.

Cross drainage structures (bridges and culverts) are at the same location and have the same capacity as those of the 1st Kohat Tunnel and Access Roads, in principle, because those structures are positioned either downstream or upstream of the existing structures. As the Project road is to provide a dual carriageway system, median drainage is necessary for the in-curve sections.

8.3.4 Pavement Design Standards

Pavement of the Project road is designed according to the standards set forth in the AASHTO Guide for Design of Pavement Structures 1993. Asphalt concrete pavement will be used for the Project road except for the toll plaza and tunnel sections where Portland cement concrete pavement (PCC) is used. Flexible pavement (AC pavement) is adopted for the design period of 10 years and rigid pavement (PCC pavement) for 20 years.

Pavement structures are designed taking into account high ambient temperature in the dry season, relatively low temperatures in the cold season, and high axle loads. Structural design of pavements is performed based on the following surveys and analysis:

- Soil condition along the road alignment
- Subgrade bearing capacity analysis
- Traffic counts, future traffic and CESA estimate
- Drainage characteristics of the subgrade soils
- Materials availability analysis.

Work experience and performance of the pavement during and after the 1st Kohat Tunnel and Access Roads construction were also incorporated in the design. Appropriate materials (see Tables 8.3.5 and 8.3.6) conforming to the Standard Construction Specifications of NHA are used for the pavement design. AC pavement is composed of 3 layers: AC Base Class A, AC Base Class B and AC wearing course Class B.

Table 8.3.5 Asphalt Concrete Materials

Item	AC Wearing Course		AC Base Course	
	Class A	Class B	Class A	Class B
Asphalt Concrete				
Asphalt Penetration Grade	40-50, 60- 70 or 80-100		40-50, 60- 70 or 80-100	
AC Content	3.5% (Min.)		3.0% (Min.)	
Stability	1000 kg (Min)		1000 kg (Min)	
Flow, 0.25 mm	8-14		8-14	
Air Void	4% - 7%		4% - 8%	
Loss in Stability	20% (Max.)		25% (Max.)	
Aggregates				
Max. Particle Size	25 mm	20 mm	50 mm	38 mm
LA Abrasion	30% (Max.)		40% (Max.)	
Loss by SS Soundness	12% (Max.)		12% (Max.)	
Flat particles	10% (Max)		15% (Max)	
Course Aggregate (>4.75 mm)	Crushed rock or crushed gravel (crushed particle =100%)		Crushed rock, crushed gravel or crushed boulder (crushed particle > 95%)	
Fine Aggregate	100% crushed rock or crushed boulder		100% crushed rock or crushed boulder	

Source: Data extracted from Standard Construction Specifications of NHA, 1998

Table 8.3.6 Base and Subbase Materials

Item	Granular Subbase		Aggregate Base	
	Grading A	Grading B	Grading A	Grading B
Max. Particles	60 mm	50 mm	50 mm	50 mm
Uniformity (D60/D10)	3 (Min.)		4 (Min.)	
CBR	50% (min) at 98% max. density		80% (min) at max. density	
LA Abrasion	50% (Max.)		40% (Max.)	
Loss by SS Soundness	-		12% (Max.)	
Fraction < 0.075 mm	LL< 25%, PI < 6		LL< 25%, PI < 6	
Materials	Natural or processed aggregate		Crushed aggregate (crushed particle > 90%)	

Source: Data extracted from Standard Construction Specifications of NHA, 1998

8.3.5 Other Road Facilities

Other road facilities including toll plaza, intersections, right/left turn lanes, road safety facilities, traffic control facilities, etc. are designed in accordance with the NHA standards and those used for the 1st Kohat Tunnel and Access Road Project.

8.4 Bridge and Culvert Design Standards

8.4.1 Design Standards and Loading

(1) Design Standards

The "Standardization of Bridge Superstructures, NHA, 2005", "West Pakistan Code of Practice for Highway Bridges (WPCHB)" and the bridge design for the 1st Kohat Tunnel and Access Roads are referred to for bridge design.

(2) Loads

a) Dead Load

The material densities to be sued for defining dead loads for the design are chosen in accordance with the WPCHB standards (Table 8.4.1).

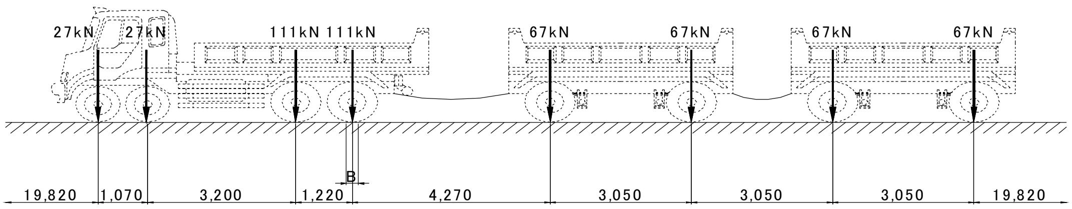
Table 8.4.1 Material Densities

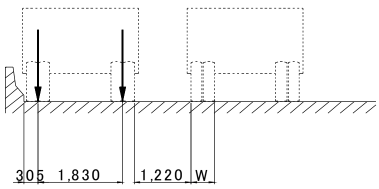
Material	Density (lbs/cu-foot)	Density (kN/cu-m)
Steel	490.0	77.0
Reinforcement Concrete	150.0	24.0
Normal Concrete	140.0	22.0
Asphalt Concrete	140.0	22.0
Macadam	140.0	22.0
Compacted Sand	120.0	19.0
Loose Sand	90.0	15.0
Water	62.5	10.0

Source: WPCHB

b) Live Load

The live load of "Class A loading (Figure 8.4.1)" specified in WPCHB, Article 2.4 is used for the design. WPCHB specifies the highway live loads on roadway bridges and incidental structures. A standard truck-trailer consists of a four-axle truck and two two-axle trailers. For the bridge deck slab, punching shear strength shall be checked in design, where the wheel load is 95 kN, and the contact area is 0.25 x 0.50 m² of tire.





Class of Loading	Axle Load	Ground Contact Area	
		B	W
	kN	mm	mm
A	111	254	508
	67	203	381
	27	152	203

Source: WPCHB

Figure 8.4.1 Class A Loading

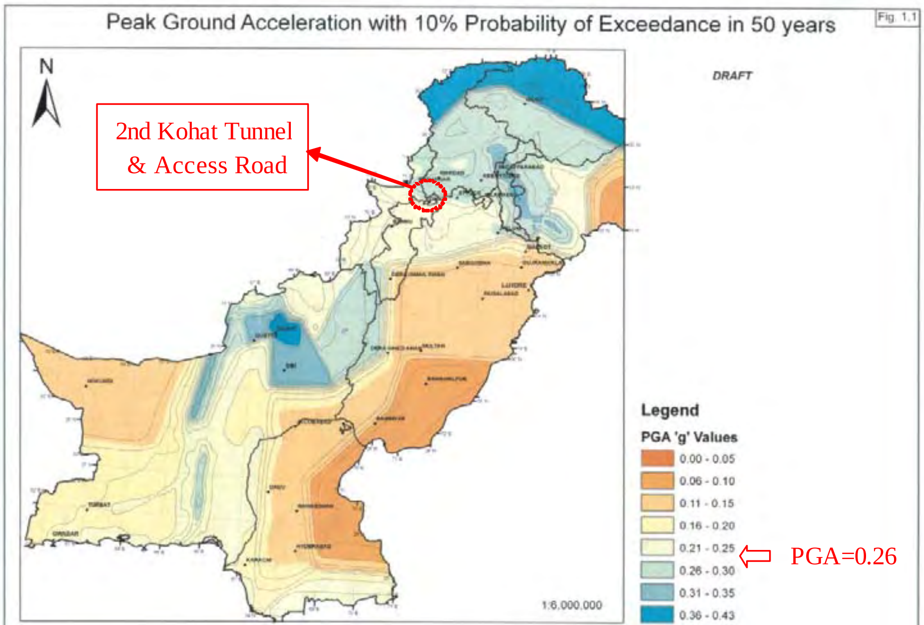
c) Other Loads

As per WPCHB and AASHTO, other loads to be considered for the bride design are the following:

- Impact effect of live loads
- Wind force
- Earth pressure
- Overall temperature
- Water flow
- Seismic force

d) Seismic Force

NHA has reviewed the Peak Ground Acceleration (seismic force) and seismic zone after the earthquake of October 8, 2005 and the review is currently at the final draft stage. The new PGA value will be used for the bridge design for the 2nd Kohat Tunnel and Access Roads.



Source: NHA (Jul 2006)

Figure 8.4.2 New Seismic Force (PGA) for Project Area

8.4.2 Bridge Planning

There are 11 bridges on the existing Kohat Tunnel and Access Roads: Seven in the south section and four in the north section. Of those, the Bridge No.4 at Sta.19+200 was already constructed with a dual carriageway (4 lanes). As others bridges are 2-lane structures, 10 new bridges are to be constructed in parallel to the existing bridges. The length, type and spans of the new bridges are same as those of the existing bridges, except the Bridge No.5 located after the north portal. A special consideration has been made for the Bridge No.5 to ensure smooth river flow. The superstructure of short span bridges consists of RC girders and that for longer span bridges consist of PC girders. The PGA value applied previously for the Project area was 0.05-0.07g (Zone III) and the new PGA is 0.26g. Therefore, more foundation piles, larger foundation and piers are required for the 2nd Kohat Tunnel and Access Roads as compared to the 1st Kohat Tunnel and Access Roads.

8.4.3 Culvert Planning

Over 80 box culverts were constructed under the 1st Kohat Tunnel and Access Road Project. Most of them require extension for the dual carriageway construction under the 2nd Kohat Tunnel and Access Roads Project. Standard structures of NHA will be used.

8.5 Tunnel Design Standards

8.5.1 Design Standards

(1) Rock Classification for Support System

There are five representative design standards for mountainous tunnel linings (support system) in the world. Those standards are almost the same. Standards of rock classification, which is a basis for tunnel design, specified in these standards are summarized in Table 8.5.1

Table 8.5.1 Standards of Rock Classification

Country	Austria,	Austria,	Switzerlan	France	South Africa	Norway	Japan
Name of Standard	ONORM B2203	Rabcewicz-Pacher	SIA 198	AFTES	Rock Mass Rating (RMR)	Norwegian Tunneling Method (NTM)	Standards for Road Tunnels
Number of Rock	7	6	6	10	5	9	7
Method of Rock Classification	Qualitative Observation of Rock Mass Behavior			Rating of Rock Mass by Protojakonof coefficient f value	Rating of Rock Mass by RMR	Rating of Rock Mass by Q-Value	Rock Mass Classfication by Parameters
Characteristics of Analysis	Qualitative			Quantitative			
Parameters/Items of Rating/Classification	None			f=tanϕ+c/σc (Soil) c: Cohesion ϕ: Angle of internal friction σc: Unconfined Compression Strength f=σk/100 (Rock) σk: Unconfined Compression Strength	-Strength (Point load or Unconfined Compression) -RQD -Spacing of Discontinuities -Condition of Discontinuities -Ground Water Condition -Strike and Dip Orientations	Q=(RQD/Jn)x(Jr/Ja)x(Jw/SRF) RQD Jn:Joint Set Number Jr:Joint Roughness Number Ja:Joint Alterration Number Jw:Joint Water Reduction SRF:Stress Reduction Factor	Seismic Wave Velocity RQD Distance between Cracks GN=qu/γh qu:Uncofined Compression strength γ:Unit Weight of Rock Mass h:Overburden Depth
Application	Medium to hard rock	Medium to hard rock	Medium to hard rock	Soil to hard rock	Medium to hard rock	Medium to hard rock	Highly weathered rock to hard rock
Remarks	The definition of the rock classification depends on the behavior of rock mass during construction stage. Therefore, it is very difficult to apply at planning and design stage. During construction stage, the suitability of applied rock classification and its support system is checked and confirmed by convergence measurement, etc.			This method is widely applied from soil to hard rock. However, the parameter for hard rock is only unconfined compression strength. Therefore, conditons related cracks in rock mass is not considered well.	RMR is rated by many parameters related to rock mass strength, condition of cracks and water, etc. Therefore, the actual rock mass condition could be defined. In order to use at planning and design stage, many boring data will be required. This method is most widely used in the world.	Q-value is rated by many parameters related to rock mass strength, condition of cracks and water, etc. Therefore, the actual rock mass condition could be defined. However, definition of the parameter is very complicated and experienced geologist is required for evaluation. In order to use at planning and design stage, many boring data will be required.	Seismic wave velocity test is carried out to compensate the geological boring data. Usually this standard is used at planning and design stage. During construction, the evaluation of rock mass is supplemented by face observation which is similar to RMR. Measurement such as convergence, stress,etc is also used to confirm the suitability of installed support system.

The standards of Germany/Austria and Switzerland classify tunnel support types according to rock classification encountered during tunnel excavation. Tunnel support types in other standards are determined by rock classified in geological investigation prior to the construction. "f–value" and "Q-value" are used for rock classification in French and Norwegian standards respectively. Seismic velocity is used in the Japanese standards.

(2) Design Standard

The 2nd Kohat Tunnel becomes the third road tunnel in Pakistan following the 1st Kohat tunnel designed and constructed based on the Japanese standards and the Lowari Tunnel (railway/road tunnel) currently under construction.

As the 2nd Kohat Tunnel is planned to provide an additional dual carriageway facility, the Japanese standards used for the 1st Kohat Tunnel will be applied for the design. The main standards are the following.

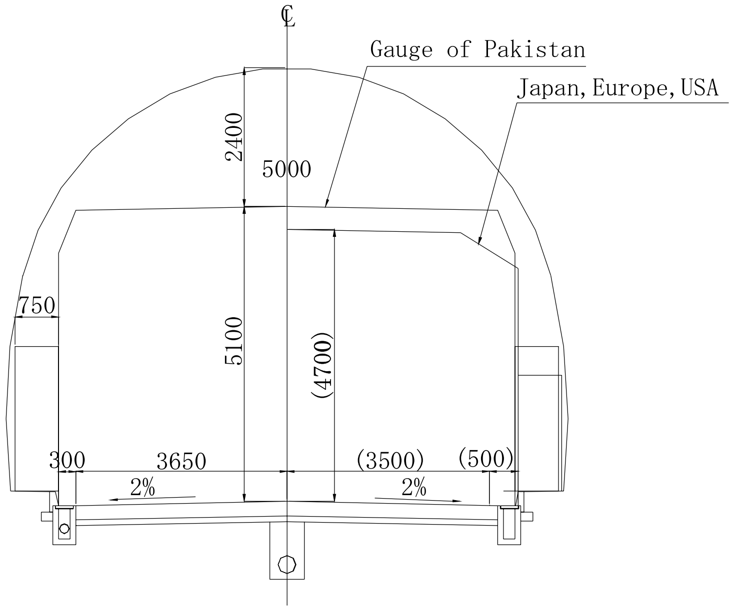
- Technical Standards for Road Tunnels (Structures), Japan Road Association, 2003
- Technical Standards for Road Tunnels (Ventilation Facilities), Japan Road Association, 2001
- Design Guide for Lighting System of Tunnel, Express Highway Research Foundation of

Japan, 1990

- Standards of Tunnel in Mountainous Terrain, Japan Society of Civil Engineers, 1996
- Design Manual Volume III (Tunnel), Japan Highway Public Corporation, 1997
- Design Manual Volume III (Electricity/Machinery Facilities), Japan Highway Public Corporation, 1990

8.5.2 Standard Cross Section of Tunnel

The standard clearance limit is based on the gauge of the 1st Kohat Tunnel. The road width is 7.9 m: two 3.65 m lanes and 0.3 m shoulder on each side. The clearance height limit is 5.1 m, which is higher than the Japanese, USA and European standards, to meet vehicles used in Pakistan. Figure 8.5.1 shows the standard cross section for the 1st/2nd Kohat Tunnel and a comparison with the world standards.



Note: Dimensions in parentheses are those defined by the Japanese standards.

Figure 8.5.1 Standard Cross Section for the 1st/2nd Kohat Tunnel and Comparison with Japanese, European and USA Standards

The cross slope of roadway (camber) is 2%. The inspection gallery width is 0.75 m. A space of 1.4 m x 5 m on tunnel ceiling is used for installation of jet fans for ventilation. Emergency parking bays (a space of 3 m x 30 m) are provided at every 750 m.

Standard Support Patterns are shown in Table 8.5.2.

Table 8.5.2 Standard Support Patterns

<table>
<tr><th rowspan="3">Grade of Ground</th><th rowspan="3">Excavation method</th><th rowspan="3">Standard round length (Upper half) (m)</th><th colspan="3">Rock bolt</th><th colspan="3">Steel arch supporting</th><th rowspan="3">Shotc-rete thick-ness (cm)</th><th colspan="2">Lining Thickness (cm)</th><th colspan="3">Over cut designed to allow ground deformation (cm)</th></tr>
<tr><th rowspan="2">Length (m)</th><th colspan="2">Installation pitch</th><th rowspan="2">Upper half</th><th rowspan="2">Lower Half</th><th rowspan="2">Standard pitch (m)</th><th rowspan="2">Arch and side wall</th><th rowspan="2">Invert</th><th rowspan="2">Upper half</th><th rowspan="2">Lower half</th><th rowspan="2">Invert</th></tr>
<tr><th>Circumferential (m)</th><th>Longitudinal (m)</th></tr>
<tr><td>B</td><td>Full face method with auxiliary bench and upper half method</td><td>2.0</td><td>3.0</td><td>1.5 (upper half only)</td><td>2.0</td><td>None</td><td>None</td><td>-</td><td>5</td><td>30</td><td>0</td><td>0</td><td>0</td><td>0</td></tr>
<tr><td>CI</td><td>Full face method with auxiliary bench and upper half method</td><td>1.5</td><td>3.0</td><td>1.5</td><td>1.5</td><td>None</td><td>None</td><td>-</td><td>10</td><td>30</td><td>0</td><td>0</td><td>0</td><td>0</td></tr>
<tr><td rowspan="2">CII</td><td>Full face method with auxiliary bench</td><td rowspan="2">1.2</td><td rowspan="2">3.0</td><td rowspan="2">1.5</td><td rowspan="2">1.2</td><td rowspan="2">H-125 or U-21</td><td>None in Principle</td><td rowspan="2">1.2</td><td rowspan="2">10</td><td rowspan="2">30</td><td rowspan="2">0</td><td rowspan="2">0</td><td rowspan="2">0</td><td rowspan="2">0</td></tr>
<tr><td>Upper half method</td><td>None</td></tr>
<tr><td>DI</td><td>Full face method with auxiliary bench and upper half method</td><td>1.0</td><td>4.0</td><td>1.2</td><td>1.0</td><td>H-125 or U-21</td><td>H-125 or U-21</td><td>1.0</td><td>15</td><td>30</td><td>45</td><td>0</td><td>0</td><td>0</td></tr>
<tr><td rowspan="2">DII</td><td>Full face method with auxiliary bench</td><td rowspan="2">1.0 or less</td><td rowspan="2">4.0</td><td rowspan="2">1.2</td><td rowspan="2">1.0 or less</td><td rowspan="2">H-125 or U-29</td><td rowspan="2">H-125 or U-21</td><td rowspan="2">1.0 or less</td><td rowspan="2">20</td><td rowspan="2">30</td><td rowspan="2">50</td><td>10</td><td>10</td><td>0</td></tr>
<tr><td>Upper half method</td><td>10</td><td>0</td><td>0</td></tr>
</table>

8.5.3 Ventilation system

The air inside a tunnel is polluted by exhaust gas from the traffic since fresh air cannot be supplied naturally. The natural wind in tunnel is developed merely by the pressure difference between the two portals and also because of movement of traffic. This kind of phenomena to provide natural ventilation is possible only in short tunnels. But in long tunnels, a mechanical ventilation system producing continuous air supply is required for proper ventilation.

The report of PIARC (Permanent International Association of Road Congress, XIV-XVII) recommends that carbon monoxide (CO) should be less than 150 ppm and smoke measured for 100m visibility is 50%. However, the above figures are applicable where engines of vehicles are well maintained.

Table 8.5.3 Limit for Exhaust Gas from PIARC (for V = 60 km/h)

<table>
<tr><th colspan="9">Trucks, buses with diesel motors (m＞3.5t)</th></tr>
<tr><th rowspan="4">Emission law</th><th rowspan="4">Control</th><th colspan="4">q^{OT} (V=60km/h)</th><th colspan="3" rowspan="3">$q^{O\ NOX}$ (V=60km/h)</th></tr>
<tr><th colspan="4">(m2/h,veh)</th></tr>
<tr><th colspan="4">Truck weight (t)</th></tr>
<tr><th>5</th><th>10</th><th>20</th><th>40</th><th>5</th><th>20</th><th>40</th></tr>
<tr><td>No Low</td><td>No</td><td>80-130</td><td>160-250</td><td>300-400</td><td>400-600</td><td>500</td><td>1400</td><td>1900</td></tr>
<tr><td>EEC R49+24</td><td>No</td><td>80</td><td>160</td><td>240</td><td>280</td><td>500</td><td>1400</td><td>1900</td></tr>
<tr><td>EEC R49+24</td><td>Yes</td><td>65</td><td>130</td><td>240</td><td>240</td><td>470</td><td>1300</td><td>1800</td></tr>
<tr><td>EEC 88/77</td><td>Yes</td><td>50</td><td>100</td><td>160</td><td>200</td><td>360</td><td>1000</td><td>1400</td></tr>
<tr><td>US Transient 88</td><td>Yes</td><td>50</td><td>100</td><td>160</td><td>200</td><td>330</td><td>900</td><td>1200</td></tr>
<tr><td>US Transient 91</td><td>Yes</td><td>30</td><td>60</td><td>100</td><td>140</td><td>270</td><td>750</td><td>1000</td></tr>
<tr><td>US Transient 94</td><td>Yes</td><td>20</td><td>40</td><td>70</td><td>110</td><td>220</td><td>600</td><td>800</td></tr>
</table>

Emission law	Control	$q^{O\ CO}$ (m^3/h,pc)
No Low	No	1-1.5
EEC R 15/04	No	0.70
EEC R 15/04	Yes	0.50
EEC 89/458	Yes	0.16
FTP75	Yes	0.12
diesel engine		0.08

Considering the old and poorly maintained vehicles being used in Pakistan and the design speed 60 km/hr in the tunnel and following the Japanese standards, the parameters for revision are given below:

- Permissible CO: 100ppm
- Smoke transmittance measured for 100m visibility: 40%

Since at this moment there are no detailed standards for road tunnels in Pakistan and the existing 1st Kohat tunnel was designed based on Japanese Road Tunnel Standards and Guidelines, the same standards used for the 1st Kohat Tunnel are applied for the design.

8.5.4 Lighting system

Sunlight is always blocked inside the tunnel. It can enter into the tunnel only up to about 10 m from the portal. Taking this into consideration, tunnels with a total length of about 50 m may not require a lighting system. However, a proper lighting system is required for tunnels longer than 50 m. Besides, to adapt car drivers to the difference in brightness between inside and outside of tunnel, a part of the portals should be arranged adapting a lighting zone with a gradual change in light brightness.

The standards for lighting system used in Europe require a lighting system of a scale bigger than that in Japan. In this context, the Japanese design standards may be more practical to be used in this Study, therefore it is recommended to apply these standards. Figure 8.5.2 shows the Japanese standards and the above facts. At this preliminarily design stage, PIARC recommendation similar to Japanese design standards were applied for the lighting system.

	Unite	CIE	CEN	BS	Japan
L1	cd/m^2	200	120	200	58
L2	cd/m^2	80	48	-	35
L3	cd/m^2	2	2	3	2.3
D1	m	-	-	70	25
D2	m	60	70	-	65
D3	m	418	285	265	135
ΣD	m	478	355	335	225

CIE: Commission Internationale de l'Eclairage (French: International Commission on Illumination - standardization body)
CEN: Comité Européen de Normalisation (French: European Committee for Standardization)
BS: British Standards

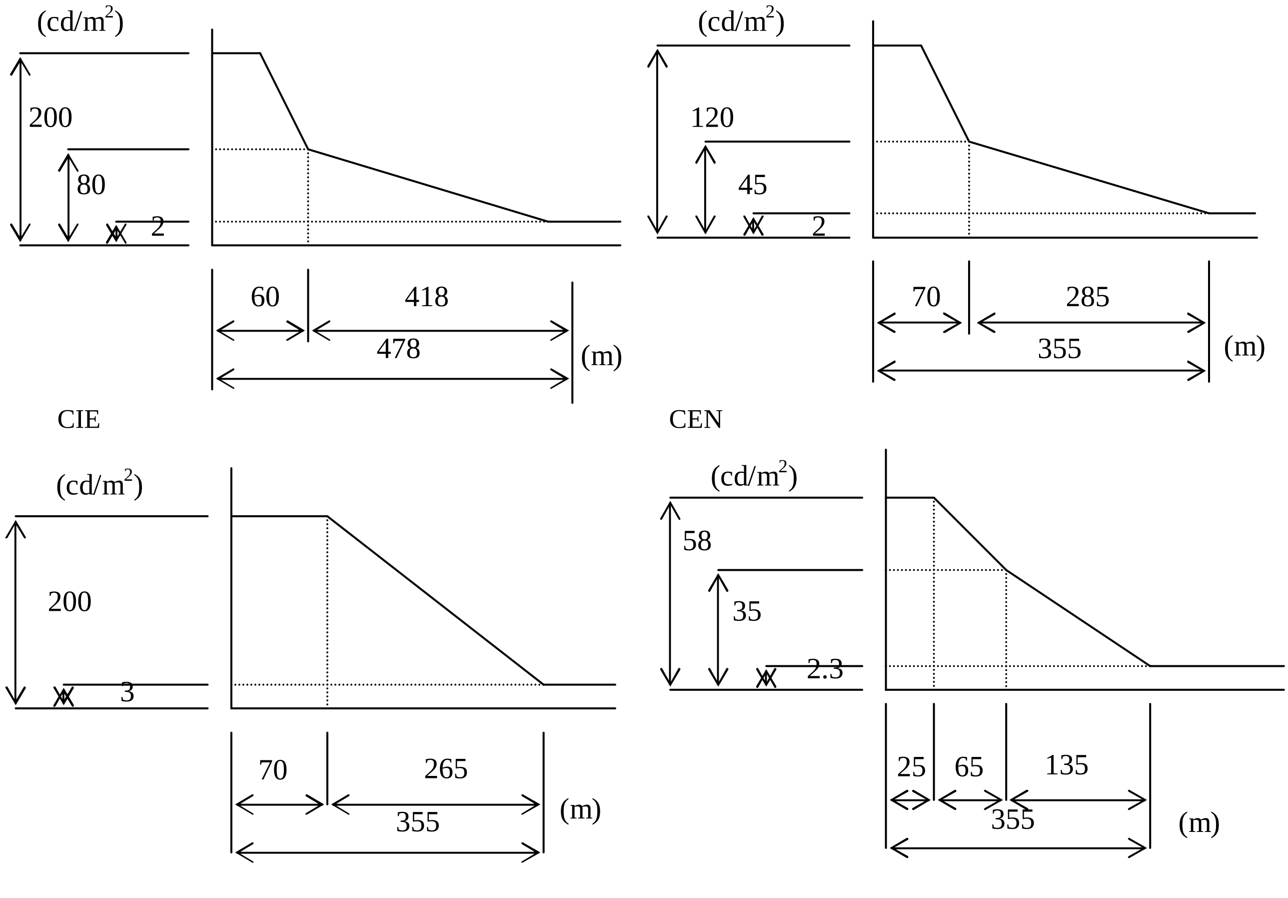


Figure 8.5.2 Comparison of Lighting Intensity (Field luminance 400cd/m^2)

8.5.5 Power Supply System

The power supply and their cable protection for high and low voltage, the entrance and the exit, the dynamotor and other equipments shall be completed in power supply system. Power lines for field equipment in tunnel shall be introduced from tunnel substations. Power distribution box shall be arranged at where field equipment is dense.

(1) Incoming Power from WAPDA

Power Supply System: 3-Phase 3-wire 50 Hz 11KV

(2) The facilities to be supplied are as follows:

Ventilation Fans, Tunnel Lighting, Tunnel Measurements Devices, Traffic Signal, Facilities in the substation and Control Room at the portal, Facilities in the Tunnel substations and etc.

(3) Wiring System for Facilities

Mainly 3 ϕ 3w 400V are used

(4) Maximum Permissible Volt Drop

Total voltage drop between the consumer's terminals and any other point in the installation must not exceed 2.5 percent of the normal voltage.

UPS shall be used for tunnel field equipment, which shall be responsible by monitoring system.

8.5.6 Emergency facilities

Tunnel emergency facilities are provided to mitigate damage in the event that fire or any other accidents occur inside of the tunnel. In case accidents occur inside tunnels, evacuation and rescue operations are very difficult because of space constraint. In this regard, proper emergency facilities must be installed. The extent of the requirements of facilities is based on the length of the tunnel and the traffic volume. Japanese standard was used for the design of emergency facilities in this Study, since the standard covers wider and more detail than other standards.

Table 8.5.4 shows the Standards for installation of emergency facilities by tunnel classification.

Emergency facilities are categorized as information and alarm equipment, fire extinguishing equipment, escape and guidance equipment, and others. Requirements of parking bays and their spaces will be studied.

Table 8.5.4 Standards for installation of Emergency Facilities by Tunnel Classification

Emergency Facilities	Tunnel Classification	AA	A	B	C	D
Communication and Alarm Equipment	Emergency Telephone	O	O	O	O	O
	Alarm Button	O	O	O	O	O
	Fire Detector	O	Δ			
	Signal and Alarm	O	O	O	O	O
Fire Fighting Equipment	Extinguisher	O	O	O		
	Fire Hydrant	O	O			
Escape and Guidance Equipment	Exit Guide Board	O	O	O		
	Smoke Discharge Equipment or Refuge Passage	O	Δ			
Other Equipment	Hydrant (Water Supply)	O	Δ			
	Radio Communication Ancillary	O	Δ			
	Radio Rebroadcast Equipment Or Loudspeaker	O	Δ			
	Sprinkler	O	Δ			
	Television	O	Δ			

Notes: O Denotes "Should be installed as the Rule" Δ Denotes "To be installed as Required "

Chapter 9. ALTERNATIVE ROUTE STUDY FOR HIGH CUT AND FILL SECTIONS

9.1 Objective of Alternative Route Study

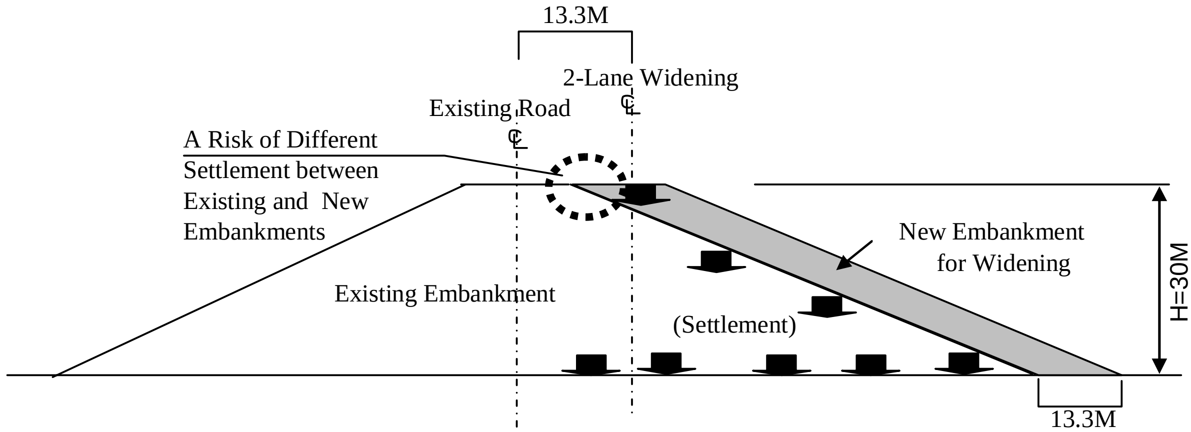
According to the original plan proposed by the design consultant for the 1st Kohat Tunnel and Access Roads Project, there are several high cuts for the 2nd Kohat Tunnel and Access Roads construction as listed in Table 9.1.1.

Table 9.1.1 List of High Cuts for the 2nd Kohat Tunnel and Access Roads Construction

No.	Station		Estimated Quantity (m^3)	Classification of Materials				Max. Cut Height (m)
				Common		Rock		
	From	To		%	(m^3)	%	(m^3)	
South Section: Kohat Toi (Start Point) - Kohat Tunnel (South Portal)								
S-1	7+325.000	7+475.000	23,800	5	1,190	95	22,610	28
S-2	14+425.000	14+625.000	4,200	60	2,520	40	1,680	10
S-3	15+250.000	15+425.000	32,100	5	1,610	95	30,490	32
S-4	**18+000.000**	**18+700.000**	**165,600**	**60**	**99,360**	**40**	**66,240**	**30**
Sub-Total:			225,700	46	104,680	54	121,020	
North Section: Kohat Tunnel (North Portal) - Dara Adam Khel (End Point)								
N-1	18+132.000	18+825.000	33,000	5	1,650	95	31,350	15
N-2	21+575.000	21+725.000	26,600	0	0	100	26,600	32
N-3	22+300.000	22+400.000	2,000	0	0	100	2,000	6
N-4	23+850.000	23+975.000	15,500	0	0	100	15,500	24
N-5	24+300.000	24+400.000	7,200	0	0	100	7,200	14
Sub-Total:			84,300	2	1,650	98	82,650	
Total:			310,000	34	106,330	66	203,670	

Except for the section S-4 (Sta.18+000 - Sta.18+700) in the above list, there are no alternative routes that are advantageous in both traffic control and cost saving aspects. Therefore, only S-4 is dealt with in this Chapter.

The roadway excavation quantity at the above section for the 2nd Kohat Tunnel and Access Roads construction was estimated at 165,600 m^3, of which 40% is rock excavation. Because of this large volume, disturbance to the traffic (7,500 veh/day) during excavation is unavoidable. The fill volume required for widening the roadway at Sta.18+700 - Sta.20+182 in the north section of the above cut section is also large, estimated at 280,000 m^3, with a height of 30m. Since it is widening work, different settlement between the existing and new embankments is a matter of concern (as illustrated below).



Under such background, a study was carried out to examine whether an economical alternative route can be set out for the section Sta.18+000 - Sta.20+182 to avoid deep cut and high embankment from the view points of reducing:

- Effects of earthworks construction on the existing traffic
- The risk of settlement between the existing and new embankments.

9.2 Alternative Route Selection

In the original road alignment for the section from Sta.18+000 to Sta.20+182 in the 1st Kohat Tunnel and Access Roads Project, it had been planned that the roadway at this section will be widened toward the east side for the construction of the 2nd access road. However the road alignment at this section was changed during its construction by modifying its grade from 6% to 4%. The new alignment plan is referred to in this chapter as the "modified original plan". The planned south portal location at 70 m from the 1st tunnel centre was retained.

Alternative routes and alignments were selected based on site reconnaissance, bird's eye view photographs (see Figure 9.2.1) from the mountain top over the Kohat Tunnel and satellite images with 1m-resolution. Alternative A is almost the same as the modified original plan except for the south portal approach. Alternative B is a new route aligned at approximately 600 m to the east of the existing road.

Table 9.1.2 summarizes the concept of the alternative route study.

Table 9.1.2 Concept of Alternative Route Study for the Sta.17+500-Sta.20+182 Section

Route/Alignment	Design Concept	Conditions / Key Issues
Original Plan / Modified Original Plan	● South portal of the 2nd tunnel was planned to be located at 70 m from the 1st tunnel centre. ● The road alignment was connected to the Bridge No.4 at Sta.19+200.	● Technically not applicable unless modifying the tunnel south portal location as discussed in Chapter 10.
Alternative A	● Change of the modified original plan for the south portal approach section (The south portal of the 2nd tunnel is planned to be located at 30 m from the 1st tunnel centre) ● Construction of the two new lanes beside the existing road	● High-cut and traffic management during construction ● Different settlement between the existing and new high embankments ● The Bridge No.4 at Sta.19+200 already constructed with a dual carriageway ● ROW already acquired for the additional two lanes
Alternative B	● New Plan ● Two new lanes about 600 m to the east of the existing access road	● Longer roadway and pavement construction ● Extent of reduction of cut and fill volumes ● New ROW acquisition

A borehole was drilled near the Bridge No.4 to evaluate the settlement of the new high embankment at Sta.18+700 - Sta.20+000 (refer to Sub-section 6.4 of this Report).

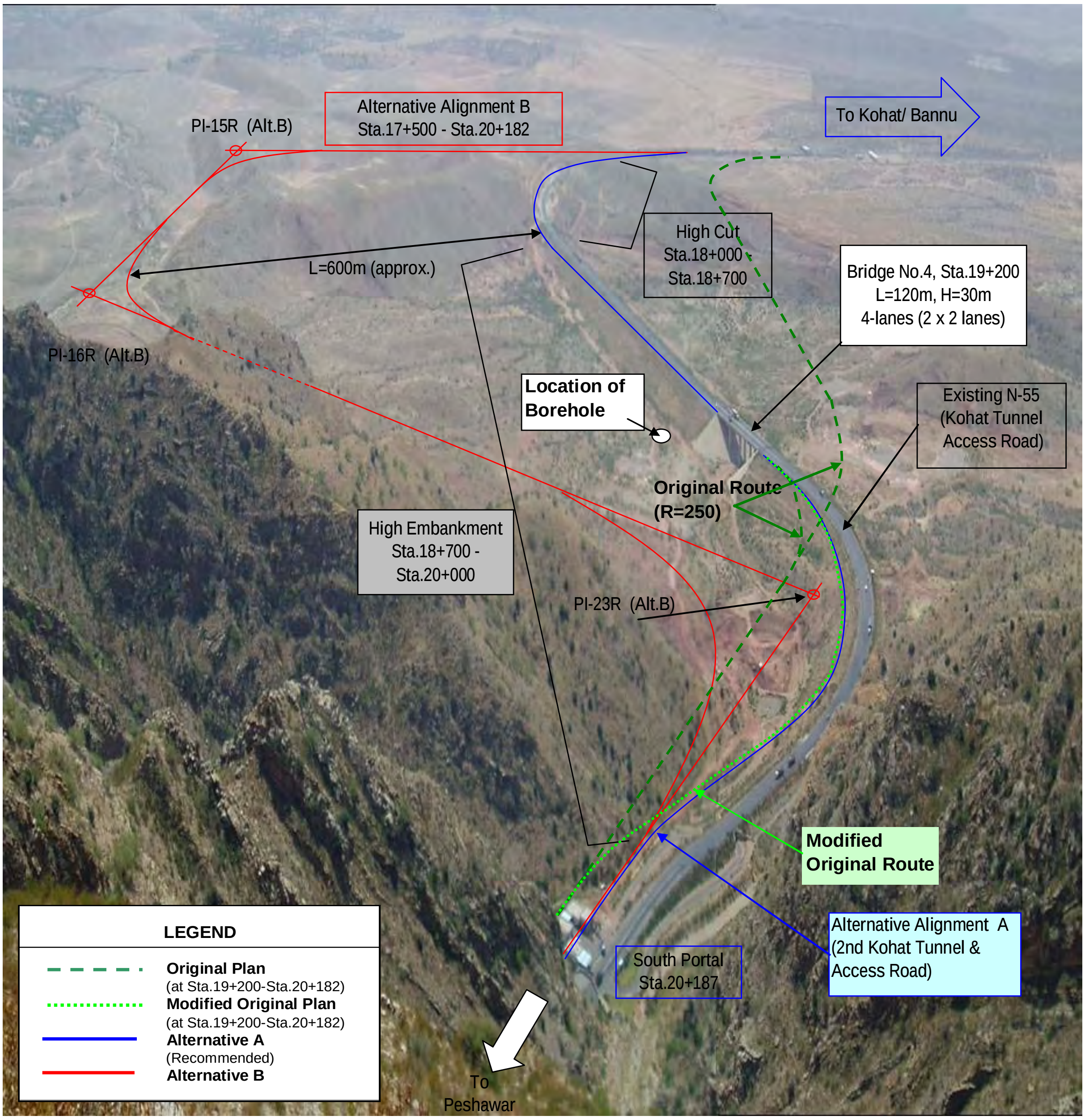
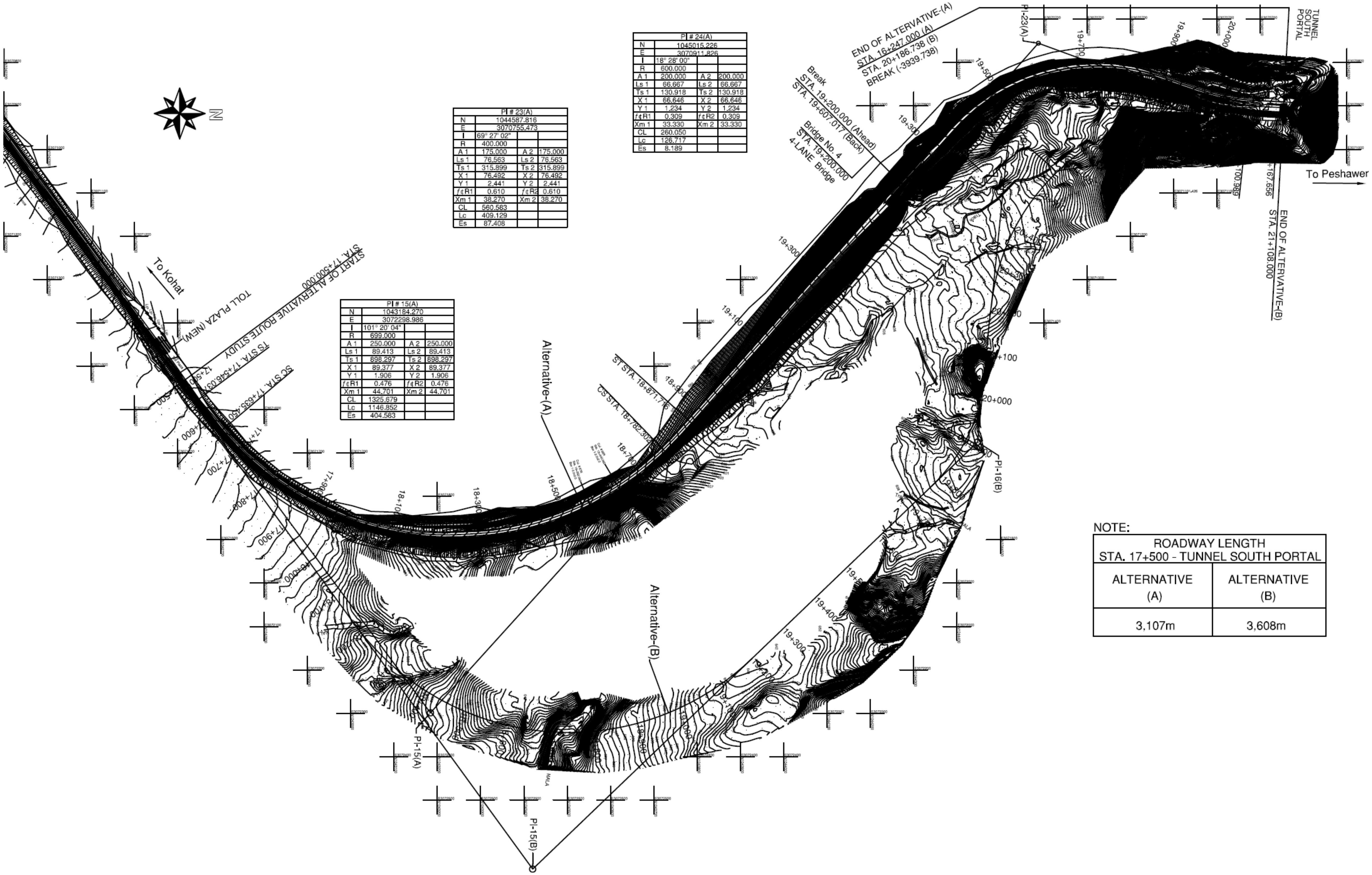


Figure 9.2.1 Alternative Routes at Sta.17+500-Sta.20+000

9.3 Preliminary Design and Cost Estimate

A topographic survey was conducted along the preliminary alternative road centreline from Sta.17+500 to Sta.20+182 (tunnel south portal) with a 100 m band and topographic maps were produced. The horizontal alignment was drawn on topographic maps (see Figure 9.3.1). Ground elevations were taken along the centreline for vertical alignment design.



PI # 23(A)			
N	1044587.816		
E	3070755.473		
I	69° 27' 02"		
R	400.000		
A 1	175.000	A 2	175.000
Ls 1	76.563	Ls 2	76.563
Ts 1	315.899	Ts 2	315.899
X 1	76.492	X 2	76.492
Y 1	2.441	Y 2	2.441
f¢R1	0.610	f¢R2	0.610
Xm 1	38.270	Xm 2	38.270
CL	560.583		
Lc	409.129		
Es	87.408		

PI # 24(A)			
N	1045015.226		
E	3070911.826		
I	18° 28' 00"		
R	600.000		
A 1	200.000	A 2	200.000
Ls 1	66.667	Ls 2	66.667
Ts 1	130.918	Ts 2	130.918
X 1	66.646	X 2	66.646
Y 1	1.234	Y 2	1.234
f¢R1	0.309	f¢R2	0.309
Xm 1	33.330	Xm 2	33.330
CL	260.050		
Lc	126.717		
Es	8.189		

PI # 15(A)			
N	1043184.270		
E	3072298.986		
I	101° 20' 04"		
R	699.000		
A 1	250.000	A 2	250.000
Ls 1	89.413	Ls 2	89.413
Ts 1	898.297	Ts 2	898.297
X 1	89.377	X 2	89.377
Y 1	1.906	Y 2	1.906
f¢R1	0.476	f¢R2	0.476
Xm 1	44.701	Xm 2	44.701
CL	1325.679		
Lc	1146.852		
Es	404.583		

ROADWAY LENGTH STA. 17+500 - TUNNEL SOUTH PORTAL	
ALTERNATIVE (A)	ALTERNATIVE (B)
3,107m	3,608m

Figure 9.3.1 Alternative Alignments at Sta.17+500 - Sta.20+182

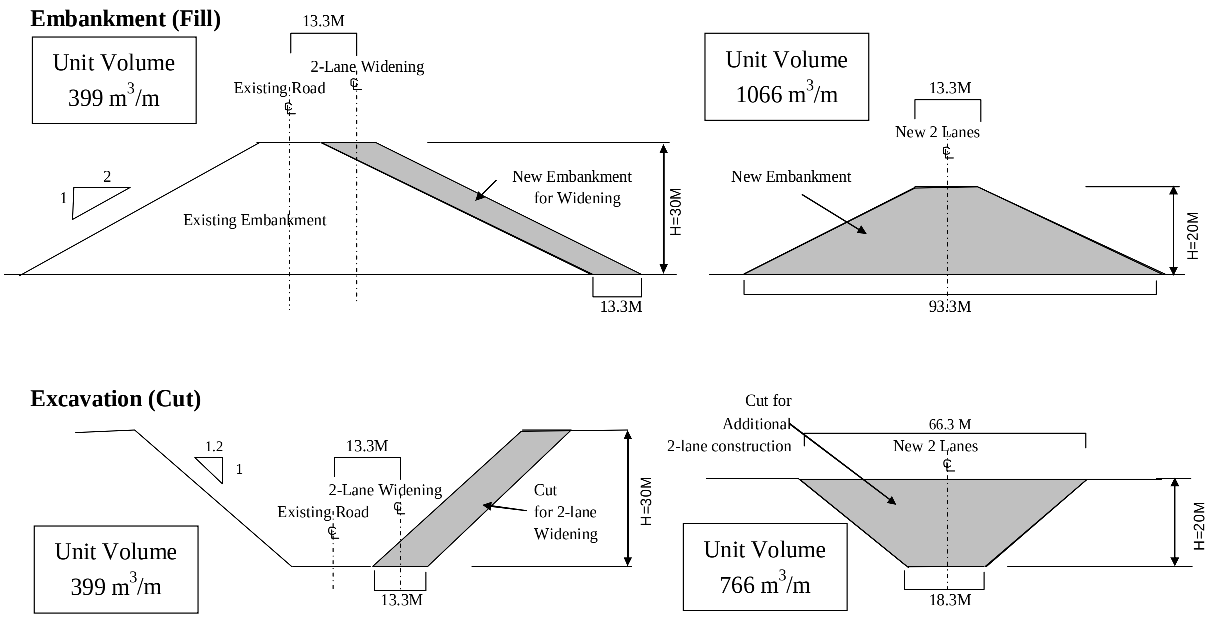
Approximate quantities and cost, including earthworks, pavement and other works were estimated for Alternatives A and B. The construction cost for Alternatives A and B was estimated at Rs.255.8 and Rs.628.1 million respectively (refer to Table 9.3.1). The construction cost of Alternative B is about 2.3 times higher than that of Alternative B.

Table 9.3.1 Construction Cost Estimate of Alternative Routes

Item / Description	Unit	Unit Price (Rs.)	Alternative A		Alternative B	
			Quantity	Amount (Rs.)	Quantity	Amount (Rs.)
Road Length from Sta.17+500 to Tunnel South Portal:			3,100 m		3,600 m	
● Earthworks						
- Roadway Embankment						
Common Materials	m^3	340	99,360	33,782,400	490,240	166,681,600
Rock	m^3	870	66,240	57,628,800	122,560	106,627,200
Traffic Safety Measure	sum	(10 of the above)		9,141,120		-
- Borrow Embankment	m^3	370	180,640	66,836,800	453,200	167,684,000
● Pavement						
- AC Wearing /AC Base	m^3	9,500	4,800	45,600,000	6,200	58,900,000
- Aggregate base / subbase	m^3	1,500	12,200	18,300,000	15,700	23,550,000
● Other Items (20% of the above)	sum			46,257,824		104,688,560
Total				277,546,944		628,131,360

Note: Approximate estimation only

The roadway length of Alternative B is approximately 500m longer than Alternative A. The largest construction quantity/cost difference between the two alternatives is in the earthworks because Alternative A consists of fill or cut work for the road widening while Alternative B is a new construction as illustrated in Figure 9.3.2.



Alternative A **Alternative B**

Figure 9.3.2 Comparison of Unit Earthworks Volume

9.4 Comparison and Evaluation of Alternative Route

Comparison and evaluation of the alternative routes were made taking into consideration technical, economic and ROW and other aspects as summarized in Table 9.4.1.

Table 9.4.1 Evaluation of Alternative Route Plans

Item	Alternative A	Alternative B
● Technical Aspects	△	○
- Roadway Length	3,100 m	3,600 m
- High Cut (Volume)	H=30 m, L=700 m ($165,600 m^3$)	H= 20 m, L=800 m ($612,800 m^3$)
- Adverse Effects to Traffic	Medium	Low
- High Embankment (Volume)	H= 30 m, L=1300 m ($280,000 m^3$)	H= 20 m, L=1000 m ($1,066,000 m^3$)
● Economical Aspects	○	△
- Construction Cost	Rs.256 million	Rs.628 million
- Additional ROW Acquisition Cost	None (ROW was already acquired)	Required
● ROW	○	×
- New ROW Acquisition	None	Required
- Relocation of Building, etc.	None	None
● Others	○	×
- Bridge No.4	Use of Bridge No.4 constructed with 4 lanes	Not using Bridge No.4
- Other Projects	-	An on-going development project at Sta.19+500 (R)
Overall Evaluation	○ (Recommended)	×

Notes 1: ○Good, △Fair, ×Bad
2: Construction methods which minimize disturbance to the existing traffic should be applied.

The Bridge No.4 (L=120 m, H=30 m) which has been constructed with a dual carriageway should be utilized. For Alternative A, the required ROW for the two new lanes was already acquired during the 1st Kohat Tunnel and Access Roads Project. For Alternative B, new ROW acquisition is required. Therefore, Alternative A (Modified Original Plan) is advantageous and is recommended from the economic, ROW acquisition and other view-points.

As approximately 30 cm settlement (1% of fill height) of the high embankment occurred during the 1st Kohat Tunnel and Access Roads construction, embankment settlement and its influence on the existing roadway were evaluated for Alternative A based on the geological investigation results. As the underneath soil is composed of granular materials and soft/hard rock, settlement will be minor and no serious adverse effects to the existing road are expected (refer to Subsection 6.4).

The estimated quantity of cuts at the section from Sta.18+000 to Sta.18+700 is 165,000 m^3 and 40% of which is classified as rock material. As the existing cut slopes are not steep (1 : 1.2), most of them could be excavated by bulldozer with ripper. Hard rock could be excavated with a combination of hydraulic breaker and control blasting (refer to Subsection 15.2) without much disturbance to the existing traffic.

Excavation will be executed to minimize materials falling down to the existing roadway. Installation of temporary concrete barriers along the roadway will be required to prevent falling rocks from hitting the traffic (refer to Subsection 15.2).

Chapter 10. LOCATION OF TUNNEL PORTALS

10.1 South Portal

10.1.1 Alternative Plans

The road alignment of the 2nd Kohat Tunnel had been examined in the design stage of the 1st tunnel, in which the south portal had been planned to be located at 70 m east of the 1st tunnel (referred to as the "original plan").

The approach road alignment for the tunnel south portal was changed during construction in order to reduce its grade (vertical alignment) from 6% to 4% without paying much attention to the creek (nullah) on the right side (referred to as the "modified original plan"). The south portal was still kept at 70 m of the 1st tunnel. The location of the Bridge No. 4 was changed accordingly, and it was planned that the tunnel approach will be connected to the Bridge No. 4 (L=120m) at Sta. 19+200, which will be constructed with a dual carriageway (4 lanes).

However, the Study Team noticed that the location of the south portal in the original and modified original plans is inappropriate for the following reasons.

- The portal is located at a site with difficult topography and geology, right on a steep and deep creek in the northeast of the tunnel, causing a risk of occurrence of flush water flow (estimated at 55 m^3/sec for a 50-year return period) and debris flow, and slope instability.
- High embankment is required in the north of the Bridge No. 4 (Sta. 19+300 - Sta. 19+800), with an estimated volume of over 1 million m^3 in the original plan. It is preferable to avoid such big embankment. The problem associated with high embankment is a need for costly construction of a large culvert or a bridge to drain water and debris flow from the creek. The possible radius of curvature in the northern approach to the bridge is only 250 m and it is inappropriate.
- For both the original and modified original plans, bridge or box culvert structures are required on the right side of south portal to release water from the creek.
- No sufficient space for the tunnel construction is available at the south portal because of the existence of the present control room.

The following two alternative locations for the south portal were examined together with the above original and modified original plans.

Alternative-A: The portal is located closer to the existing portal to avoid debris flow from the steep creek, but at a normally required minimum distance of 30 m center-to-center (about three times the tunnel width) to ensure safe construction of two parallel tunnels.

Alternative-B: The tunnel passes under the creek and becomes 420 m longer than the original plan. Taking into account the topographic and geological conditions, the location of south portal was selected at 100 m east from the existing portal.

The location of the south portal and the alignment of the tunnel approach road by alternative are illustrated in Figures 10.1.1 and 10.1.2, together with the original and modified original plans.

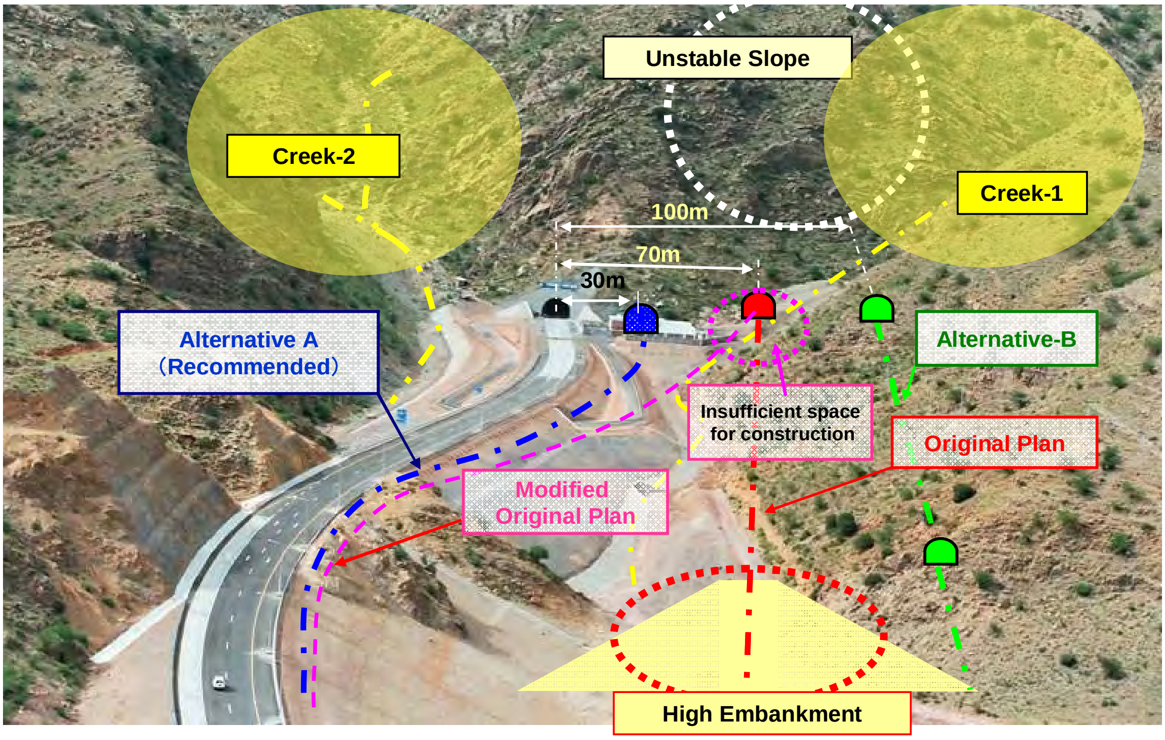


Figure 10.1.1 Location of the South Portal

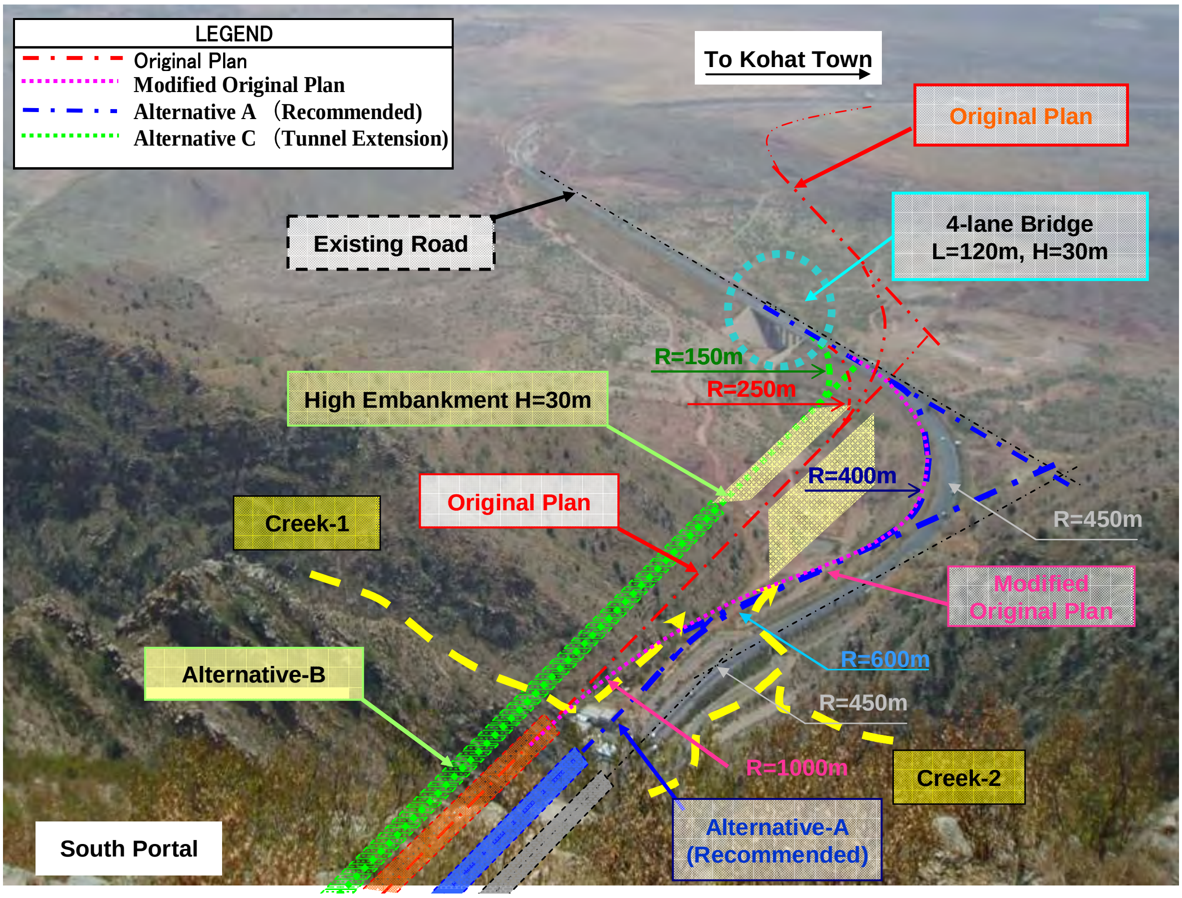


Figure 10.1.2 Road Alignment for the South Portal Approach Road

10.1.2 Evaluation of Alternative Plans

The two alternatives and the original/modified original plans were compared and evaluated in the following technical and economic aspects as summarised in Figure 10.1.3.

- Risk of flush water flow (estimated at 55 tonnes/sec) and debris flow from the steep creek located in the east of the portal.
- Radius of curvature for the approach road connecting to the existing 4-lane bridge.
- Availability of space for tunnel construction at the south portal.
- Earthworks quantities for the approach road construction from Sta.19+200 to Sta.20+186 (south portal).
- Requirements of additional structures for approach road or tunnel extension.
- Necessity of relocation of the existing tunnel control room at the south portal.
- Overall cost including earthworks, additional structures and relocation of the control room.

The original plan is not feasible because of technical defects and high cost. The radius of curvature for the bridge approach is only 250 m which is too small to ensure safety. This plan needs a high-fill just after the Bridge No.4 and a high-cut before the south portal

In the case of the modified original plan, the curved section starts inside the tunnel and extends along the existing road for about 200 m after the south portal. This alignment will provide safe approach for the Bridge No.4. However, this plan requires a bridge or box culvert construction to provide an opening for debris flow from the creek. As this plan intends to maintain the existing control room, securing a space for the tunnel construction between the control room and the creek is difficult as it is quite narrow.

Alternative-B has a critical defect. Apparently, an alignment which satisfies the minimum radius of curvature cannot be set at the northern approach to the Bridge No.4, meaning that the already constructed 4-lane bridge cannot be utilized. In addition, this alternative needs to extend the tunnel length by 420 m, as well as high embankment construction and additional ROW acquisition.

For the above reasons, Alternative-A is recommended because it is considered superior technically and economically, avoiding problems in the original and modified original plans. A 400m-radius curvature is secured for the approach to the Bridge No.4 satisfying the design standard. At the tunnel portal, a 600m-radius curvature is secured, which is sufficient for tunnel approach alignment. This plan can also secure a sufficient space for construction of an open channel for draining debris flow.

Alternative-A requires the relocation of the existing tunnel control room. A space can be secured in the west of the existing portal or on the left side of the north portal. The relocation should be completed in the initial stage of construction, but no difficulty is foreseen. As the current tunnel monitoring facilities (computers, CCTV, etc.) will require replacement at every 8 - 10 years, the time for the next replacement will coincide with the time for the 2nd tunnel construction.

It is considered that Alternative-A is the most economical and safe plan compared with other plans, while Alternative-B is the most expensive plan because of the need for extension of the tunnel length. The original plan is also expensive as high-fill, high-cut and structures are required. The modified original plan is not much expensive compared with the original plan and Alternative-B but this plan is not safe.

Items / Description		Original & Alternative Plans			
		Alternative-A (Realignment)	**Original Plan**	**Modified Original Plan [1]**	**Alternative-B (Tunnel Extension)**
Position of South Portal	**Distance from the existing tunnel**	**30m**	**70m**	**70m**	**100m**
Basic Concept of Plan	Alignment between the south portal and Bridge No.4.	The alignment of the new two lanes just beside the existing road from the south portal.	Connect the bridge No.4 and South portal by straight line.	The alignment of the new two lanes just beside the existing road from the south portal.	Passing under the deep creek by tunnel and connect to Bridge No.4
	Treatment of a deep creek on the right hand of the south portal	Provide a space for open channel [3]	No plans were shown how to treat the creek	No plans were shown how to treat the creek	No influence by creek
Technical Aspects	Risk of Debris Flow from Creek on East (55 ton/sec)	O A little	× Large A bridge or box culvert is required	× Large A bridge or box culvert is required	O None
	Radius of curve (m) for Bridge No.4 Approach	400m O Good	250m △ Poor	400m O Good	150m × Bad
	Space for the tunnel construction at South Portal	Sufficient O	Insufficient ×	Insufficient ×	Sufficient O
Economical Aspects	Embankment Quantity (m^3)	120,000 O Small	1,010,000 × Very Large	130,000 O Small	660,000 △ Large
	Roadway Excavation Quanity(m^3)	500 (rock) O Small	24,500 (rock) × Large	6,000 (rock) △ Medium	1000 (rock) O Small
	Requirements of Additional Structures at the south portal	O No	△ Yes Bridge or Culvert: L=100m	△ Yes Bridge or Culvert: L=100m	× Yes Tunnel Extension: L=420m
	Necessity of relocation of Control Room	× Yes	O No	O No	O No
	Overall Cost	O Low	△ High	△ Medium	× Very High
Evaluation		O **Good (Recommended)**	× **Bad**	△ **Fair**	× **Bad**

Notes: 1. The original road alignment was modified during the construction of the 1st tunnel access road to reduce a grade from 6% to 4% at Sta.17+500 - Tunnel South Portal.
2. O Advantageous △ Fair or possible with some measures × Disadvantageous
3. Measures against risk pf debris flows from the right hand creek

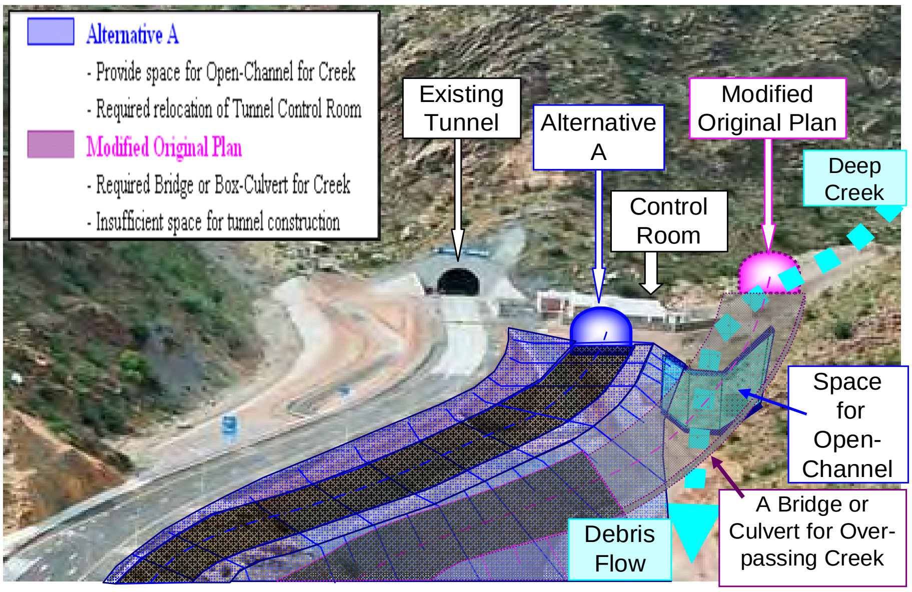


Figure 10.1.3 Evaluation of Alternative Plans

10.1.3 Elevation of South Portal

It is recommended to lower the elevation of the south portal of the 2nd tunnel by 4m from that of the 1st tunnel, for the following purposes:

- To secure enough opening for water and debris flow from the creek from the northeast of the tunnel (Creek-1 in Figures 10.1.1 and 10.1.2).
- To reduce the embankment height of the tunnel approach, thus reducing the construction cost.
- To prevent headlight beams of the northbound vehicles from disturbing the vehicles coming out from the tunnel

Keeping the same elevation for the north portal, the grade of the 2nd tunnel will be 2.4%. 0.2 % steeper than the 1st tunnel. Since the 2nd tunnel will be used for the southbound traffic in the down grade section, this grade will not affect the traffic flow and safety.

10.2 North Portal

In the original design, the north portal of the 2nd tunnel is located 30m center-to-center from the 1st tunnel, and at the same elevation. There is no problem in the original design, therefore it will be retained.