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Designing Roads and Retaining Structures for Nangarhar Province, Afghanistan

Rosa T. Affleck, Shad Sargand, Andrew Russ, Farid Momand and Michelle C. Perez Canals

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Abstract: Since 2002, coalition forces have focused on efforts to reconstruct and create new infrastructure in Afghanistan to help stabilize the nation and improve trade and opportunities for a livelihood. The U.S. projects in Afghanistan used common American construction standards. Applying these standards to the projects appeared reasonable, but, in actual practice, these standards have been difficult to implement owing to limited access to technical information, lack of information about local terrain and environment, and limited understanding of local social customs.

Afghanistan lacks skilled labor capable of performing quality work. In addition, there are few Afghan testing laboratories for conducting quality control. The quality of available construction materials is unreliable. Logistics are complicated because of the poor state of the transportation infrastructure, and there are no local systems necessary to sustain procurement efforts for major infrastructure projects. Even if a trained engineer is available and willing to work from a set of standards, the information needed to apply the standards may not be available and testing facilities to measure necessary parameters are lacking.

This report aims to overcome the lack of availability of actual data for Nangarhar Province. A rational range of values for parameters used in designing and building infrastructure is presented in this report from available geological information and climatic data. The geological data were used to obtain local soil classification and other relevant engineering information. Nangarhar Province was selected as a test case to gather together a pertinent set of design information and useful data with the idea that the process used to generate this report could be replicated for other regions in Afghanistan.

Preface

This study was conducted for the Army Study Program, project, *Development of Best Practice Engineering Design and Construction Criteria Incorporating Local and Cultural Influence for Nangarhar Province, Afghanistan*. The technical monitor at the sponsor agency is Rob Lambert, email: robin.b.lambert2@usace.army.mil.

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Colonel Kevin J. Wilson was Commander and Executive Director of ERDC. Dr. Jeffery P. Holland was ERDC Director.

Acronyms and Abbreviations

AED	Afghanistan Engineer District
AEN	Afghanistan Engineer District–North
AES	Afghanistan Engineer District–South
ANA	Afghan National Army
ANP	Afghan National Police
ANSF	Afghan National Security Forces
CD	Capacity Development
CERP	Commander's Emergency Response Program
COIN	Counterinsurgency
CSTC-A	Combined Security Transition Command–Afghanistan
DOD	Department of Defense (U.S.)
HMA	Hot Mix Asphalt
IED	Improvised Explosive Device
IJC	ISAF Joint Command
ISAF	International Security Assistance Force
JCISFA	Joint Center for International Security Force Assistance
JRAC	Joint Regional Afghanistan Security Forces Compound
LNs	Local Nationals (Afghans)
MILCON	Military Construction
MOD	Ministry of Defense (Afghanistan)
MOI	Ministry of Interior (Afghanistan)
NCDS	National Climatic Data Center
NGO	Non-Governmental Organization
PE	Project Engineer
PgMP	Program Management Plan

PM	Project Manager
POC	Point of Contact
PRT	Provincial Reconstruction Team
QC & QA	Quality Control and Quality Assurance
QAR	Quality Assurance Representative
RC(N)	Regional Command–North
RC(W)	Regional Command–West
SSTR	Stability, Security, Transition, and Reconstruction
USACE	United States Army Corps of Engineers
USCS	Unified Soil Classification System
USGS	United States Geological Survey
USFOR-A	United States Forces–Afghanistan
USG	United States Government

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1 Introduction

1.1 Importance

This study was intended to support Stability, Security, Transition, and Reconstruction (SSTR) operations for the following objectives:

- According to Department of Defense Directive 3000.05 (2009), DOD's SSTR mission is responsible for ensuring that DOD possesses the capability and capacity to establish civil security and civil control, to restore or provide essential services, to repair critical infrastructure, and to provide humanitarian assistance in theater. Some of the requirements under this directive are providing the local populace with security, restoring essential services, and meeting humanitarian needs. Restoring essential services requires Afghan nationals to obtain the technical capacity to support and manage their infrastructure.
- The U.S. Army Corps of Engineers (USACE) supports contingencies in combat, stability, and disaster operations (USACE 2009a). As USACE executes and manages projects in a challenging, in-theater environment, USACE must utilize capabilities, knowledge, and scientific, technological, and engineering expertise that are available internally as well as externally.
- USACE has embraced Capacity Development (CD) programs domestically and internationally. For STTR operations, CD is employed as ways to teach local nationals to manage, operate, and maintain the new or repaired infrastructure (i.e., facilities, systems, and equipment) upon handover, without additional support from the U.S. Government (USG) or other coalition partners. CD includes coaching, training, teaching, and mentoring programs aimed at strengthening public and private sector management, engineering, and technical capabilities to support self-reliance among foreign and domestic entities (USACE 2008). Thus, local infrastructure capability will be developed and ownership will be gained.

Since 2002, coalition forces have focused on efforts to build new and repair existing infrastructure to stabilize Afghanistan and improve the nation's economy. These projects were funded by the U.S. and other countries, working through construction contractors. The U.S. projects used the common construction standards of the USA. Applying these standards in the projects appeared reasonable, but, in actual practice, these standards are difficult to implement owing to inadequate access to technical information because of the country's limited capability, and narrow understanding of local social customs.

Afghanistan has limited engineering capability because only a few engineers have graduated from Afghan universities in the past 35 years, and these engineers often lack basic knowledge. The country also lacks the skilled labor capable of performing quality work. Expatriates have not returned, and many are not likely to do so. In addition there are few qualified local quality control laboratories. The quality of available construction materials is unreliable. Logistics are complicated owing to the poor state of the transportation infrastructure (Affleck and Freeman 2010). There is no local system to sustain procurement efforts necessary to major infrastructure projects. Even if a trained engineer is available and works from a set of standards, the information needed to apply the standards may not be available because of the lack of accurate data and testing facilities to measure necessary parameters.

Many projects in Afghanistan take place in remote or dangerous locations (or both). Both military and civilian engineers find it difficult to assess sites, plan projects, and control quality. Because of this lack of information during the planning process, the projects are often redesigned as they are built, thus leading to schedule creep and unanticipated problems during construction.

The terrain in Afghanistan is quite complex. Afghanistan has three distinct geographic regions: the Central Highlands, the Southern Plateau, and the Northern Plains. The Central Highlands are part of the Himalayan Mountains, having deep, narrow valleys, deserts, and some meadows. The mountains are more than 6562 ft (2000 m) above sea level in elevation. The two most strategically significant passes are the Shebar Pass, northwest of the capital, Kabul, and the Khyber Pass, leading to the Indian subcontinent. The Southern Plateau contains a variety of deserts and generally is infertile, except for the river deltas. The Northern Plains are mostly flat, with some fertile foothills.

In parallel to this study, experiences and lessons learned from construction projects in Afghanistan were compiled from construction personnel (Affleck et al., in press). The most common problems observed included:

westernized requirements are not applicable in some cases, affecting design and construction, and materials; climatic and weather conditions are not accounted for; there are security problems during construction, such as material theft, and delays caused by insurgent attacks, all affecting the schedule; the quality of materials used varies dramatically, which affects the quality of the construction. All of these problems reinforce the need to examine all relevant information to develop an appropriate design matrix. Thus, it becomes necessary to make assumptions based on previous experience.

This study assessed the appropriate design criteria for Nangarhar Province. Designs of infrastructure included the constraints imposed by “real world” conditions. Here, we integrated design criteria and algorithms developed in the U.S. and merged them with the terrain, geology, soils, climate, and weather conditions of the area. We reviewed the literature to select a range of parameters that apply to construction in Nangarhar. This integrated study should facilitate long-lasting infrastructure, planned and built with social and cultural knowledge in mind, and with fundamental engineering research.

In addition, the information in this study would aid Afghan engineers and practitioners as they develop their essential design and construction standards. Transferring and sharing fundamental engineering could boost knowledge necessary for effective design appropriate for locals’ technical capability. It would strengthen an understanding of what it takes to maintain the infrastructure being constructed in the rebuilding of their region and country. Thus, local engineering capability is developed and ownership is gained for the local population. This study increased guidance and engineering awareness to promote technical knowledge in Afghanistan, primarily for Nangarhar Province. This would be essential for planning and developing infrastructure in Afghanistan, and could be valid in similar climatic and environmental conditions in other parts of the world.

1.2 Objective

An information gap was found between expecting to apply traditional U.S. engineering and construction standards to projects in places like Iraq and Afghanistan, and the reality on the ground. The constraints imposed by “real world” conditions must be faced. Although the design and construction standards proposed herein are specific to Nangarhar Province, the methods used to develop these standards could be tailored and applied to the rest of Afghanistan and, potentially, to other post-conflict and developing countries of similar climate. These methods could be valid in developing countries by providing sustainable infrastructure solutions that can withstand the local physical and environmental conditions. Therefore, the results from this study would provide positive benefits for SSTR operations, and will support the current U.S. Army efforts in Afghanistan.

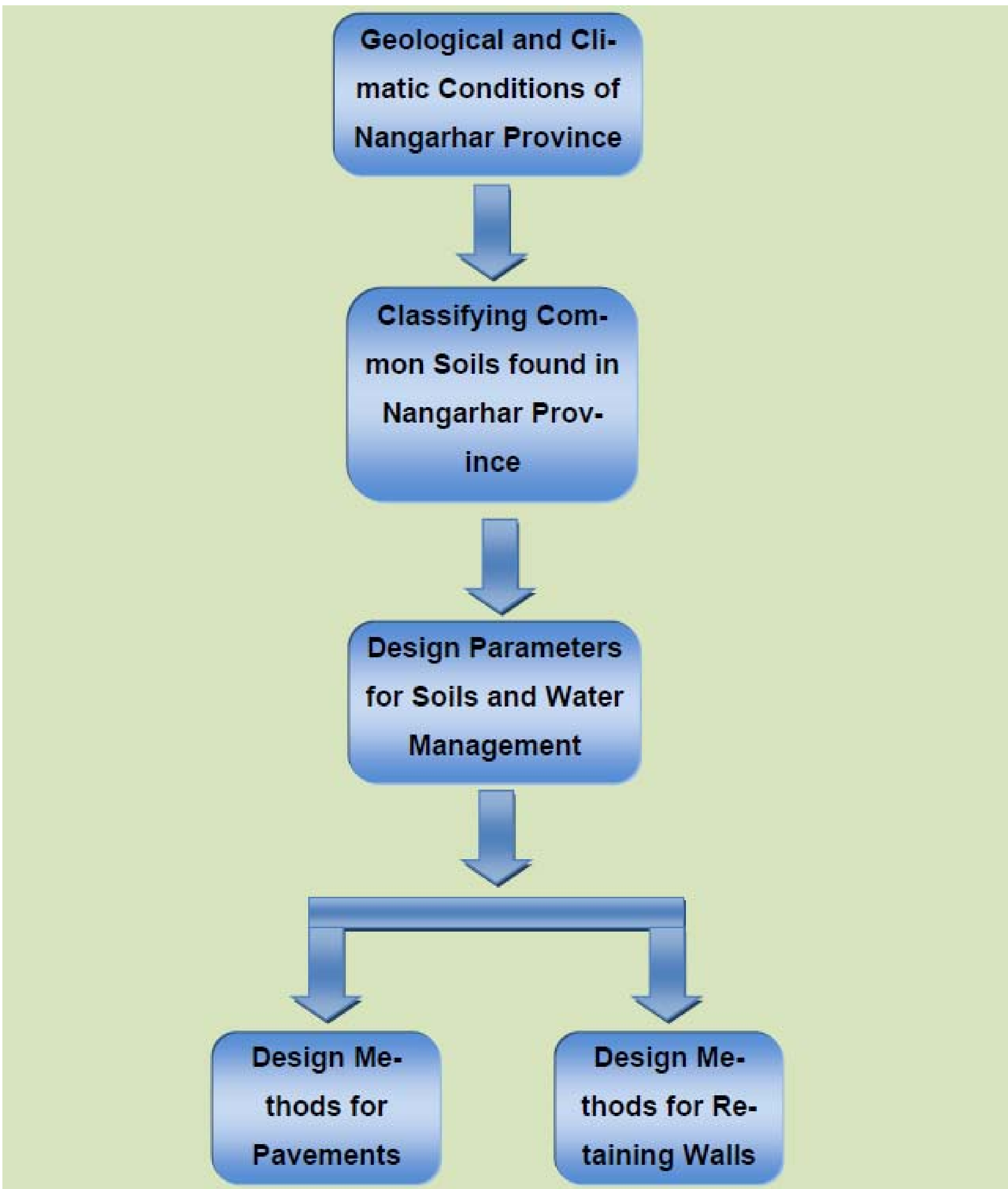


Figure 1. Flowchart illustrating how to use this report for designing projects.

1.3 Approach

This report is outlined in the flowchart in Figure 1, around which the report is designed. The report is organized as follows. This introductory chapter is followed by background information on Nangarhar Province, including the economy, people, and type of structures they use. The next chapter discussed the geology, geophysics, and soils of the region. Then the following chapter described climate and general weather data and followed by typical parameters for soils and other infrastructure materials from the region. A summary section for design and construction procedures that can be applied in Nangarhar is included for flexible pavements, rigid pavements, and retaining walls. Finally, a concluding summary is presented.

The background information specific to Nangarhar Province is gathered from the authors of this report who are Afghans and are very familiar with the area. In particular they grasped the economy, the existing infrastructure, and the local ways of housing construction. Various sources provided the topography and vegetation of Nangarhar based on a terrain analysis of Afghanistan (East View Cartographic, Inc. 2003). This terrain information was derived from Soviet military topographic mapping and analysis.

The geology and geophysics data are compiled from U.S. Geological Survey (USGS). These data are important for defining the soil properties critical for design and construction parameters.

The general climate information of the area is taken from terrain analysis (East View Cartographic, Inc. 2003). Temperature and precipitation (i.e., rain and snow) data recorded at various stations in Nangarhar Province are used to understand the climatic regime influencing the design parameters. Snow cover assessment is used to determine the affected areas.

The typical parameters for soils and other infrastructure materials from the region are summarized using the design and construction procedures that can be applied in Nangarhar, including design procedures for flexible pavements, rigid pavements, and retaining walls.

2 Background of Nangarhar Province

Nangarhar Province is located in eastern Afghanistan between the border with Pakistan and the provinces of: Konar, and Laghman to the north; Kabul and Lowgar to the west. The capital is Jalalabad, and the province includes 22 districts (Fig. 2). The population is about 1.35 million, overwhelmingly (more than 90%) Pashtun, with small minorities of Pashai, Arabs, Tajiks, etc. The predominant Pashtun groups are Shinwaru, Khogiani, Mohmand, Ghilzai, and Kuchi. The literacy rate is about 27%. There are two universities: Jalalabad University, which is an agricultural school, and Nangarhar University, which has an engineering program.

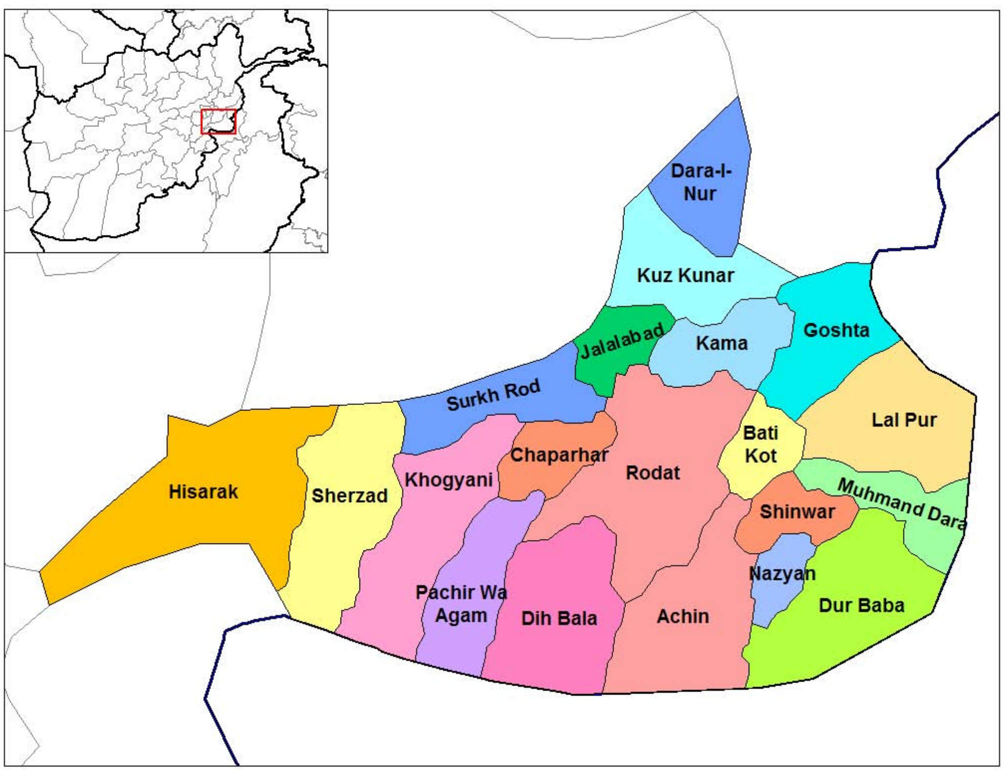


Figure 2. Nangarhar Province and its districts.

The Kabul River flows through the province, and joins the Surkh Ab and Kunar rivers near Jalalabad. To the west, south, and east of the Kabul River Valley lay the Safed Koh Range of mountains. Nangarhar also includes the Khyber Pass. Thus, Nangarhar province and the Kabul River Valley serve as a corridor between the Afghan capital of Kabul and Pakistan via Peshawar.

2.1 Economy

The plains around Jalalabad form one of the major agricultural areas of Afghanistan. A strong agricultural base and the major trade route between Kabul and Peshawar make Nangarhar's economy diverse. The agricultural sector includes wheat, rice, corn, vegetables, sugar, and cotton. Animals, such as cows, sheep, and goats, are raised for meat and milk, including animals kept by the nomadic Kuchi people, who generally migrate outside the province after winter ends. There is a significant group of people engaged in trading, whether import/export businesses, shopkeepers, or truck drivers. Many others are engaged in day labor.

2.2 Infrastructure

There are three main roads or highways: one from Kabul to Jalalabad to Torkham, one from Mehtarlam to the Kabul–Jalalabad highway, and one from Marawara to Jalalabad. There are also paths or tracks connecting the 22 districts in the province. The roads in the province are classified as follows: 54% can accept car traffic in all seasons, 34% can take car traffic in some seasons, and the remaining 12% of the province has no roads. Electricity is available to about 47% of the population, predominantly in the provincial capital of Jalalabad (Program for Cultural & Conflict Studies 2010) though another source reports 19% access overall and 83% access in urban areas (Ministry of Rural Rehabilitation and Development 2010). The province has 9 hospitals and 229 clinics. Water is obtained from karezes, shallow wells, rivers, and springs. Safe drinking water is available to 43% of households (62% in urban areas). Wireless telecommunication from Roshan, Areeba, and AWCC covers much of the province, including the major transit routes.

2.3 Housing construction

The variety of climates and elevations in the province ensures that a range of building materials and techniques are used to match local conditions. Significant factors include society, affordability, accessibility of the site, and temperature and climate. New construction tends to consist of a single story (ground floor) house; this is typically expanded by building a first floor on top of the house rather than expanding the ground floor. Family sizes are large—8 to 18 people—often augmented by extended family members. There are four main house styles used in Nangarhar: Adobe brick, stack wall (Paskha wall), stone masonry, and modern brick masonry. These are discussed in more detail below.

The most common type of house in Nangarhar is that made from adobe, or sun dried brick. Adobe is used in about 90% of construction, widely chosen because of its ready availability and low cost. Different sizes of brick may be obtained; the most common size is 20 ×10 ×5 or 6 cm. The bricks are held in place with a mud mortar. Burnt bricks are placed at the top of two layers of a wall to protect against rain erosion. Roofing is made by erecting a timber structure atop the walls and covering it with mud.

Stack wall or Paskha wall buildings are made of locally available mud without additives. Local earth is mixed with water into a puddle and left for 12 hours. The mud is then dug out in lumps and stacked to build a wall, often on a dry stone foundation. The mud may be reinforced with wood or stone. Older houses may have walls as thick as 2 m, but more typical today are walls about 60 cm thick at the base, and gradually narrowing to about 40 cm at the top, which may be 2.5 to 6 m high. The top portion of the wall is called Sarmate, and is specially formulated to resist weather. The Sarmate may be one of three types: use of additional mud to make a wider top so that rain falls directly to the ground rather than running down the wall, a layer of thin stones topped with an additional layer of Pakha (mud), or a layer of wood topped by Paskha. Some Paskha houses in Nangarhar are still standing after more than a century of use.

In more mountainous areas where stone is available, stone masonry houses are more common. Walls are typically 60-cm thick and the houses are one or two stories. Wood is used for bond beams. Semi-liquid dried mud (adobe) may be used as a mortar.

Lastly, some more modern style buildings also exist, typically using burnt brick masonry, with or without a frame, and with a reinforced concrete foundation, columns, stairs, and roofs. These dwellings can be as tall as five stories.

2.4 Topography

The terrain is mountainous, dissected by numerous ravines, gullies and river valleys (East View Cartographic, Inc. 2003). The horns of the Hindu

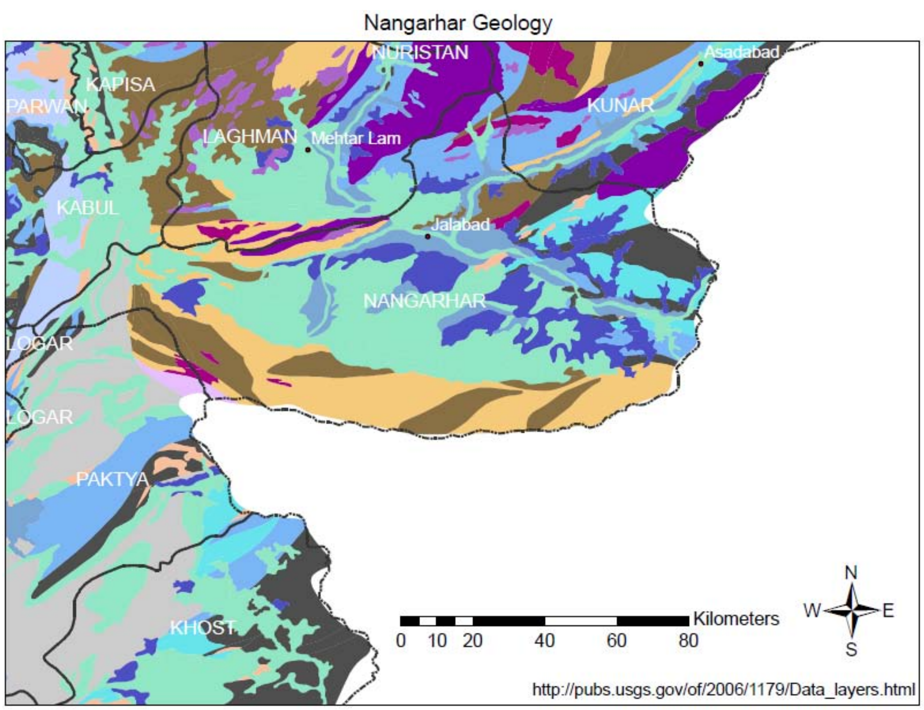
Kush Mountains are located in the north of the Jalalabad, and the Safed Koh Range and its horns are located in the south. The typical elevations of the mountains are in the range of 1500–3000 m above sea level. In the higher elevations, the slopes ranged from 20–40°; while at the lower elevations, the slopes are relatively flat (under 10°). The north faces of the Safed Koh range above 3800 m and have glaciers. The valleys are gentle rolling and dissected with numerous rivers, ditches, and irrigation canals for agriculture.

2.5 Vegetation

The mountains are mostly forested. Deciduous forests are common at elevations of 1300–1800 m and conifers are present at higher elevations. The lower parts of mountainsides are covered with high steppe type vegetation that are drought resistant and patches of shrubs (East View Cartographic, Inc. 2003). Orchards and farm lands are common in the valleys.

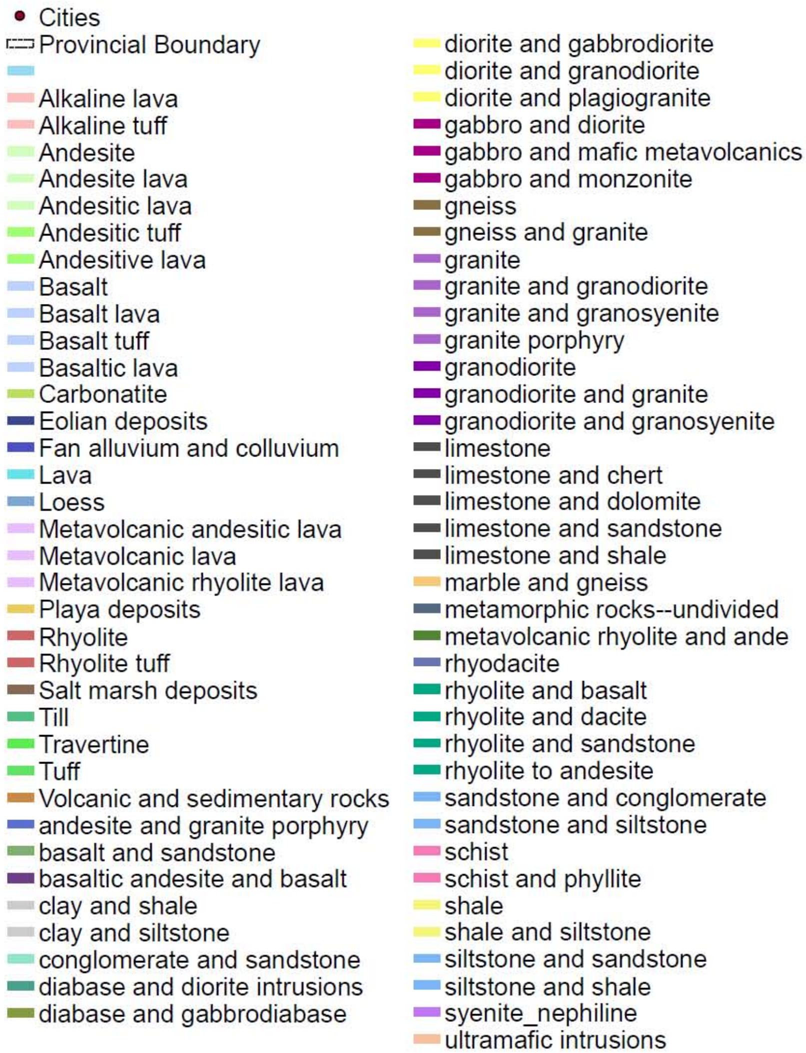
3 Geology, Geophysics, and Soils of the Nangarhar Region

The geospatial data for the Nangarhar Province are acquired from USGS (http://afghanistan.cr.usgs.gov/). The geology of Afghanistan is mapped and zoomed-in to focus on Nangarhar Province (Fig. 3). The central apart of the region is mostly conglomerate and sandstone (which are the bright greenish blue areas), sandwiched between fans of alluvium and colluvium (areas shaded in dark blue) and loess (areas shaded in purple gray). Marble, gneiss, and gneiss–granite border the southern part of the province (areas shaded with light brown and dark brown). Limestone is found on the upper western corner of the province (shaded areas of dark gray). The geology was used to determine the soils information pertaining to design and construction standards for the area.



a. General geology (data from USGS).

Figure 3. Geology of Nangarhar Province.



b. Legend.

Figure 3 (cont'd). Nangarhar Province.

The geology of the region revealed a general description of the engineering properties of rocks and soils. Visual identification of rock types and crushing tests could provide the general type of gravel materials available in the area. Acid tests are also performed to determine the characteristics of other materials, such as shale, tuff, marble, and limestone. These characteristics are described in Figure 4. For example, conglomerate and sandstone can be found in the central part Nangarhar Province. The conglomerate rocks appeared to be fragmental, such that the pieces are similar to broken concrete. The sandstone is characterized by a gritty sandpaper feel. Both of these rocks could produce hard coarse- and fine-grained materials for construction.

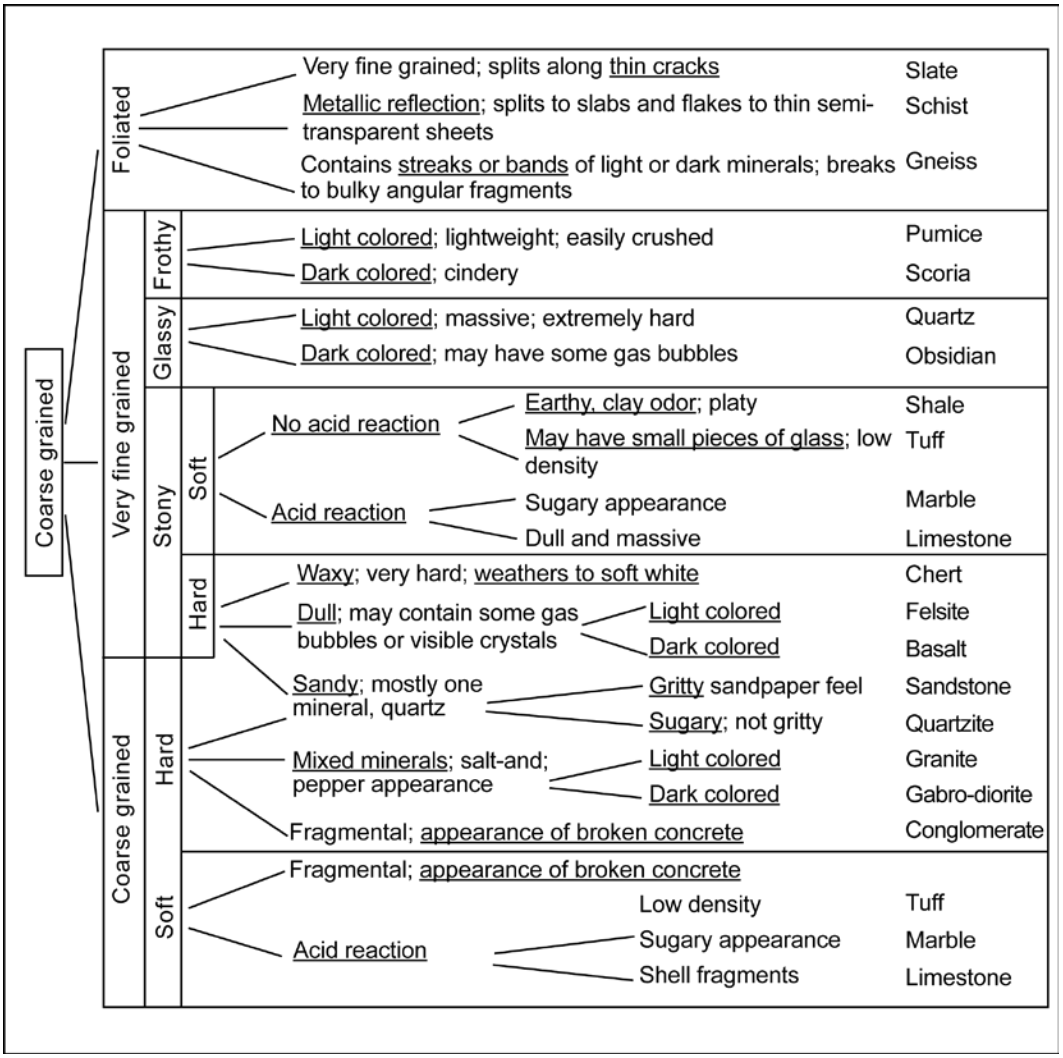


Figure 4. Rock identification (U.S. Army 2006).

Table 1 describes the engineering properties of various rocks and their relative uses for roads and retaining walls, as an aggregate, and base course, or as subbase. These characteristics include:

- Toughness—mechanical strength determined by resistance to crushing and breaking. Estimate this property by trying to break a rock with a hammer or by measuring a rock’s resistance to penetration using impact drills.
- Hardness—resistance to scratching or abrasion. Estimate this property by trying to scratch the rock with a steel knife or nails. Soft materials will scratch readily; hard materials are difficult or impossible to scratch.

- Density—weight per volume. Estimate this property by hefting a rock sample and comparing two samples of equal size.
- Durability—resistance to slaking or disintegration caused by alternating cycles of wetting and drying or freezing and thawing. Estimate this property by observing the effects of weathering on natural exposure of the rock.
- Chemical stability—resistance to reaction with alkali materials in Portland cements. Several rock types contain impure forms of silica that react with alkalis in cement to form a gel that absorbs water and expands to crack or disintegrate the hardened concrete. Estimate this potential alkali–aggregate reaction (in the field only) by identifying the rock and comparing it to known reactive types or by investigating structures in which the aggregate has been previously used.
- Crushed shape—form that a rock takes after crushing; bulky angular fragments provide the best aggregate for construction. Estimate this property by breaking a sample of the rock into smaller pieces.
- Surface character—bonding characteristics of material; excessively smooth, slick, nonabsorbent aggregate surfaces bond poorly with cement. Excessively rough, jagged, or absorbent surfaces are undesirable because they resist compacting and placement and require excessive cementing material. Visually inspect the rock surface and feel the surface texture.

The process used for classifying the general soil properties was derived from the USGS soil taxonomy (Table 2). The soil taxonomy was translated for each polygonal area (Buol et al. 2003). These areas were verified using the descriptions from *Terrain Analysis of Afghanistan* (East View Cartography, Inc.2003) as some of these locations were described in the textbook. Figure 5 illustrates the predominant soil types within Nangarhar, using the Unified Soil Classification System (USCS).

Soil samples from several boreholes in Jalalabad were analyzed for a couple of USACE projects. The soil samples, classified in USCS, are summarized in Table 3. Such soil data having higher fidelity are very limited and difficult to find in the province (and even for the entire country).

Table 1. Rock characteristics. Highlighted in gray are potentially common rocks in Nangarhar Province.

Rock Type	Toughness	Hardness	Durability	Chemical Stability	Surface Character	Crushed Shape	Use as Aggregates		Use as Base Course of Subbase
							Asphalt	Concrete	
Granite	Good	Good	Good	Excellent	Fair to good	Good	Fair to good	Fair to good**	Good
Syenite							Good	Good	
Gabbro diorite	Excellent	Excellent	Excellent	Excellent	Excellent	Good	Excellent	Excellent	Excellent
Diabase basalt	Excellent	Excellent	Excellent	Excellent	Excellent	Fair	Excellent	Excellent	Excellent
Felsite	Excellent	Good	Good	Questionable	Fair	Fair	Poor*	Fair	Fair to good
Conglomerate brecia	Poor	Poor	Poor	Variable	Good	Fair	Poor	Poor	Poor
Sandstone	Variable	Variable	Variable	Good	Good	Good	Poor to fair	Poor to fair	Fair to good
Shale	Poor	Poor	Poor	Questionable	Fair to good	Poor	Poor	Poor	Poor
Limestone	Good	Good	Fair to good	Good	Good	Good	Fair to good	Good	Good
Dolomite							Good	Good	
Chert	Good	Excellent	Poor	Poor	Fair	Poor	Poor*	Poor**	Poor to fair
Gneiss	Good	Good	Good	Excellent	Good	Good to fair	Good	Good	Good
Schist	Good	Good	Fair	Excellent	Poor to fair	Poor to fair	Poor to fair	Poor to fair	Poor to fair
Slate	Good	Good	Fair to good	Excellent	Good	Poor	Poor	Poor	Poor
Quartzite	Excellent	Excellent	Excellent	Excellent	Good to fair	Fair	Good	Fair to good**	Fair to good
Marble	Good	Fair	Good	Good	Good	Good	Fair	Fair	Fair

*Reacts (Alkali-aggregate)

**Use anti-stripping agents

Table 2. Description and translation of general engineering soil classes.

Soil Taxonomy	Soil Description (Terrain Analysis)	Engineering Soil Class (USCS)
Calcixeralfs with Xerochrepts	Sandy loam	SM
Rocky land with Lithic Haplocryids	Rock	BR
Xerochrepts with Xerorthents	Rubble and loam or rubble and sandy loam	GM
Xerorthents with Xeropsamments	Sandy loam	SM

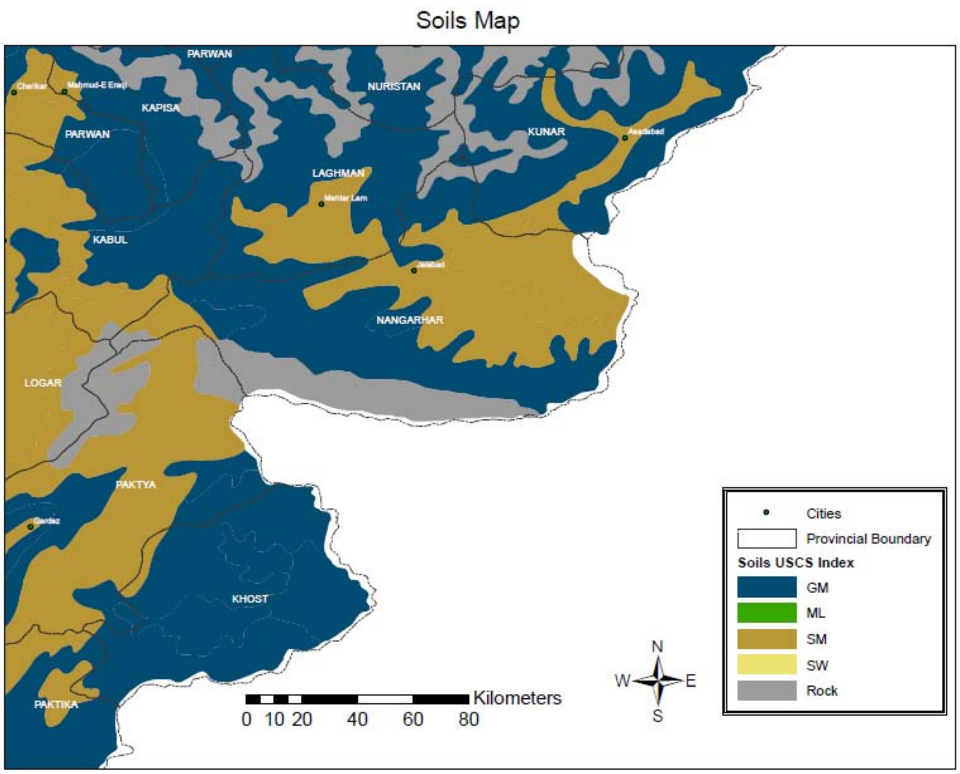


Figure 5. General engineering classification of soils in Nangarhar Province.

The results indicated variation in soils that can be found at least within Jalalabad: silty gravel and silty gravel with sand; in a few places silty sand to silty sand with gravel; sandy, silty clay with gravel; and silt and sandy silt with gravel in some places. Based on the boreholes data, the soils at 1- and 1.5-m depths are predominantly silty gravel with sand (GM), silty sand with gravel (SM) and sandy silt (ML). Poorly graded gravel with sand (GP) and sandy silty clay with gravel (CL-ML) soils are found in place and at depth. The soils data are used to verify the general engineering soils class

(Fig. 5). The soil in Nangarhar Province is composed of silty sand (SM) in the north region, Silty gravel, Silty gravel with sand (GM) in central area, and exposed rock in the south.

Table 3 in Chapter 5 shows the correspondence between the USCS and the American Association of State Highway and Transportation Officials (AASHTO) soil classification systems, which engineers predominantly, use. The tables that provide typical material properties for the types of soils seen in Nangarhar can be found in the Chapter 5.

Table 3. Summary of engineering soil classification on found in several bore holes from two USACE projects in Jalalabad.

USCS	Description	At depths (m)
CL-ML	Sandy silty clay with gravel	2 and below
GP	Poorly graded gravel with sand	1.5 and below
GM	Silty gravel, Silty gravel with sand	1 and below
GP-GM	Poorly graded gravel with silt and sand	4 and below
ML	Silt, sandy silt with gravel	1 and below
SM	Silty sand, Silty sand with gravel	0.75 and below

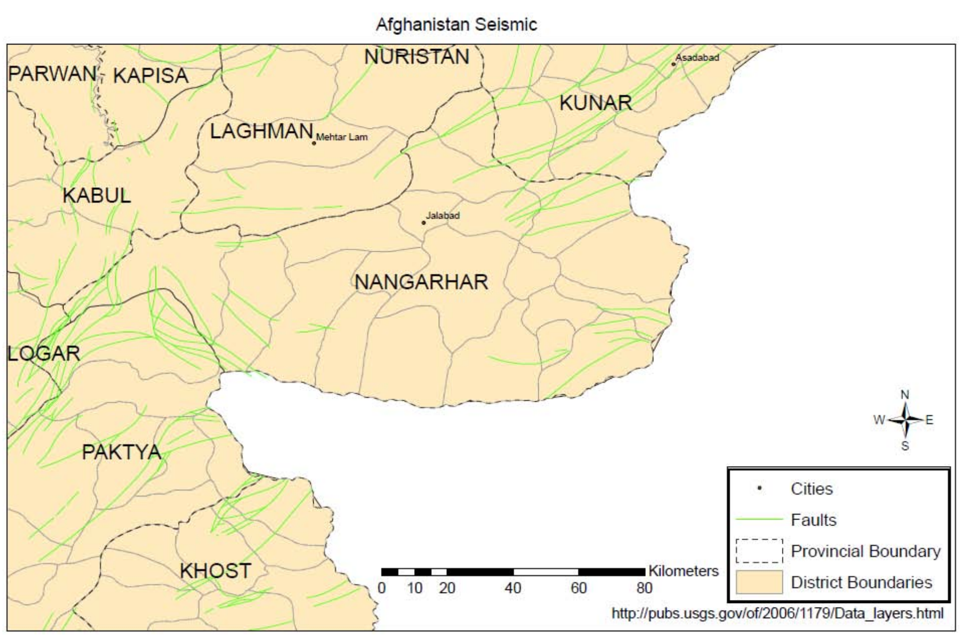


Figure 6. Location of seismic faults in Nangarhar Province.

From a design engineer's perspective, it is important to know the history of seismic activity at the site and where seismic faults can be found. In

Nangarhar Province, faults are found in the northeast, southeast, and west of the province (Fig. 6).

4 Regional Climate

The general climate of Nangarhar Province is continental[1] and varies according to elevation. The weather is moderately warm in the valleys and low lying areas. Where the climate at higher elevations is cold, the air can have a severe effect the on human activity (East View Cartographic, Inc. 2003).

The climatic factors of the region play a significant role in the performance of infrastructure. These factors must be incorporated into design and construction criteria. The weather in Nangarhar can affect construction schedules. Temperatures influence the materials that can be used, and affect many aspects of the construction, including scheduling. Also, the precipitation of an area has to be taken into consideration when designing for drainage to prevent flooding and erosion. For this reason, Nangharhar weather data were collected. Various data sets were available through CRREL researchers. One data set was compiled into USACE Hydrologic Engineering Center Data Storage System (HEC-DSS) obtained through National Climatic Data Center (NCDC).

Figure 7 is a map of Nangarhar showing the locations of the weather stations used in this report, which are Jalalabad, Ghazi Abad, Agam, and Camp Torkham. Data came from Afghan Meteorological Department Agromet (AMA) and World Meteorological Organization (WMO) stations. These sources are captured by NCDC and METAR format. METAR is a data format for reporting weather information used by pilots and meteorologists; the acronym roughly translates from French as Aviation Routine Weather Report. The weather station's approximate locations (latitude and longitude in decimal degrees) and elevations are as follows:

- Agam—Latitude 34.04, Longitude 70.41; Elevation 2370 m.
- Ghazi Abad—Latitude 34.24, Longitude 70.87; Elevation 500 m.

1 Continental climate is a climate that is characterized by important annual variation in temperature owing to the lack of significant bodies of water nearby. Regions having a continental climate exist in portions of the Northern Hemisphere continents (especially North America and Asia), and also at higher elevations in other parts of the world. Only a few areas in Iran, adjacent Turkey, Afghanistan, Pakistan, and Central Asia show a winter maximum in precipitation, and this typically melts in early spring to give short-lived floods.

- Camp Torkham—Latitude 34.133, Longitude 70.067; Elevation 2770 m.
- Jalalabad—Latitude 34.4, Longitude 70.5; Elevation 550 m.

4.1 Temperature

Temperature data from Jalalabad are plotted as follows: Average daily temperature 2003–2010 in Figure 8, maximum daily temperature 2005–2010 in Figure 9, and minimum daily temperature 2005–2010 in Figure 10. Average daily temperatures in Jalalabad are about 5–15°C in winter, 15–30°C in spring and fall, and 30–40°C in summer. Maximum daily temperature in summer can reach 45°C, and minimum daily temperatures are rarely below 0°C. Figure 11 shows average daily temperatures measured at Camp Torkham, in the southeast near the border with Pakistan, in 2009 and 2010; they ranged between 7 and 37°C.

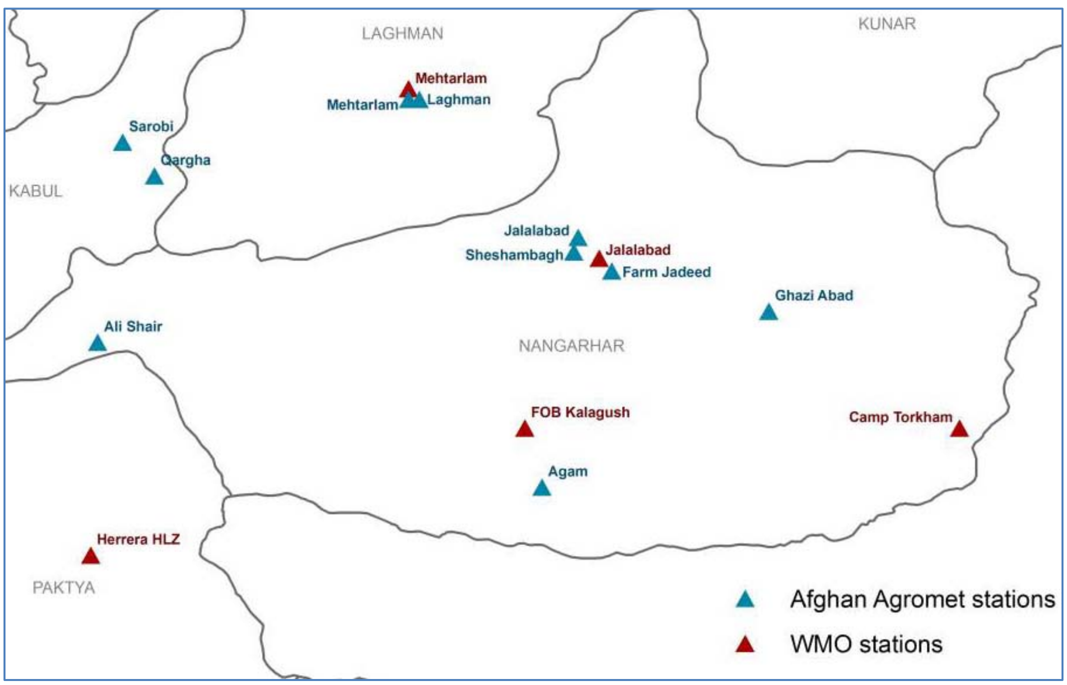


Figure 7. Locations in Nangarhar province where weather data were recorded. Data came from Afghan Meteorological Department Agromet (AMA) and World Meteorological Organization (WMO) stations.

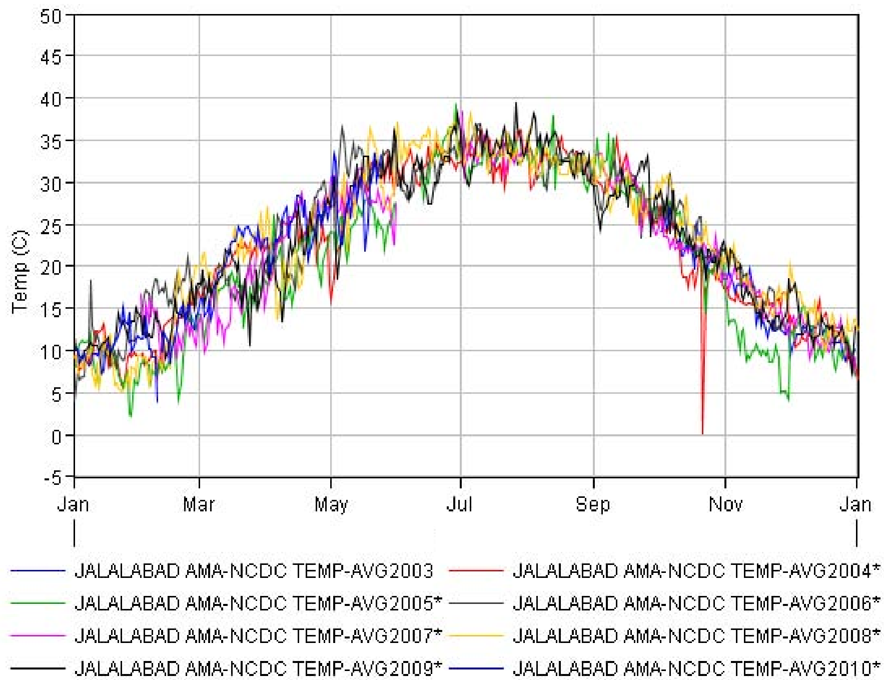


Figure 8. Average daily temperature (NCDC) in Jalalabad in central Nangarhar from 2003 to 2010.

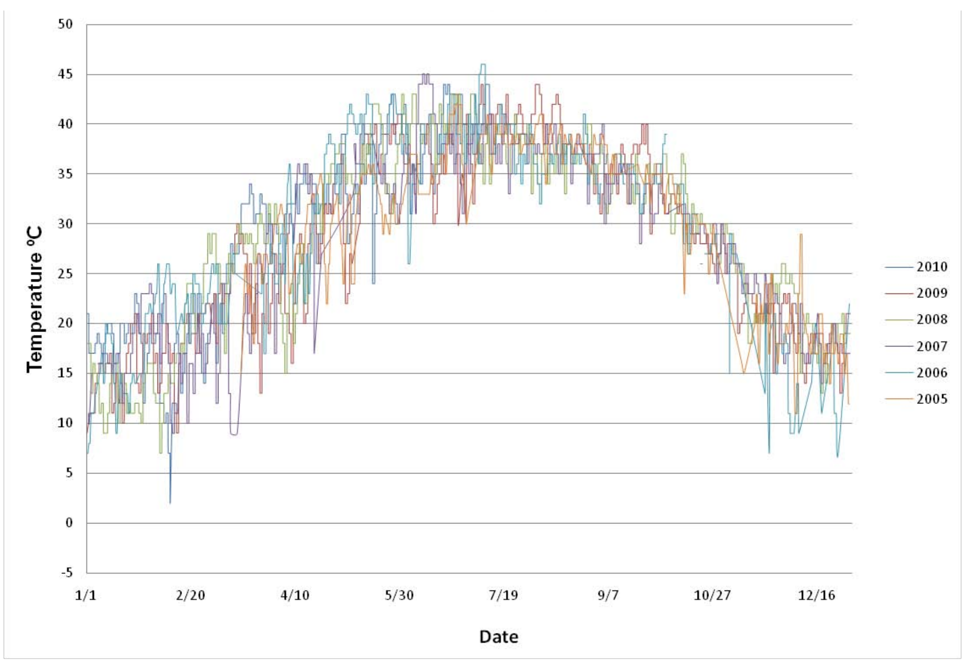


Figure 9. Maximum temperature (METAR) in Jalalabad from 2005 to 2010.

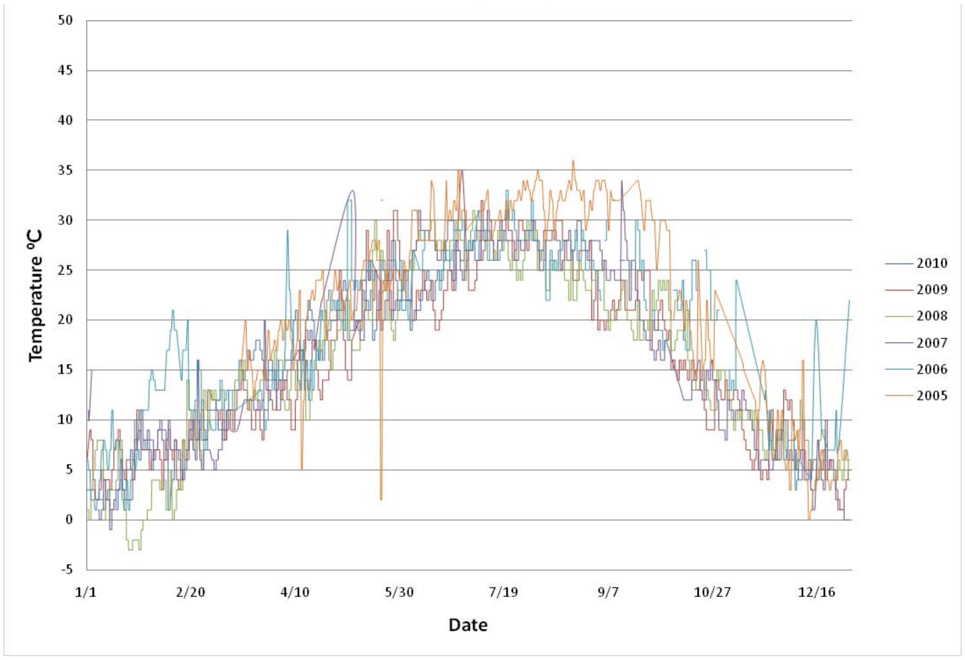


Figure 10. Minimum temperature (METAR) in Jalalabad from 2005 to 2010.

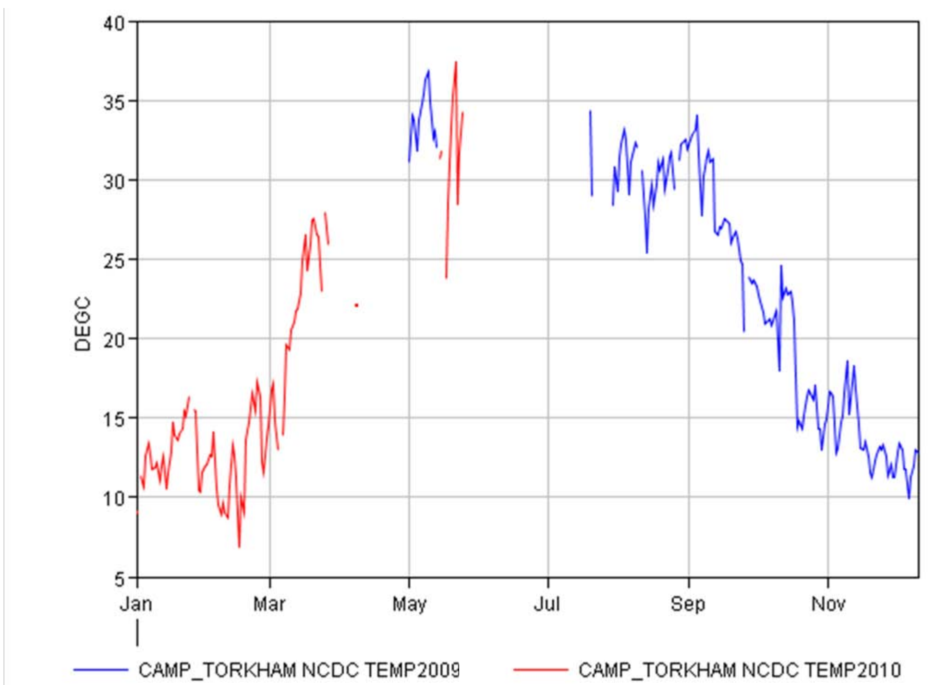


Figure 11. Average daily temperature (from NCDC) for Camp Torkham.

4.2 Precipitation

Daily precipitation data recorded at the weather stations were plotted to determine the amount of precipitation the area receives that may influence designs. Plots of daily precipitation were generated for Jalalabad in 2006 and 2008–2010 (Fig. 12); Camp Torkham in 2009–2010 (Fig. 13); Ghazi Abad in 2003–2009 (Fig. 14); and Agam in 2003–2009 (Fig. 15). Figure 16 shows the annual precipitation in Jalalabad in 2006–2009. The highest daily precipitation in Jalalabad was about 10–19 mm/day, and typically occurred in late spring of each year; most of the rain fell between January and June each year. The average annual precipitation was about 140 mm/yr, as seen in Figure 16. The camp Torkham data were sparse, including fewer years, with precipitation concentrated in February, though the largest single day of precipitation was in October. The rain in Ghazi Abad was primarily in February through mid-April. The largest rainfall occurred in February at 80 mm; the next largest rainfall days were at about 25–30 mm, one of these in a second rainy period in September and October. The rainfall data from Agam were plotted differently from the other sites. Rainfall in Agam was also concentrated in the early part of the year, and the peak precipitation amount was 55 mm, with a few other rains hitting the 25–20 mm mark.

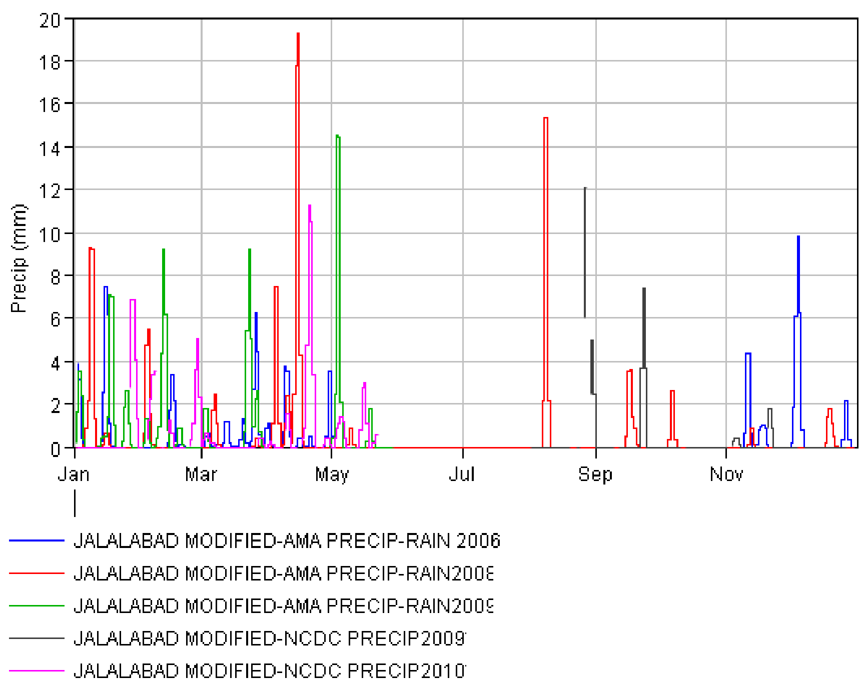


Figure 12. Daily precipitation observed in Jalalabad from 2006 and 2008 to 2010.

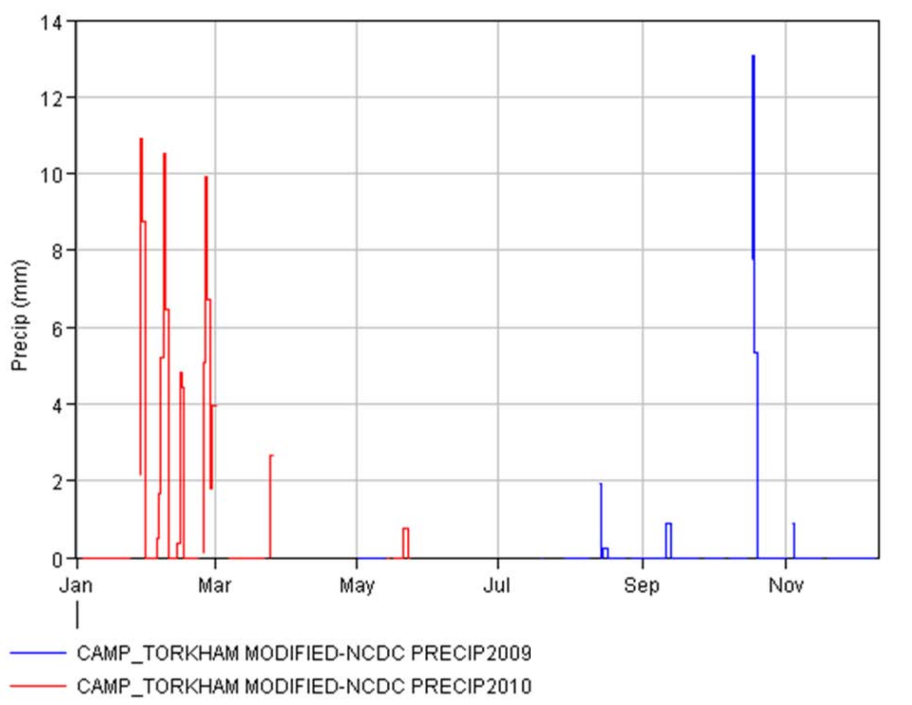


Figure 13. Daily precipitation observed in Camp Torkham from 2009 to 2010.

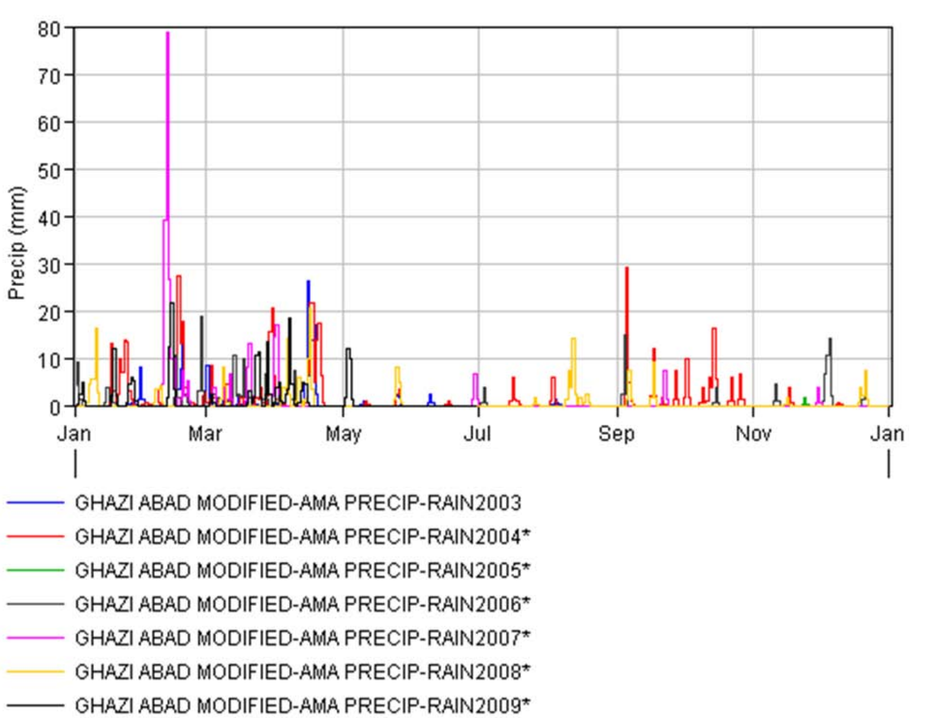


Figure 14. Precipitation observed in Ghazi Abad from 2003 to 2009.

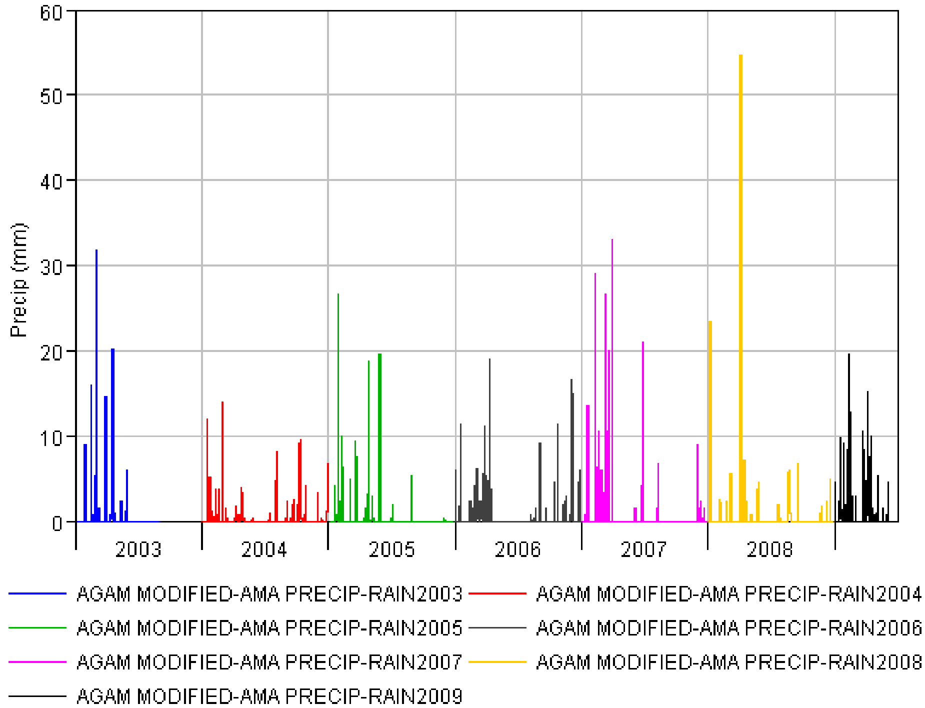


Figure 15. Precipitation observed in Agam from 2003 to 2009.

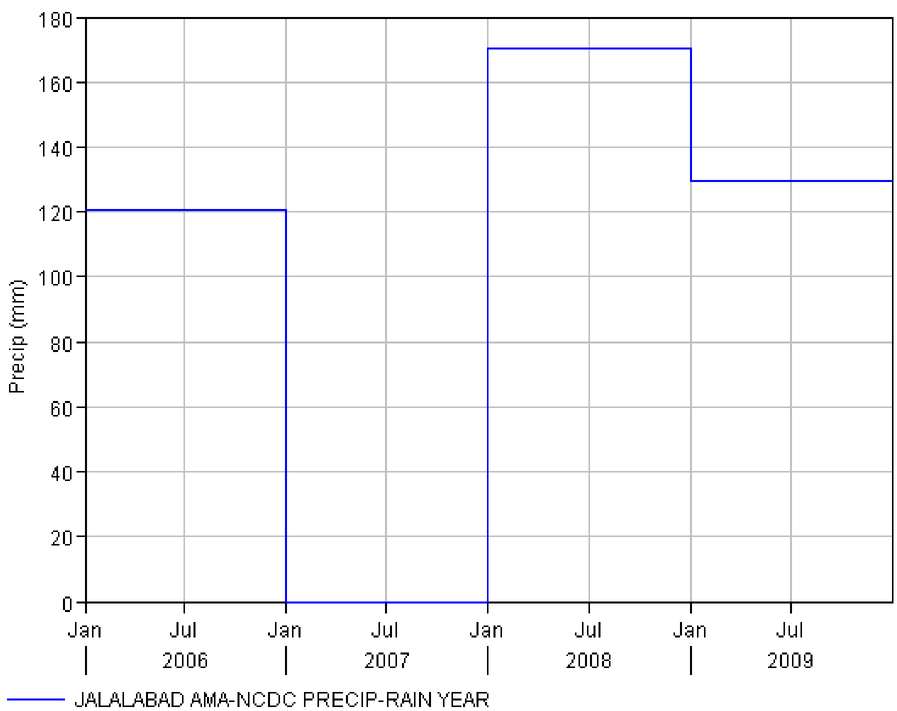


Figure 16. Precipitation accumulation observed in Jalalabad from 2006 to 2009.

4.3 Snow Cover

A snow assessment was conducted for Afghanistan on 1 February 2010. During this assessment, we found that snow-covered areas were primarily in the south and southwest of Nangarhar Province (Fig. 17). When snow fell in lower elevations of Nangarhar Province, it melted quickly, as the daily temperature was rarely below freezing (Fig. 8–11). However, snow is likely to stay in higher elevations, including most of the southern area, part of the west, and the north of the Province.

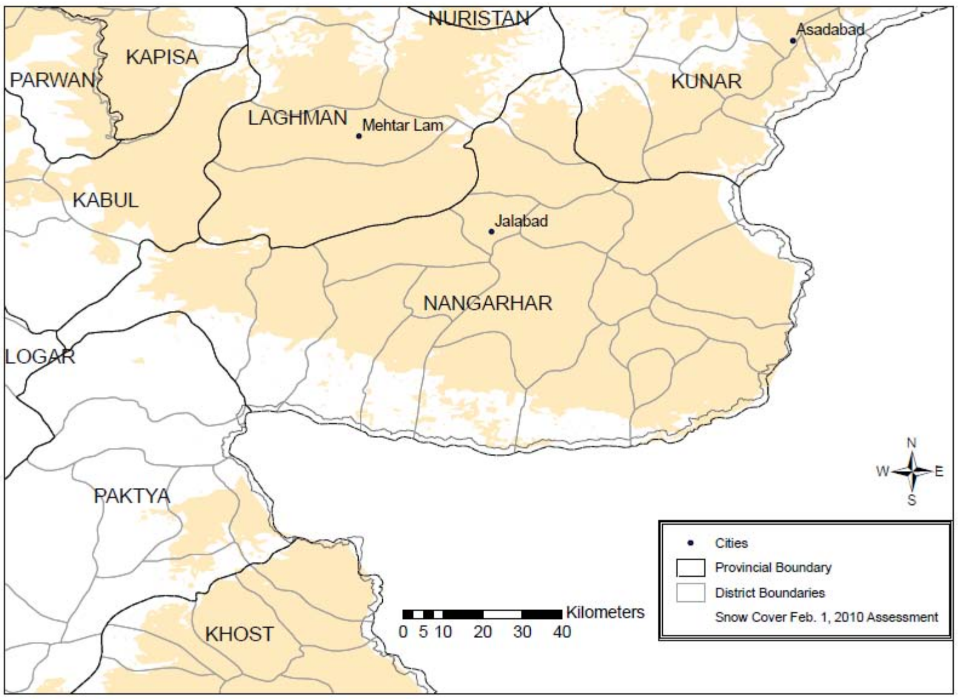


Figure 17. Snow cover (white background) in Nangarhar Province from a snow cover map of Afghanistan for 1 February 2010.

4.4 Summary of geologic and soil characteristics, and climatic effects on design parameters

The geological materials of Nangarhar Province can provide an abundant source of reasonable and competent gravel for construction. The common soils in the province are likely silty gravel with sand (GM), silty sand with gravel (SM) and sandy silt (ML) at 1- and 1.5-m depths. However, poorly graded gravel with sand (GP) and sandy silty clay with gravel (CL-ML) are found at depth. The soils in the mountains are composed of eroded bedrock or exposed rock. Seismic faults are found in places. However, the activity of these faults is not known and this uncertainty can be a problem.

Even though climatic factors within the region play a significant role in the performance of infrastructure, temperatures don't particularly influence design parameters because it is hot in the summer months and the winter is very mild at the lower elevations. Precipitation occurs throughout the year (i.e., rain and possibly snow in the winter, especially at higher elevations). Soil erosion may occur in the low-lying areas, cause by severe runoff during rainfall and snowmelt in the spring. It is important to incorporate drainage into design and construction criteria in these areas.

5 Suggested Design and Construction Parameters for the Nangarhar Region

With the reconstruction in Afghanistan for the past decade, in many cases the data needed for design are not available or are very limited. In most cases, engineers assume values for parameters based on personal experience, usually in their home countries. This chapter provides soil parameters useful for the design and construction of pavements and retaining structures in the Nangarhar region. These data were compiled by determining the parameters required for the most commonly employed engineering practices and designs used for pavements and retaining structures. Geospatial data presented elsewhere in this report were used, along with the literature, to provide values for these parameters that would apply to this region. Although these data will be presented in the context of designing pavements and retaining walls, they could be used for other applications as appropriate. The reader should understand that actual data from the site, either published or obtained by experiment, are to be preferred over the values in this report. Therefore, the values published here can be used as a reference to verify the quality of the data obtained in the field or laboratory.

Table 4. Approximate equivalence between AASHTO and Unified Soil Classification systems (Courtesy of USDA). (Gray shaded rows are common soils found in Nangarhar Province.)

AASHTO	Unified Soil Classification System
A-2-6	GC, SC
A-2-7	GC, SC
A-3	SP
A-4	ML, OL
A-5	MH
A-6	CL
A-7-5	CL, OL
A-7-6	CH, OH

In this chapter, the materials data for soils in the Nangarhar Province have been collected for ease of reference. Table 4 provides a correspondence between the USCS and AASHTO soil classification systems so that some of these Jalalabad soils can be reclassified into the AASHTO system.

Table 5 provides some general ranges of modulus of elasticity and Poisson's ratio for basic types of soil. These are some additional general data that may be useful and include soil types found in Nangarhar.

Table 5 provides some general ranges of modulus of elasticity and Poisson's ratio for basic types of soil (from Braja 2005).

Type of soil	Modulus of elasticity, E_s, (MN/m^2)	Poisson's ratio, μ_s
Loose sand	10–25	0.20–0.40
Medium dense sand	15–30	0.25–0.40
Dense sand	35–55	0.30–0.45
Silty sand	10–20	0.20–0.40
Sand and gravel	70–170	0.15–0.35
Soft clay	4–20	0.20–0.50
Medium clay	20–40	0.20–0.50
Stiff clay	40–100	0.20–0.50

Table 6 lists typical values of the angle of friction for soils at rock interfaces, while Table 7 lists representative values of the angle of internal friction for cohesionless soils. These values can be used to compute the earth pressure coefficients for retaining walls. Table 8 shows typical values of cohesion of clay. Shaded rows indicate types more common in Nangarhar.

Table 6. Typical values of angle of friction (from Braja 2005). (Gray shaded rows are common rocks found in Nangarhar Province).

Rock type	Angle of friction, φ' (deg)
Sandstone	27–45
Limestone	30–40
Shale	10–20
Granite	40–50
Marble	25–30

Table 7. Representative values of the angle of internal friction for cohesionless soils (from Derucher et al. 1998). (Gray shaded rows are likely to be found in Nangarhar Province).

Soil type	φ (deg)
Gravel and sand-gravel mixture	34–38
Sands (well graded)	32–35
Sands (Poorly graded)	29–33
Sand-silt mixture	26–32
Nonplastic silt	25–30

Table 8. Representative values of cohesion, *c*, for any type of clay. (from Derucher et al. 1998). Any of these clays may be found in Nangarhar Province.

Clay condition	*c* (kPa)	*c* (psf)
Soft	12–24	250–500
Medium	24–48	500–1000
Stiff	48–96	1000–2000
Very stiff	96–192	2000–4000
Hard	>192	> 4000

Table 9. Correlation between resilient modulus (M_R) and *CBR* and *R*-value for unbound granular materials (after AASHTO 1993).

θ (psi)[1]	M_R (psi)
100	740 x *CBR* or 1000 + 780 x (*R*-value)
30	440 x *CBR* or 1000 + 450 x (*R*-value)
20	340 x *CBR* or 1000 + 350 x (*R*-value)
10	250 x *CBR* or 1000 + 250 x (*R*-value)
θ (kPa)[1]	M_R (MPa)
689	5.10 x *CBR* or 6.89 + 5.38 x (*R*-value)
207	3.03 x *CBR* or 6.89 + 3.10 x (*R*-value)
138	2.34 x *CBR* or 6.89 + 2.41 x (R-value)
68.9	*CBR* or 6.89 + 1.72 x (*R*-value)

[1] θ = sum of the principal stresses, $\sigma_1 + \sigma_2 + \sigma_3$; referring to AASHTO T274, this corresponds to $\sigma_d + 3\sigma_3$ when $\sigma_d = \sigma_1 - \sigma_3$.

Table 10. Typical M_R values for unbound granular and subgrade materials at unsoaked optimum moisture content and density conditions (after NCHRP 2004). Entries highlighted in gray are soils commonly found in Nangarhar Province.

Materials Classification		M_R Range*		Typical M_R	
		(ksi)	(MPa)	(ksi)	(MPa)
AASHTO Soil Class	A-1-a	38.5–42	265–290	40	276
	A-1-b	35.5–40	245–276	38	262
	A-2-4	28–37.5	193–259	32	221
	A-2-5	24–33	165–228	28	193
	A-2-6	21.5–31	148–214	26	179
	A-2-7	21.5–28	148–193	24	165
	A-3	24.5–35.5	169–245	29	200
	A-4	21.5–29	148–200	24	165
	A-5	17–25.5	117–176	20	138
	A-6	13.5–24	93–165	17	117
	A-7-5	8–17.5	55–121	12	83
	A-7-6	5–13.5	34–93	8	55
USCS Soil Class	GW	39.5–42	272–290	41	283
	GP	35.5–40	245–276	38	262
	GM	33–42	228–290	38.5	265
	GC	24–37.5	165–259	31	214
	GW-GM	35.5–40.5	245–279	38.5	265
	GP-GM	31–40	214–276	36	248
	GW-GC	28–40	193–276	34.5	238
	GP-GC	28–39	193–269	34	234
	SW	28–37.5	193–259	32	221
	SP	24–33	165–228	28	193
	SM	28–37.5	193–259	32	221
	SC	21.5–28	148–193	24	165
	SW-SM	24–33	165–228	28	193
	SP-SM	24–33	165–228	28	193
	SW-SC	21.5–31	148–214	25.5	176
	SP-SC	21.5–31	148–214	25.5	176
	ML	17–25.5	117–176	20	138
	CL	13.5–24	93–165	17	117
	MH	8–17.5	55–121	11.5	79
	CH	5–13.5	34–93	8	55

Table 9 provides a series of relationships that can be used to estimate resilient modulus given an empirically determined California Bearing Ratio (CBR) or R-value for unbound granular materials. Table 10 provides typical values and ranges for resilient modulus (M_R) for various soil types.

6 Overview of Design and Construction Procedures for Roads

Technical information and resources can be found at USACE, Afghanistan Engineer District-North (AEN) and Afghanistan Engineer District-South (AES) websites. For example, the AEN website provides a background of very fundamental information on design and construction with generic information for roads:

(http://www.aed.usace.army.mil/Design.asp).

This report, however, provides more guidance for the design and construction of pavements and retaining structures in the Nangarhar Province with information specific to the region than any existing resources.

6.1 Flexible Pavement

6.1.1 Overview

By definition, the surface course of a flexible pavement remains in contact with the base course. Flexible pavements are typically composed of an asphalt concrete surface placed on base and subbase courses supported by prepared roadbed, which is referred to as subgrade. Each underlying layer is designed in a manner to provide support for the top layer, which in turn is built to accommodate a design load.

6.1.1.1 Surface layer

The surface course is the top layer of the flexible pavement structure, which consists of a mixture of mineral aggregates and bituminous materials, referred to as asphalt concrete or hot mix asphalt (HMA). The surface course is generally constructed on a base course, but some flexible pavements will have multiple asphalt concrete courses or sublayers. The surface course is in contact with traffic and provides structural support for the load. In addition, this layer must resist the abrasive force of traffic, reduce the amount of water penetrating through the surface of the pavement, provide skid resistance, and serve as a smooth and uniform riding surface for the traffic.

6.1.1.2 Base layer

This layer is constructed directly below the surface course and may consist of either treated or untreated aggregates, such as crushed stones, crushed slag, crushed gravel, sand, or a combination. The base course is constructed on the subbase course or directly on the subgrade, depending on whether the pavement requires a subbase course or not. The main function of this layer is to provide structural support.

Untreated aggregate base must be compacted to at least 95% of maximum laboratory density or higher based on AASHTO test T180, Method D (AASHTO 1993), or equivalent. In the case of treated aggregate base, suitable stabilizing materials, such as Portland cement, asphalt, lime, cement-fly ash, or lime-fly ash, are mixed with aggregate. Consideration should be given to selecting suitable treated materials and the amount to be used in a base course and they should be economically feasible, especially when they are in short supply.

6.1.1.3 Subbase layer

The subbase course is constructed on the top of subgrade and underlies the base course. It consists of a compacted layer of either treated or untreated granular material or of a layer of treated soil. Depending on the strength of subgrade soil, the subbase layer may be omitted to save money.

When subgrade soil is of relatively low strength and the structural design indicates that a considerable thickness of pavement is required, several alternative structural designs with and without subbase may be proposed. The basis for selecting the best alternative will then be the availability and relative costs of the materials required. As the materials used in a subbase course are generally of relatively lower quality and cheaper than those used in a base course, the use of a subbase course with a thinner base or surface course is often a more economical approach for constructing a flexible pavements on a poor subgrade.

Untreated aggregate subbase must be compacted to 95% of maximum laboratory density or higher based on AASHTO Test T180, Method D, or equivalent. Besides serving as structural support for the pavement, the subbase course can also help prevent fine-grained subgrade soils from entering the base course; reduce the damages caused by frost; prevent the accumulation of water within or below the pavement structure; and provide a platform to carry heavy equipment during construction.

6.1.1.4 Subgrade

The subgrade is the in-situ layer of roadbed soil or borrowed material that has been compacted to a specified density. The most important material property used to characterize subgrade soils for pavement design is the resilient modulus (M_R).

The resilient modulus is a measure of the modulus of elasticity (E) of the subgrade soil under rapidly applied loads. The resilient modulus can be used directly for the design of flexible pavements, but it must be converted to a modulus of subgrade reaction (k-value) for the design of rigid or composite pavements.

6.1.2 Flexible pavement design procedure

6.1.2.1 Steps

The design process involves selecting the materials for each layer and determining the thickness of each layer for the road to bear the amount of traffic anticipated during its design life. The thickness of each layer in a flexible pavement structure designed to accommodate a specific number of Equivalent Single-Axle Loads (*ESALs*) during its lifetime can be determined using the following procedure:

Step 1. Starting with the number of years the road is to last before rebuilding, called the design life in years, determine the number of ESALs of heavy truck traffic the road will carry. Light passenger vehicles such as automobiles and motorcycles are assumed to cause negligible wear.
Step 2. Determine the resilient modulus (M_R) of the subgrade based on available soil data, such as California Bearing Ratio (*CBR*) or R-value.
Step 3. Select reliability (R) and then the corresponding standard normal deviate Z_R.
Step 4. Select the initial and final Present Serviceability Index and determine their difference, ΔPSI.
Step 5. Determine overall structural number (SN) by solving the structure number equation either numerically or by using a nomograph.
Step 6. Determine the structural numbers for each separate layer and then the thickness for each layer.

6.1.2.2 Design life

The period of time for which the analysis is to be conducted is called the "design life" or the "analysis period." Table 11 gives some general guidelines for design life based on the category of road.

Table 11. Guidelines for length of design life (AASHTO 1993).

Highway conditions	Design life (years)
High-volume urban	30–50
High-volume rural	20–50
Low-volume paved	15–25
Low-volume aggregate surface	10–20

6.1.2.3 Computing the number of ESALs based on expected traffic

The basis for considering the effect of traffic loads in a design procedure is the cumulative expected 80-kN (18-kip) *ESALs* during the design life—in other words the number of repetitions of each axle group during the design life. To determine the total number of passes of the standard axle load, the mixed axle loads of traffic should be converted to standard *ESALs*. For example, the effect of a single pass of a 53-kN (12-kip) single axle on a pavement is equivalent to 0.23 *ESAL*.

The total number of passes of the standard axle (80-kN or 18-kip) load (*ESAL*) during the design life can be determined using the following equation:

$$ESAL = (ADT)(T)(T_f)(D)(G)(L)(365)(Y) \tag{1}$$

where

ADT = average daily traffic at the start of the design period, in number of vehicles per day
T = percentage of traffic in the ADT consisting of trucks
T_f = truck factor
G = growth factor
D = direction distribution (0.5 if amount of traffic is the same in each direction)
L = lane distribution factor
Y = design life, or analysis period, in years.

The *ADT* is the average number of vehicles that pass a specific section of a highway in 24 hours and is usually averaged over a year.

The truck factor is determined on the basis of growth factors. If there is one common growth factor for different types of trucks, then a single truck factor can be used for all trucks. If there are separate growth factors for each class of trucks, then different truck factors will be used for each type. These truck factors can be looked up in tables published by the Asphalt Institute, reproduced in Huang (1993).

The growth factor G is calculated from the following correlation:

$$G = (1 + r)^{0.5Y} \quad (2)$$

where

r = yearly traffic growth rate
Y = design period in years.

The total growth factor G_t for the entire design period is determined using the following equation:

$$G_t = [(1 + r)^Y - 1]/r \quad (3)$$

The lane distribution factor (L) varies, depending on the volume of traffic and the number of lanes in each direction. For a highway with two lanes—one lane in each direction—each lane is the design lane; meaning that the distribution factor is 100%. For highways with two or more lanes in each direction, the design lane is the outside or driving lane.

6.1.2.4 Reliability

Reliability is the degree of certainty, expressed as a percentage, that the pavement will maintain an acceptable level of serviceability throughout its design life (Table 12). Serviceability is defined in the next section.

Table 12. Suggested levels of reliability for various functional classifications (AASHTO 1993).

Functional Classification	Recommended level of reliability *R*, in percent	
	Urban	Rural
Interstate and other freeways	85.0–99.9	80.0–99.9
Principal arterials	80.0–99.0	75.0–95.0
Collectors	80.0–95.0	75.0–95.0
Local	50.0–80.0	50.0–80.0

The reliability factor R is a function of the overall standard deviation (S_o), which considers chances of variation in the traffic prediction and also in pavement performance prediction. The S_o values for flexible pavements range from 0.40 to 0.50; a typical value is 0.45. The overall standard deviation (S_o) can be calculated from the following correlation:

$$S_o = (\log W_{18} - \log W_{t18})/Z_R \tag{4}$$

where

W_{18} = allowable 18-kip (80-kN) single-axle loads for a given reliability

W_{t18} = number of 18-kips (80-kN) single-axle loads at end of time t

Z_R = standard normal deviate.

The standard normal deviate (Z_R) for a given reliability can be selected from Table 13. The reliability and standard normal deviate are used to help determine the structure number.

Table 13. Standard normal deviate (Z_R) Values corresponding to selected levels of reliability (AASHTO 1993).

Reliability	Standard normal deviate
(%)	(Z_R)
50	0.000
60	–0.253
70	–0.524
75	–0.674
80	–0.841
85	–1.037
90	–1.282
91	–1.340
92	–1.405
93	–1.476
94	–1.555
95	–1.645
96	–1.751
97	–1.881
98	–2.054
99	–2.327
99.9	–3.090
99.99	–3.750

6.1.2.5 Serviceability

Serviceability is a rating of how well a constructed pavement provides traffic with an acceptable level of service, determined by the smoothness of the pavement. It is represented by the Present Serviceability Index (PSI), a 0 to 5 rating scale with 5 representing a perfect road. The total allowable loss in serviceability of a pavement can be computed using eq 5. A typical value of initial serviceability index for newly constructed flexible pavement is 4.2. For major highways a terminal serviceability index of 2.5 or higher is recommended.

$$\Delta PSI = P_i - P_t \tag{5}$$

where

P_i = initial serviceability index
P_t = terminal serviceability index.

The initial serviceability index (P_i) is actually a *PSI* immediately after the road is opened for operation; whereas the terminal serviceability index (P_t) is the lowest acceptable *PSI* level before resurfacing is needed. The major parameters affecting the serviceability of a pavement include traffic, age, and environmental conditions. Based on the type of highway, the initial and terminal serviceability indices can be selected from the Table 14.

Table 14. Initial and terminal serviceability indices for flexible pavement (after AASHTO, 1993).

Type of highway	Initial serviceability index P_i	Terminal serviceability index P_t
Major highways	4.2	2.5 or 3.0
Highways with lower traffic volume	4.2	2.0
Relatively minor highways	4.2	1.5

6.1.2.6 Resilient modulus

Resilient modulus M_R can be directly determined by the AASHTO Test Method T274. Or it can indirectly be derived from standard *CBR* and *R*-values using eq 6 and 7.

$$M_R\,(\text{MPa}) = 10.34 \times CBR \tag{6}$$

$$M_R\,(\text{psi}) = 1500 \times CBR$$

Equation 6 is considered reasonable for fine-grained soil with a soaked *CBR* of 10 or less.

$$M_R = A + B \times (R\text{-value}) \tag{7}$$

where

A = 4.98 to 7.96 MPa or 722 to 1155 psi
B = 2.54 to 3.83 MPa or 369 to 555 psi.

For estimating resilient modulus of fine-grained soils (where R-value is 20 or less) the following correlation in eq 8 should be used.

$$M_R \text{ (MPa)} = 6.89 + 3.83 \times (R\text{-value}) \quad (8)$$

$$M_R \text{ (psi)} = 1000 + 555 \times (R\text{-value})$$

To estimate resilient modulus (M_R) for unbound granular materials used in base and subbase for which the CBR or R-values are available, the correlations given previously in Table 9 can be used. If test data are not available or are suspect, default values of M_R for different classes of soils presented in Table 10 may be consulted.

6.1.2.7 Computation of required pavement thickness using structural number (SN)

The structural number (SN) is a function of structural layer coefficients (a_i), layer thicknesses (D_i), and layer drainage coefficients (m_i, except for surface) and can be calculated from the following correlation:

$$SN = \sum_{i=1}^{n} sN_i = \sum_{i=1}^{n} a_i D_i m_i = a_1 D_1 + a_2 D_2 m_2 + a_3 D_3 m_3 \quad (9)$$

where the subscripts are assigned starting with 1 for the surface course, as in Figure 18.

a_1 = structural layer coefficient for the surface course
a_2 = structural layer coefficient for the base course
a_3 = structural layer coefficient subbase course
D_1 = thicknesses of the surface course
D_2 = thicknesses of the base course
D_3 = thicknesses of the subbase course
m_1 = 1 = the layer drainage coefficient for the surface course
m_2 = layer drainage coefficient for the base course
m_3 = layer drainage coefficient for the subbase course.

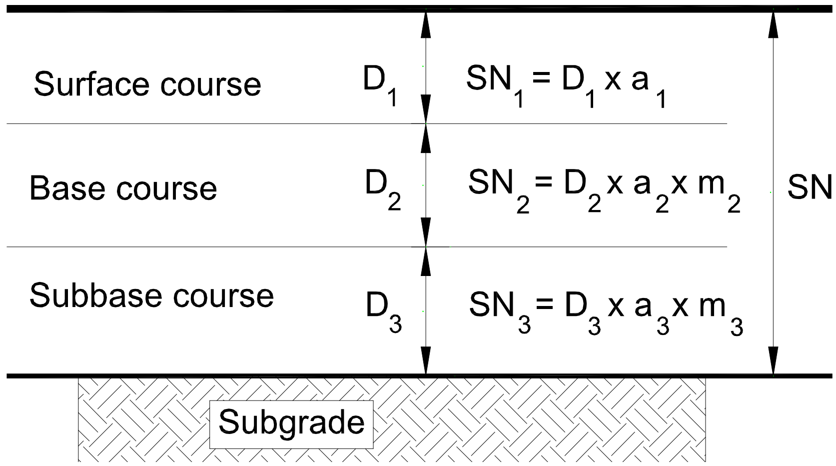


Figure 18. Pavement layer variables.

The structural layer coefficients (a_i) are used to convert actual layer thickness to structural number (SN).

The layer drainage coefficients (m_i) are considered in the design procedure to assure drainage the structure of the pavement. Different levels of drainage are defined in Table 15.

Table 15. General definitions of corresponding to different drainage levels from the pavement structure (AASHTO 1993).

Quality of drainage	Water removed within
Excellent	2 hours
Good	1 day
Fair	1 week
Poor	1 month
Very poor	water will not drain

One can determine the overall structural number (SN) using eq 10 below:

$$\log(W_{18}) = (Z_R \times S_o) + 9.36\log(SN+1) - 0.2\log[\Delta PSI/(4.2 - 1.5)]/[0.4 + (1094/[SN+1]^{5.19})] + 2.32\log(M_R) - 8.07 \qquad (10)$$

where

W_{18} = allowable 80-kN (18-kip) single-axle load application for a given reliability (same as ESALs earlier)

Z_R = normal deviate for a given reliability
S_o = overall standard deviation
SN = design structural number
ΔPSI = total changes, or loss, in serviceability P of a pavement
M_R = resilient modulus of subgrade materials in psi.

This equation does not have a closed form solution, so it is generally solved using a computer model, an iteration process, or a nomograph. An example showing the use of the nomograph is given in Appendix A. Then one determines the structural numbers for each separate layer and then the thickness for each layer.

6.2 Rigid Pavement Design procedure

Rigid pavements typically consist of Portland cement concrete pavement slabs that are cast in place on a granular base course overlaying the subgrade soil. The rigid pavement design is based on the following assumptions: the rigid pavement slab is homogeneous, isotropic, and elastic; and the subgrade acts as linear spring, which exerts a vertical reactive pressure proportional to the deflection of the slab. In contrast to flexible pavements, after curing, rigid pavements will not necessarily stay in contact with the base course.

The design procedure for rigid pavements is similar to that for flexible pavements, with some variations due to the differences in the materials used. The overall steps are the same, namely:

Step 1. Starting with the number of years the road is to last before rebuilding, called the design life in years, determine the number of ESALs of heavy truck traffic that the road will carry. Light passenger vehicles such as automobiles and motorcycles are assumed to cause negligible wear.
Step 2. Select reliability (R) and then the corresponding standard normal deviate Z_R.
Step 3. Select the initial and final PSI and determine their difference, ΔPSI.
Step 4. Determine the modulus of subgrade reaction (k) based on available soil data, such as California Bearing Ratio (CBR) or R-value.
Step 5. Determine overall SN by solving the structure number equation either numerically or by using a nomograph.

Step 6. Determine the structural numbers for each separate layer and then the thickness for each layer.

6.2.1 Design life or analysis period

The design life or analysis period for rigid pavement is the same as for flexible pavement.

6.2.2 Traffic consideration

The general procedure for determining the cumulative expected 80-kN (18-kip) equivalent single axle loads (*ESALs*) during the design life for rigid pavement is the same as for flexible pavement. Note that the truck factor (T_f) will be different for rigid pavements than for flexible pavements; refer to AASHTO (1993) to determine the truck factor.

6.2.3 Reliability

The values of S_o for rigid pavements range between 0.3 and 0.45, with a typical value of 0.35 used for design. The reliability R and standard normal deviate Z_R, can be determined in the same manner as for flexible pavements.

6.2.4 Serviceability

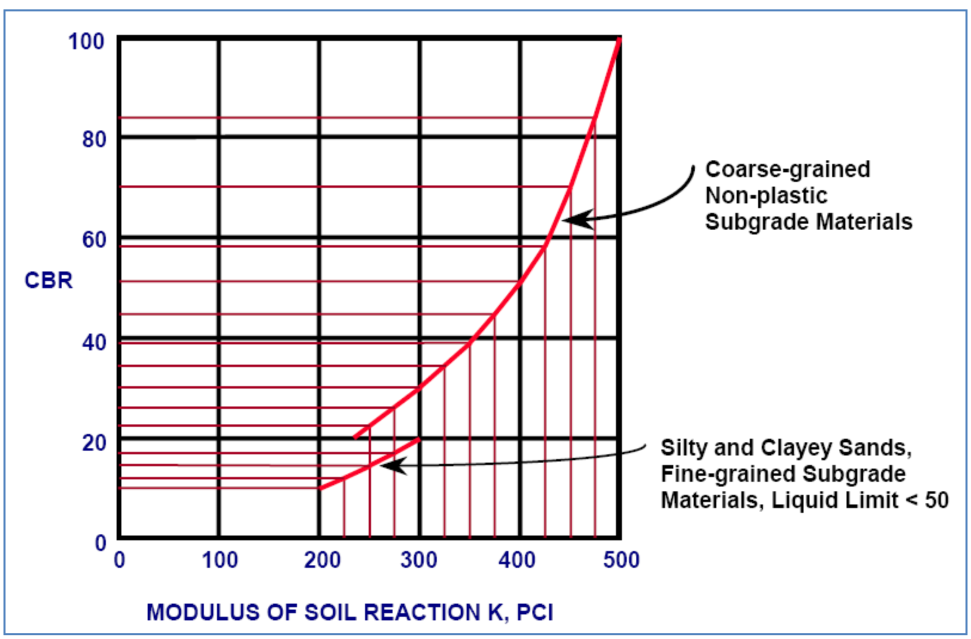
The typical value of the initial serviceability index P_i for rigid pavement is 4.4. The terminal serviceability index for major highways is recommended at 2.5. The procedure to determine the overall change in serviceability index for rigid pavements is the same as that for flexible pavements.

6.2.5 Modulus of subgrade reaction

The modulus of subgrade reaction (k) is a function of subgrade resilient modulus, thickness of granular subbase, resilient modulus of granular subbase, depth of bedrock (if shallower than 10 ft or 3 m), and loss of service (an index of the erodibility of the granular subbase). Table 16 provides typical k-values for different soil types and moisture content (AFCESA, ETL 02-19, 2002). *CBR* of soil can be converted to k-values. Figure 19 shows correlations of *CBR* and k-values.

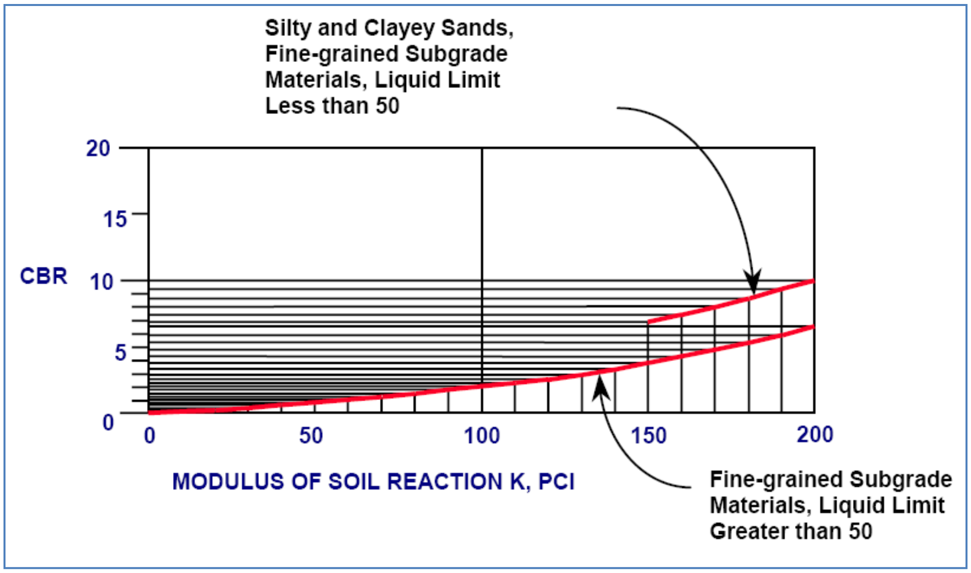
Table 16. Typical values for modulus of soil reaction, *k* (gray highlight indicates common soils in Nangarhar Province).

Soil Type	Modulus of reaction, *k* for range of moisture content							
	1–4	5–8	9–12	13–16	17–20	21–24	25–28	Over 28
Silts & clays LL > 50 (OH, CH, MH)	–	175	150	125	100	75	50	25
Silts & clays LL , 50 (OL, CL, ML)	–	200	175	150	125	100	75	50
Silty & clayey sands (SM & SC)	300	250	225	200	150	–	–	–
Sand & gravelly sands (SW & SP)	350	300	250	–	–	–	–	–
Silty & clayey gravels (GM & GC)	400	350	300	250	–	–	–	–
Notes: Values of *k* shown are typical for materials having dry densities of 90 to 95% of the maximum.								



(a)

Figure 19. Correlation of CBR to modulus of soil reaction, *k* (after AFCESA, ETL 02-19, 2002).



(b)

Figure 19 (cont'd). Correlation of CBR to modulus of soil reaction, *k* (after AFCESA, ETL 02-19, 2002).

6.2.6 Other design inputs

The overall structural number (*SN*) obeys the following equation:

$$\log(W_{18}) = (Z_R \times S_o) + 7.35\log(D+1) - 0.06 + \log[\Delta PSI/(4.5 - 1.5)]/[1 + (1.624 \times 107/[D + 1]8.46)] + (4.22 - 0.32p_t) \times \log\{[S_c C_d\ (D^{0.75} - 1.132)]/215.63J[D^{0.75} - (18.42 / [E_c / k]^{0.25})]\} \quad (11)$$

where

- W_{18} = allowable 18-kip single-axle load application for a given reliability
- Z_R = normal deviate for a given reliability
- S_o = overall standard deviation
- ΔPSI = total changes, or loss, in serviceability of a pavement
- P_t = terminal serviceability index
- MR = resilient modulus of subgrade materials in psi
- k = modulus of subgrade reaction in pci
- J = empirical joint load transfer coefficient
- E_c = PCC modulus of elasticity in psi
- S_c = PCC modulus of rupture in psi
- C_d = an empirical drainage coefficient
- D = required PCC slab thickness in inches.

There are several design inputs not previously defined. The empirical joint load transfer coefficient (J) is a function of the load transfer condition between the pavement slab and shoulders as well as the shoulder type. Mean values for the standard material properties of Portland cement concrete, modulus of rupture (S_c) and modulus of elasticity (E_c), should be used. The subbase drainage coefficient (C_d) for rigid pavements ranges from 0.70 to 1.25 based on level of the quality of drainage.

As with the design equation for flexible pavements, a closed form solution does not exist, so the equation is commonly solved numerically or through a nomograph, as in the example in Appendix B.

6.3 Unsurfaced or gravel roads

Unsurfaced or gravel roads are common in Nangarhar Province. Dirt roads are also widespread. The base of improved dirt roads is reinforced in places with coarse and fine gravel (East View Cartographic, Inc. 2003). Some of these dirt roads and trails are vulnerable of becoming soggy in areas on silty soils with poor drainage conditions.

6.3.1 Procedures

Design criteria for gravel roads are described in detail by USACE (2004). The method used to determine design thickness of aggregate surfaced roads is similar to the approach used in flexible pavement. In this procedure, a class is assigned to the road being designed based upon the number of vehicles per day. A design category is then assigned to the traffic, from which a design index is determined. This design index is used with Figure 20 to select the thickness (minimum of 4 in. [10 cm]) of aggregate required above a soil with a given strength expressed in terms of *CBR* for non-frost areas or in terms of a frost area soil support index (*FASSI*) in frost areas.

6.3.2 Classes

The classes of aggregate surfaced roads vary from A to G. Selection of the proper class depends upon the traffic intensity (Table 17).

6.3.3 Design index

The design of gravel roads will be based on a design index, which represents all traffic expected to use the road during its life. The design index is based on typical magnitudes and compositions of traffic reduced to equivalents in terms of repetitions of an 18,000-lb (8166-kg)single-axle, dual-wheel load.

Table 17. Criteria for selecting aggregate surface road class.

Road Class	Number of Vehicles per day
A	10,000
B	8,400–10,000
C	6,300–8,400
D	2,100–6,300
E	210–2,100
F	70–210
G	Under 70

Tracked vehicles having gross weights not exceeding 15,000 lb (6804 kg) and forklift trucks having gross weights not exceeding 6000 lb (2722 kg) may be treated as two-axle trucks (Group 2 vehicles, Table 18) in determining the design index. Tracked vehicles having gross weights exceeding 15,000 lb (6804 kg) but not 40,000 lb (18,144 kg) and forklift trucks having gross weights exceeding 6000 lb (2722 kg) but not 10,000 lb (4536 kg) may be treated as Group 3 vehicles.

Table 18. Traffic composition categories.

Category	Traffic Composition
I	Primarily of passenger cars, panel and pickup trucks (Group 1 vehicles), and containing not more than 1% two-axle trucks (Group 2 vehicles).
II	Primarily of passenger cars, panel and pickup trucks (Group 1 vehicles), and containing as much as 10% two-axle trucks (Group 2 vehicles). No trucks having three or more axles (Group 3 vehicles) are permitted in this category.
III	Contains as much as 15% trucks, but with no more than 1% of the total traffic composed of trucks having three or more axles (Group 3 vehicles)
IV	Contains as much as 25% trucks, but with no more than 10% of the total traffic composed of trucks having three or more axles (Group 3 vehicles).
IVA	More than 25% trucks or more than 10% trucks having three or more axles (Group 3 vehicles).

6.3.3.1 Design index for pneumatic-tired vehicles

For designs involving rubber-tired vehicles, traffic is classified in three groups: 1) passenger car, panel and pickup trucks; 2) two-axle trucks; and 3) three-, four-, and five-axle trucks. Table 19 is the design index to be used in designing a gravel road for the usual pneumatic-tired vehicles.

Table 19. Design index for pneumatic-tired vehicles and for traffic of tracked vehicles weighing less than 40,000 lb (18,144 kg), and forklift trucks weighing less than 10,000 lb (4536 kg).

Road Class	Category I	Category II	Category III	Category IV
A	3	4	5	6
B	3	4	5	6
C	3	4	4	6
D	2	3	4	5
E	1	2	3	4
F	1	1	2	3
G	1	1	1	2

6.3.3.2 Design index for tracked vehicles and forklift trucks

Traffic composed of tracked vehicles exceeding 40,000-lb (18,144-kg) gross weight and forklift trucks exceeding 10,000-lb (4536-kg) gross weight has been divided into the three categories given in Table 20. Roads sustaining traffic of tracked vehicles weighing less than 40,000 lb, and forklift trucks weighing less than 10,000 lb, will be designed in accordance with the pertinent class and category from Table 20. Roads sustaining traffic of tracked vehicles heavier than 40,000 lb, and forklift trucks heavier than 10,000 lb, will be designed in accordance with the traffic intensity and category from Table 21.

Table 20. Maximum vehicle gross weight.

Category	Tracked Vehicles, lb (kg)	Forklift Trucks, lb (kg)
V	60,000 (27,216)	15,000 (6804)
VI	90,000 (40,823)	20,000 (9072)
VI	120,000 (54,431)	35,000 (15,876)

Table 21. Design index for pneumatic-tired vehicles.

Traffic Category	Number of vehicles per day (or week as indicated)							
	500	200	100	40	10	4	1	1 per week
V	8	7	6	6	5	5	5	—
VI	—	9	8	8	7	6	6	5
VI	—	-	10	10	9	7	7	6

Table 22. Frost-area soil support indexes of subgrade soils.

Frost Group of subgrade soils	Frost Area Soil Support Index (FASSI)
F1 and S1	9.0
F2 and S2	6.5
F3 and F4	3.5

The life assumed for design is 25 years. For a design life of fewer than 5 years, the design indexes in Tables 21 and 22 may be reduced by one. Design indexes below three should not be reduced.

6.3.4 Thickness criteria

Thickness requirements for aggregate surfaced roads are determined from Figure 20 based on a given soil strength and design index. The minimum thickness requirement will be 4 in. (10 cm) Figure 20 will be entered with the *CBR* of the subgrade to determine the thickness of aggregate required for the appropriate design index. The thickness determined from the figure may be constructed of compacted granular fill for the total depth over the natural subgrade or in a layered system of granular fill (including subbases) and compacted subgrade for the same total depth. The layered section should be checked to ensure that an adequate thickness of material is used to protect the underlying layer based on the CBR of the underlying layer.

Where frost-susceptible subgrades are encountered, the section thickness required will be determined according to the reduced subgrade strength method. Based on the frost design soil classification (Fig. 21), the reduced subgrade strength method requires the use of frost area soil support indexes listed in Table 21. Frost-area soil support indexes (FASSI) are used as if they were *CBR* values; FASSI numbers are weighted average values for an annual cycle, and their values cannot be determined by *CBR* tests.

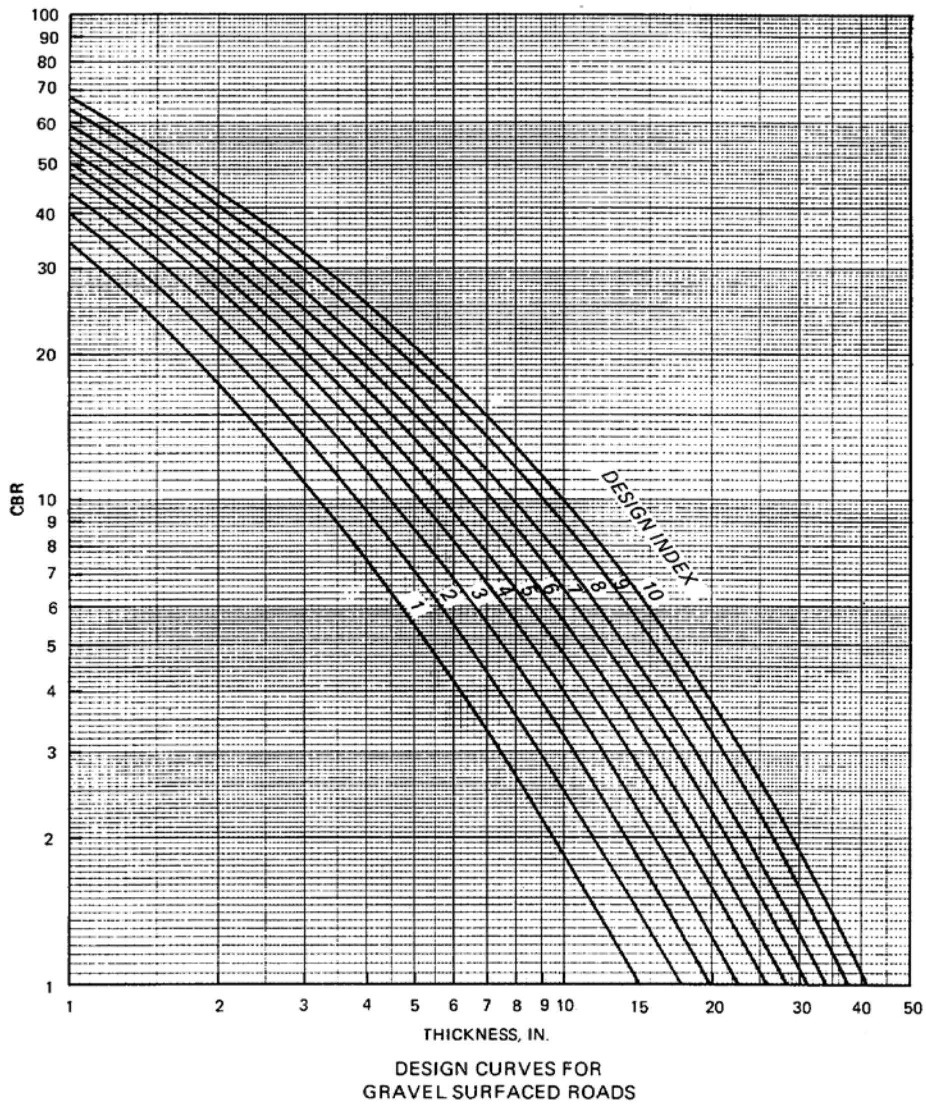


Figure 20. Thickness design curves for aggregate surfaced roads (1 in. = 2.54 cm) (USACE 2004).

Frost Group	Kind of Soil	Percentage Finer Than 0.02 mm by Weight	Typical Soil Types Under Unified Soil Classification System
NFS*	(a) Gravels Crushed stone Crushed rock	0-1.5	GW, GP
	(b) Sands	0-3	SW, SP
PFS**	(a) Gravels Crushed stone Crushed rock	1.5-3	GW, GP
	(b) Sands	3-10	SW, SP
S1	Gravelly soils	3-6	GW, GP, GW-GM, GP-GM
S2	Sandy soils	3-6	SW, SP, SW-SM, SP-SM
F1	Gravelly soils	6 to 10	GM, GW-GM, GP-GM
F2	(a) Gravelly soils	10 to 20	GM, GW-GM, GP-GM
	(b) Sands	6 to 15	SM, SW-SM, SP-SM
F3	(a) Gravelly soils	Over 20	GM, GC
	(b) Sands, except very fine silty sands	Over 15	SM, SC
	(c) Clays, PI > 12	--	CL, CH
F4	(a) All silts	--	ML, MH
	(b) Very fine silty sands	Over 15	SM
	(c) Clays, PI < 12	--	CL, CL-ML
	(d) Varved clays and other fine-grained banded sediments	--	CL and ML CL, ML, and SM CL, CH, and ML CL, CH, ML and SM

*Nonfrost-susceptible.
**Possibly frost-susceptible, but requires laboratory test to determine frost design soil classification.

Figure 21. Frost design soil classification.

6.3.5 Wearing surface requirements

The requirements for the various materials to be used in the construction of aggregate surfaced roads depend upon whether or not frost is a consideration in the design.

6.3.5.1 Non-frost areas

The material used for gravel surfaced roads should be sufficiently cohesive to resist abrasive action. It should have a liquid limit no greater than 35 and a plasticity index of 4 to 9. It should also be graded for maximum density and minimum volume of voids to enhance optimum moisture retention while resisting excessive water intrusion. The gradation, therefore, should consist of the optimum combination of coarse and fine aggregates that will ensure minimum void ratios and maximum density. Such a material will then exhibit cohesive strength as well as intergranular shear strength. Recommended gradations are as shown in Table 23. If the fine fraction of the material does not meet plasticity characteristics, modification by addition of chemicals might be required. Chloride products can, in some cases, enhance moisture retention, and lime can be used to reduce excessive plasticity.

6.3.5.2 Frost areas

The percentage of fines should be restricted in all the layers to facilitate drainage and reduce the loss of stability and strength during thaw periods.

Gradation numbers 3 and 4 shown in Table 23 should be used with caution as they may be unstable in a freeze–thaw environment.

Table 23. Gradation for aggregate surface courses.

Sieve Designation		No. 1	No. 2	No. 3	No. 4
25.0mm	1 in.	100	100	100	100
9.5 mm	3/8 in.	5-85	60-100	-	-
4.7 mm	No. 4	35-65	50-85	55-100	70-100
2.00 mm	No. 10	25-50	40-70	40-100	55-100
0.425 mm	No. 40	15-30	24-45	20-50	30-70
0.075 mm	No. 200	8-15	8-15	8-15	8-15

Note: The percent by weight finer than 0.02 mm shall not exceed 3%.

6.3.6 Construction practice for gravel roads

Construction practices are considered where gravel roads are common. Crowns of gravel roads should be ½ to ¾ in. (1.8 to 1.9 cm) for each foot (0.3 m) of road width from the center of the road (Vermont Better Backroads 2008). Thus, slope roads with over-the-bank drainage toward the ditched side of the road, if necessary. Another suggestion is indicated resurfacing the road every 4 to 5 years with 2–3 in. (5–8 cm) of new gravel.

7 Slopes and Retaining Wall Construction

7.1 Slope stability

As facilities and infrastructure are built in mountainous areas, potential slope failures should be recognized. Slope stability problems have several characteristics: natural soil slope, natural rock slope, cut slope, open excavation, earth dam embankment, embankment of over soft soils, and hillside fill. The only solution to slope failures is to recognize that their potential exists. Experts have emphasized that often slopes fail when deformations exist. Extensive information to analyze various slope stability approaches are available (Chen 1995). Retaining structures are used if slope stability is uncertain. Various types of retaining walls include: roc-filled buttress; gabion wall; crib wall; reinforced earth wall; concrete-reinforced semigravity wall; cantilever wall; counterfort wall; and anchor wall. For this report, examples are focused primarily on gravity and cantilever retraining structures.

7.2 Design of retaining structures

Mortar–stone walls have been common in Afghanistan for centuries, as part of the country's architectural heritage. The design approach for mortar–stone walls is similar to that used for rigid retaining walls. However, mortar–stone walls need to be meticulously built because the appropriate materials, stones and mortar, are integral parts of the design criteria. In addition, skilled laborers or masons are necessary. The Afghans are quite familiar with masonry. For simplicity, this report provides information for building rigid retaining walls.

Retaining walls are commonly built to provide lateral support to vertical soil slopes. Retaining walls stabilize the backfill soil so that it remains in place and thus slopes do not fail. Rigid retaining walls are widely constructed, and can be classified into two types: gravity retaining walls (Fig. 22) and cantilever retaining walls (Fig. 23).

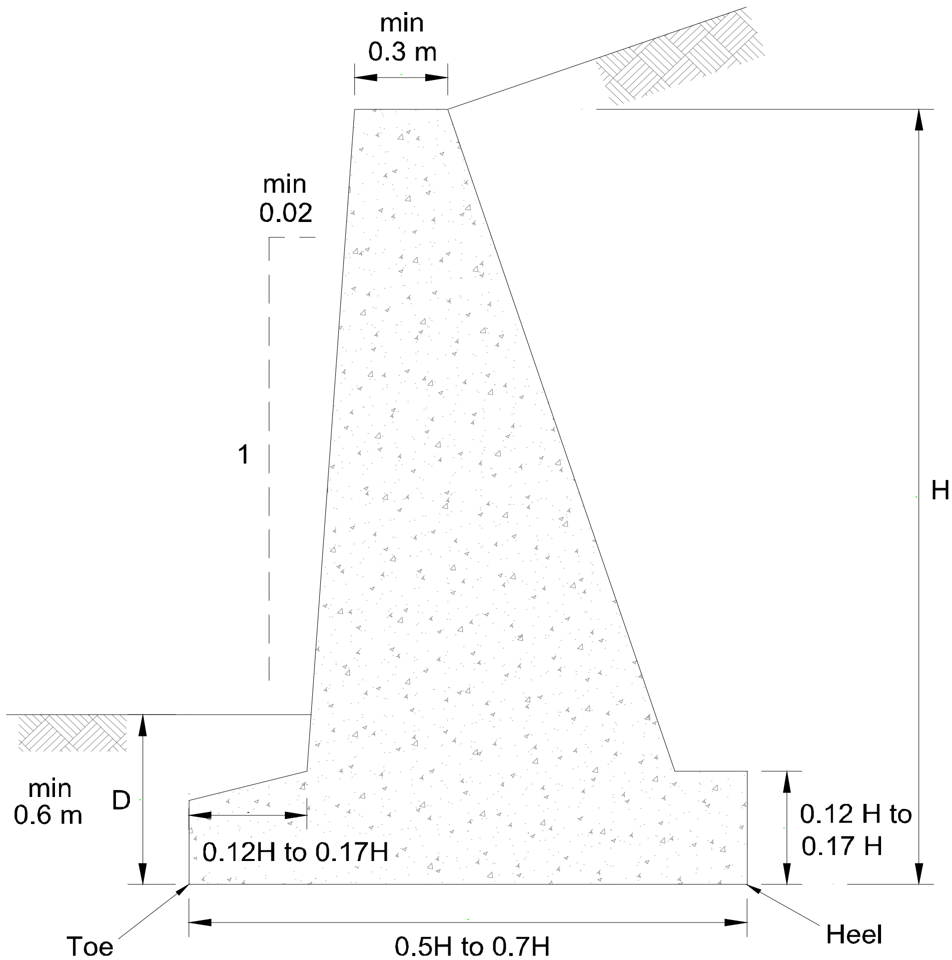


Figure 22. Typical dimensions for different components of gravity retaining wall.

7.3 Step by step design of gravity and cantilever retaining walls by Coulomb and Rankine methods

7.3.1 Determine the earth pressure coefficients

There are two common methods, that of Coulomb and that of Rankine, used to determine the active earth pressure coefficient (K_a) and passive earth pressure coefficient (K_p). Coulomb's method is based on limit equilibrium with or without friction between soil and wall. Rankine's method is based on the stress state of the soil mass. Rankine's method is more commonly used, as the procedure is simpler. For both methods, soil parameters can be taken from the tables in Chapter 5 if actual field data are not available. In addition, the following equations work in either English or SI units.

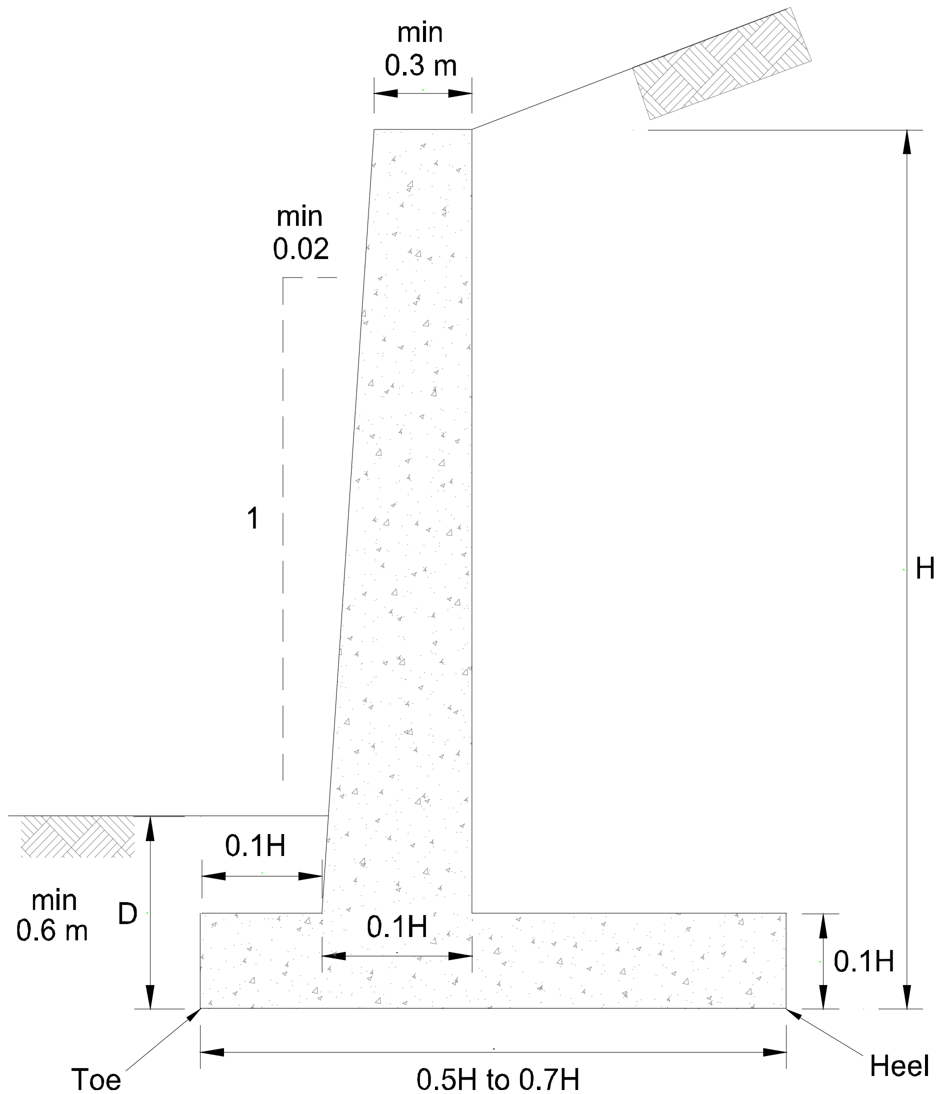


Figure 23. Typical dimensions for different components of cantilever retaining wall.

Equations 20 and 21 summarize Coulomb's method of determining K_a and K_p, based on the drawings in Figures 23 and 24.

$$K_{aC} = \frac{\cos^2(\phi' - \eta)}{\cos^2\eta\cos(\eta+\delta)\left[1+\left\{\dfrac{\sin(\phi'+\delta)\sin(\phi-\beta)^{1/2}}{\cos(\eta+\delta)\cos(\eta-\beta)}\right\}\right]^2} \quad (20)$$

$$K_{pC} = \frac{\cos^2(\phi' + \eta)}{\cos^2\eta\cos(\eta-\delta)\left[1-\left\{\dfrac{\sin(\phi'+\delta)\sin(\phi+\beta)}{\cos(\eta-\delta)\cos(\eta-\beta)}\right\}^{1/2}\right]^2} \quad (21)$$

where

K_{aC} = Coulomb active earth lateral pressure coefficient
K_{pC} = Coulomb passive earth lateral pressure coefficient
ϕ' = generic friction angle
η = angle of inclination of the wall face to the vertical
δ = wall friction angle
β = slope angle of the backfill.

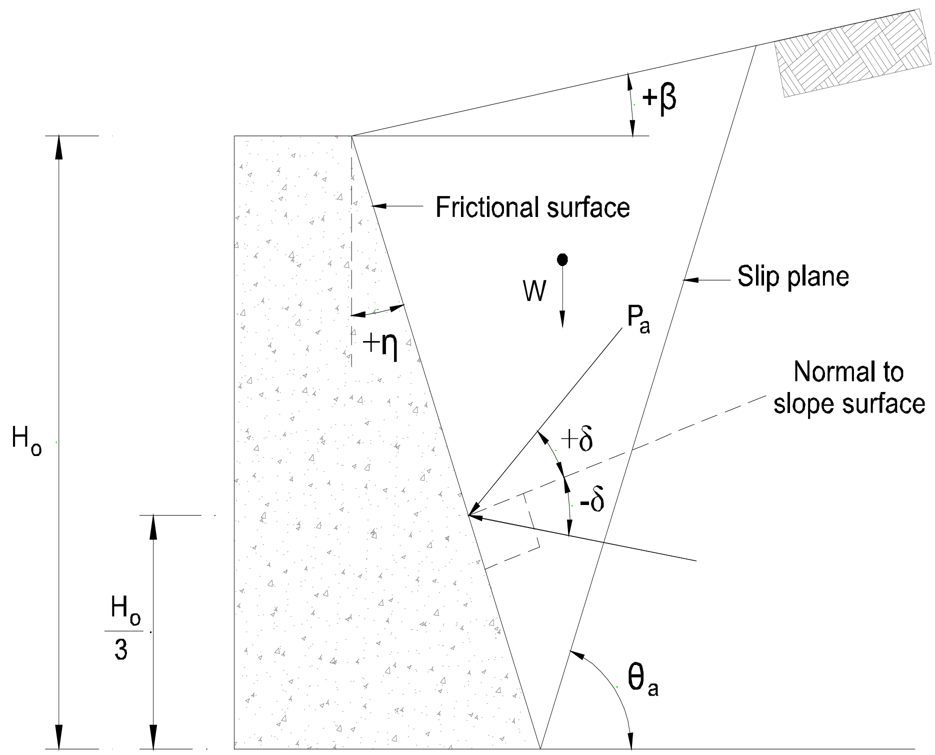


Figure 24. Retaining wall with sloping back, wall friction, and sloping soil surface for use with Coulomb's method for active conditions (adapted from Budhu 2008).

Equations 22 and 23 summarize Rankine's method, which is based on Figure 25.

$$K_{aR} = \frac{\cos(\beta - \eta)\sqrt{1 + \sin^2\phi' - 2\sin\phi'\cos\varphi_a}}{\cos^2\eta\,(\cos\beta + \sqrt{\sin^2\phi' - \sin^2\beta})} \tag{22}$$

$$K_{pR} = \frac{\cos(\beta - \eta)\sqrt{1 + \sin^2\phi' + 2\sin\phi'\cos\varphi_p}}{\cos^2\eta\,(\cos\beta - \sqrt{\sin^2\phi' - \sin^2\beta})} \tag{23}$$

where

K_{aR} = Rankine active earth lateral pressure coefficient

K_{pR} = Rankine passive earth lateral pressure coefficient

ϕ' = generic friction angle

η = angle of inclination of the wall face to the vertical

β = slope angle of the backfill

φ_a = $\sin^{-1}(\frac{\sin\beta}{\sin\varphi'}) + \beta + 2\eta$

φ_p = $\sin^{-1}(\frac{\sin\beta}{\sin\varphi'}) + \beta + 2\eta$

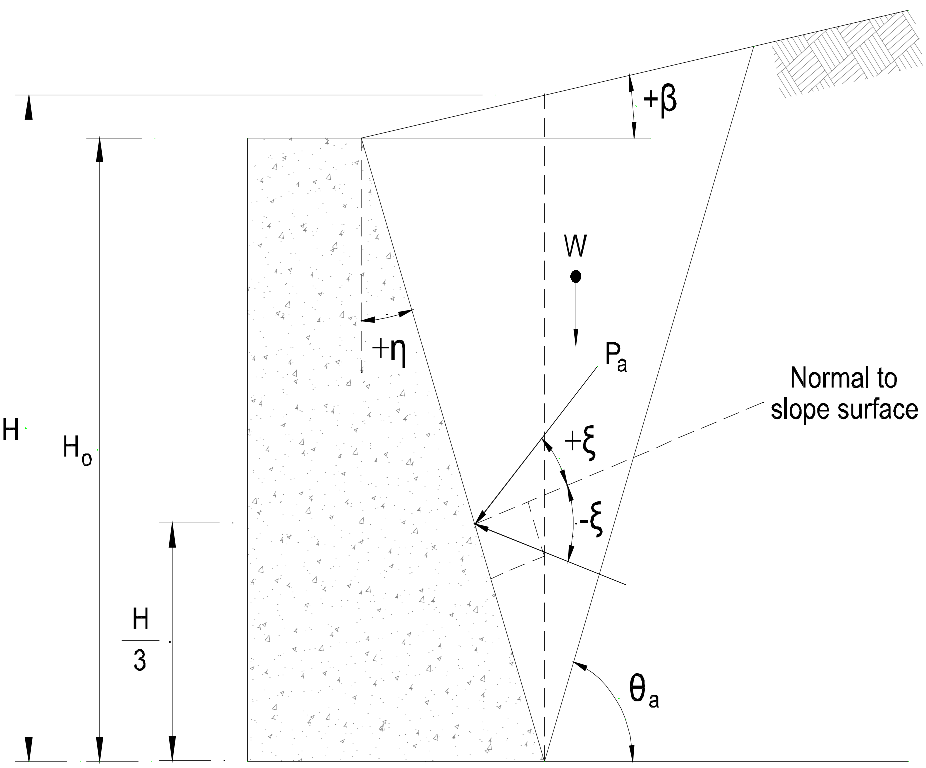


Figure 25. Retaining wall with sloping back, frictionless soil-wall interface, and sloping back for use with Rankine's method (adapted from Budhu 2008).

7.3.2 Determine the lateral forces

Equations 24 or 25 can be used to determine the active pressure P_a using the Coulomb or Rankine methods, respectively.

$$P_{aC} = \frac{1}{2} K_{aC} \gamma_{sat} H_O^2 \tag{24}$$

$$P_{aR} = \frac{1}{2} K_{aR} \gamma_{sat} H_O^2 \tag{25}$$

where γ_{sat} is the unit weight or density of the saturated soil.

7.3.3 Determine the wall stability

The stability of a gravity retaining wall depends mainly on its self-weight, while cantilever retaining walls also use the weight of soil under part of the structure and the strength of the structure to resist the soil pressure. Stability of any retaining wall should be checked for *translation*, *rotation*, and *bearing capacity*.

The sliding resistance at the base of the wall must resist translation that is caused by the resultant lateral force pushing the wall. The factor of safety against translation (FS_T) must be equal or greater than 1.5, expressed in eq 26.

$$FS_T \geq 1.5 \tag{26}$$

The retaining wall must resist rotation. The wall is safe against rotation if the eccentricity of the resultant vertical load is equal or smaller than width of the base divided by 6, as per eq 27.

$$e \leq \frac{B}{6} \tag{27}$$

where

e = eccentricity
B = width of base of the wall.

The retaining wall must be safe against failure of soil bearing capacity. The maximum stress at the base of the wall must not exceed the allowable soil bearing capacity, given by eq 28.

$$\sigma_{\max} \leq q_a \tag{28}$$

where

σ_{max} = maximum vertical stress at the base
q_a = allowable soil bearing capacity.

8 Summary and Conclusion

There have been extensive construction and reconstruction efforts undertaken throughout Afghanistan since the incursion by coalition forces in 2002. Many infrastructure projects were designed to recover the trade and livelihood of Afghanistan by improving infrastructure. However, the intended improvements have often not occurred, or not occurred with the intended speed, in part because the infrastructure has not been built to a suitable quality standard. This is often because of a lack of basic information about local conditions and materials, but may also be caused by limited local expertise and resources.

Local expertise is limited because there are few engineers who have graduated or practiced in the past 35 years in Afghanistan, and those often lack basic knowledge. The majority of local labor is unskilled as well. There are few local laboratories capable of performing material tests and quality control measurements. Access to quality construction materials is spotty. Logistics are complicated owing to the poor state of the transportation infrastructure. Lack of local infrastructure makes procurement difficult.

Even if a trained engineer is available and works from a set of standards, the information needed to apply the standards may not be available because of the limited amount of actual data available and lack of access to testing facilities to measure necessary parameters. In the past decade, this lack of reliable local data has led engineers to assume values based on experience in their home country, which may have very different conditions. Many projects occur in remote or dangerous places where it is difficult to conduct adequate site assessment and planning. Because of a lack of information during the planning process, the projects have limited design input and as result have encountered problems during construction.

This report aimed to overcome the lack of availability of actual data. Available geological information was used to obtain local soil classification and other engineering information pertinent to the Province of Nangarhar. The geological materials of Nangarhar Province can provide an abundant source of reasonable and competent gravel for construction. The common soils in the province are likely silty gravel with sand (GM), silty sand with gravel (SM), and sandy silt (ML) at 3- and 5-ft (1- and 1.5-m) depths.

However, poorly graded gravel with sand (GP) and sandy silty clay with gravel (CL-ML) are found at depth.

General ranges of modulus of elasticity and Poisson's ratio for basic types of soil are provided. Seismic faults are found in places and the activity of these faults is not known; therefore, uncertainty can be assumed. Even though climatic factors within the region play a significant role in the performance of infrastructure, temperatures would not particularly influence the design parameters because it is hot in the summer and the winter is very mild at the lower elevations. Precipitation occurs throughout the year (i.e., rain and possibly snow in the winter, especially at higher elevations). Soil erosion could occur in the low-lying areas, cause by severe run-off during rainfall and snowmelt in the spring. Proper drainage design must be incorporated in low-lying and erosion-prone areas.

In this report, Nangarhar was selected as a test case. A rational range of values for parameters used in designing and building infrastructure is provided. Design information and range of parameters compatible with common types of soil found in the region are obtained with the idea that the process used to generate this report could be replicated for other regions in Afghanistan. The reader should understand that actual data from the site, preferably obtained from experiments, are to be preferred over the values in this report.

The information compiled here can be applied in other areas of Afghanistan where equivalent soil types are located and where the soil classification is known. Experiments can be conducted in the field and in the laboratory to determine soil classification and other properties to build and compile physical data for the Afghans' infrastructure development. Even better, advanced lab tests can directly obtain data for parameters, and lab results can be compared to values given in this report to verify their plausibility and the quality of the lab work. In particular the *Mechanistic-Empirical Pavement Design Guide (*NCHRP 2004), currently used in the U.S., provides state of the art methods that can be applied equally well in Afghanistan with "real world" conditions.

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Appendix A: Flexible Pavement Example

Problem

Design a flexible pavement for rural principal arterial road with 10 million *ESALs*, provided that: subgrade has *CBR* of 10, P_t is 2.2, and the properties of materials used in the pavement are as follows:

Material	*a*	Modulus (psi)	Unit price ($/yd^3)
HMA	0.44	500,000	124
Aggregate Base *	0.13	25,000	30

*Assume coefficient of drainage (*m*) = 1.0

Solution

Need to obtain inputs needed to determine the required thickness of the AC and base layers.

$W_{18} = ESALs = 10 \times 10^6$

R = 85% (rural principal arterial)

S_0 = 0.49 (value typically used for flexible pavements)

$\Delta PSI = p_i - p_t = 4.2 - 2.2 = 2$ (4.2 is selected for flexible pavements) and 2.2 (given).

Solve for SN_1 and SN_2 to determine the required AC and Base layer thicknesses.

SN_2 = 3.95 (use nomograph as shown below).

SN_1 =3.05 (use nomograph).

Thickness of HMA

Required SN_1 = 3.05

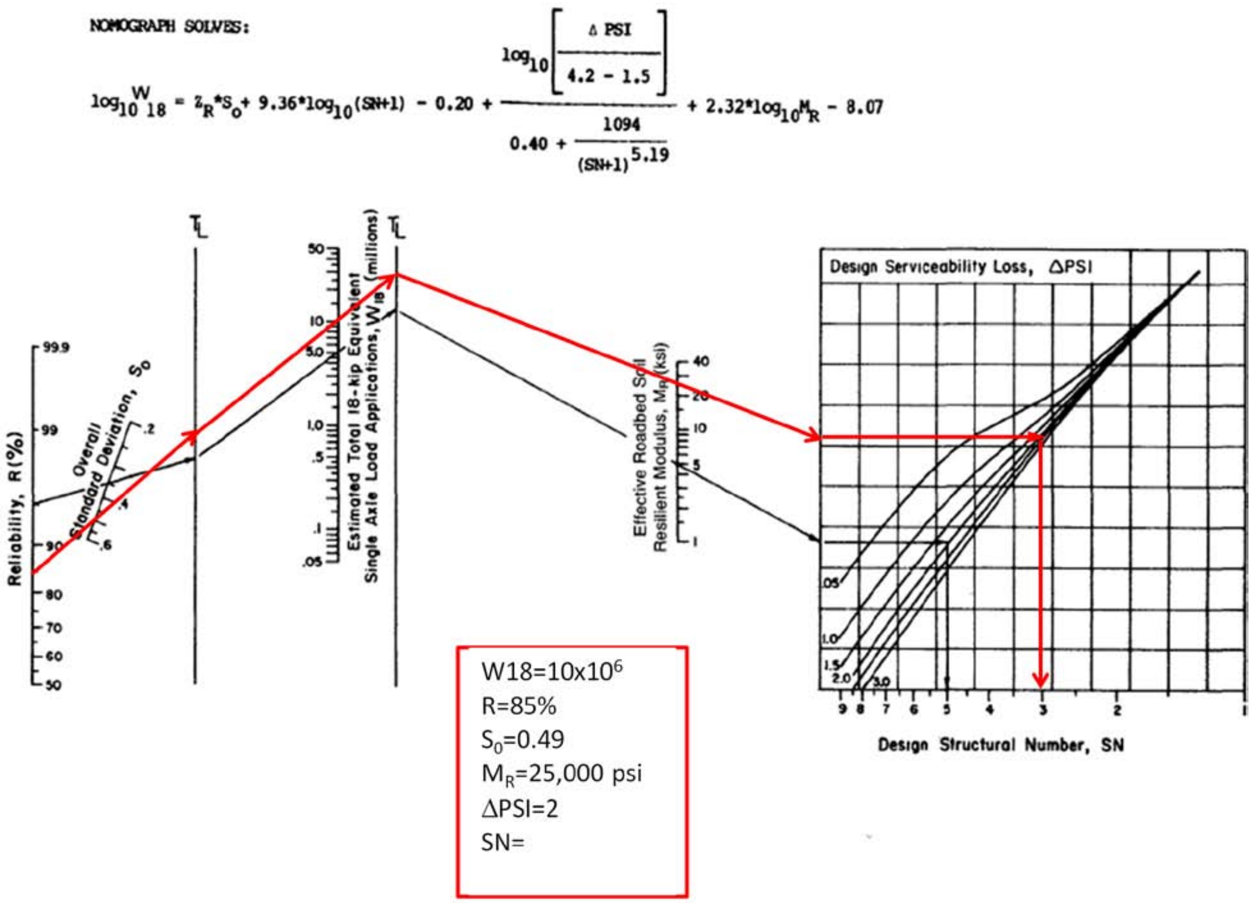
Required thickness =3.05/0.44= 6.93 in., select thickness of 7 in.

Thickness Base aggregate

$$D_2^* \geq \frac{SN_2 - D_1^* \cdot a_1}{a_2 m_2}$$

$$D_2^* \geq \frac{3.95 - 7 \cdot 0.44}{0.13 \times 1} = 6.59$$

Select thickness of 7 in.



Appendix B: Rigid Pavement Example

Problem

Design jointed plain concrete pavement for an urban interstate highway. The expected design *ESAL* is 16 × 106. The pavement structure is to consist of concrete slabs with an elastic modulus of 5.0 × 106 psi (34 GPa), a modulus of rupture of 650 psi (4.5 MPa), and a 12-in. (30.5-cm) thick cement-treated granular subbase. The pavement will be doweled and have an asphalt concrete shoulder. The climate at the pavement location consists of a two seasons (dry and wet). Freezing of the subbase and subgrade is considered negligible. The elastic modulus of the subbase is 1.0 x 106 psi (6.9 GPa) and is insensitive to climate. The *MR* of the subgrade is 10,000 psi (69 MPa) during the dry season and 5000 psi (35 MPa) during the wet season. There is no bedrock within 10 ft (3 m) of the subgrade. It is estimated that it will take a day for water to drain from the pavement and that the pavement will be saturated about 25% of the time. Assume p_t = 2.5 and an initial slab thickness is 11 in. (28 cm).

Solution

Compute the k_{design} using the following steps:

Interval	M_R (psi)	Subbase Modulus, E_{SB} (psi)	Composite *k*-value k_∞(pci) *	Relative Damage, u_r**
1	10,000	1.0×10^6	1500	81.
2	5,000	1.0×10^6	900	105

*Using Figure 12.18 [Huang 2002]

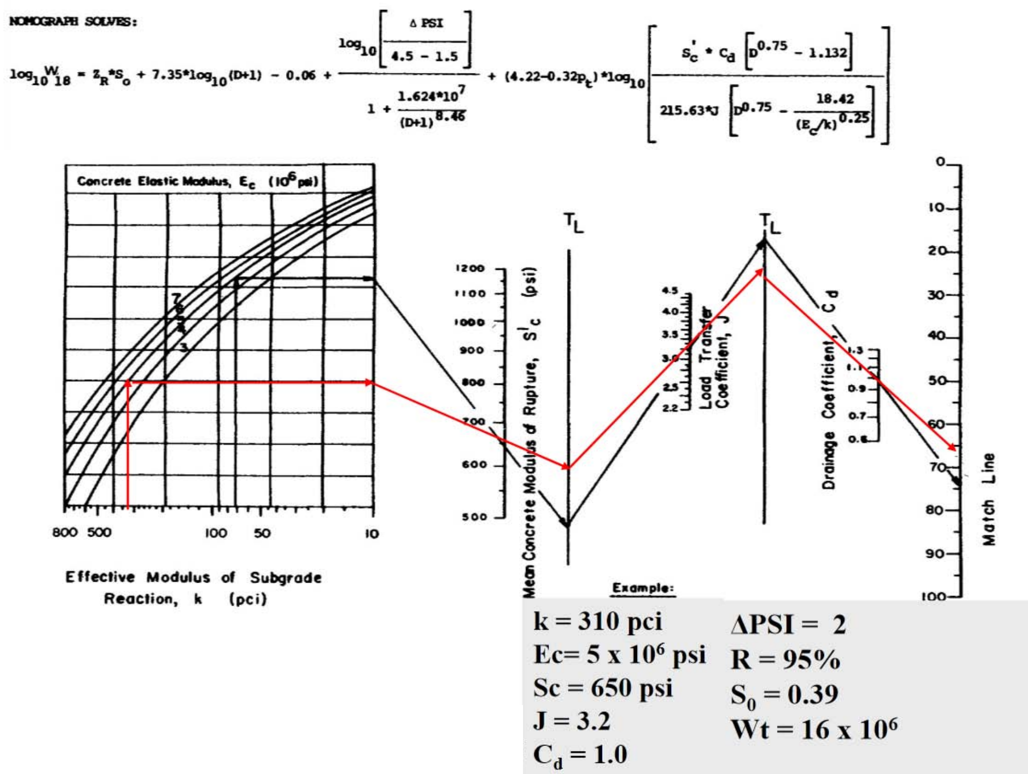
** $u_r = (D^{0.75} - 0.39k^{0.25})^{3.42}$ $\qquad \overline{u_r} = \dfrac{\sum u_r}{n} = \dfrac{81+105}{2} = 93$

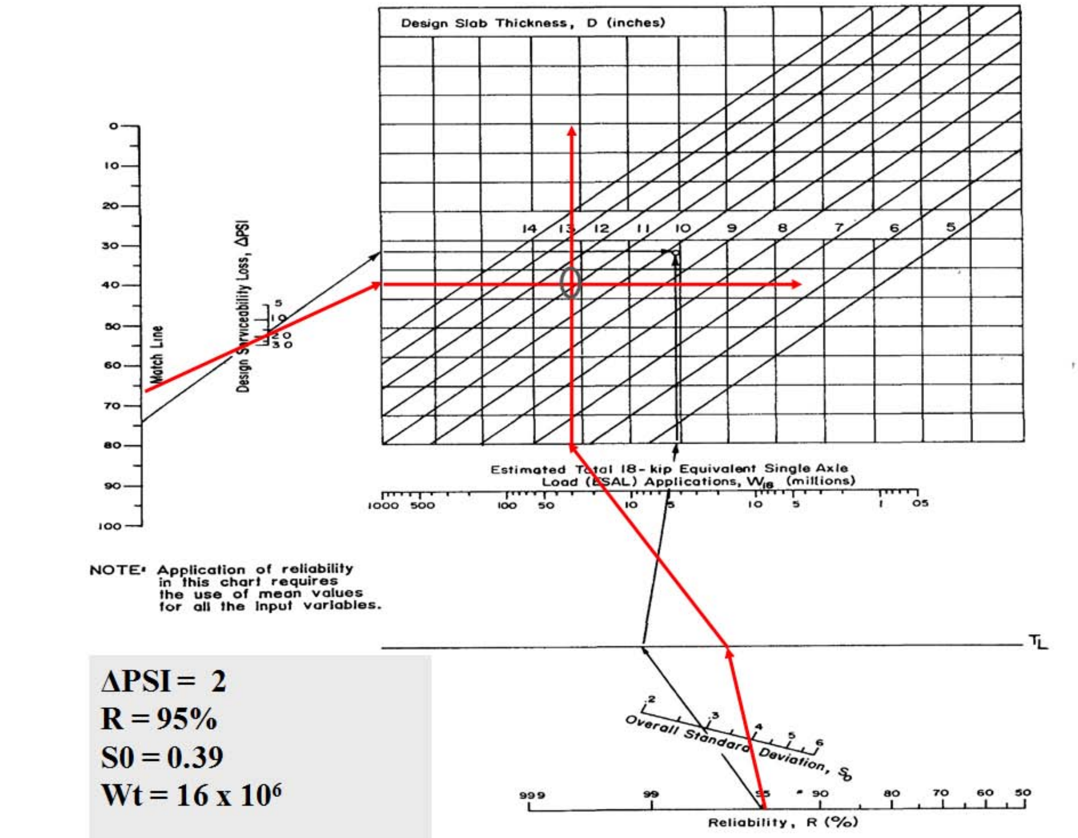
Effective k (k_{eff}) is 1160 pci.

Correct k_{eff} by a loss of support factor to account for erosion or differential soil movement. k_{design} = 310 pci.

Use the input values shown below to obtain a minimum slab thickness of 11.25 in. (28.575 cm) based on the following nomograph for rigid pavements design:

$$k = 310 \text{ pci}$$
$$E_c = 5 \times 10^6 \text{ psi (given)}$$
$$S_c = 650 \text{ psi (given)}$$
$$J = 3.2$$
$$C_d = 1.0$$
$$\Delta PSI = 4.5 - 2.5 = 2$$
$$R = 95\% \text{ (Urban interstate highway)}$$
$$S_0 = 0.39 \text{ (value typically used for rigid pavement)}$$
$$W_{18} = ESALs = 16 \times 10^6 \text{ (given)}$$




Design Slab Thickness, D (inches)
14 13 12 11 10 9 8 7 6 5
Match Line
Design Serviceability Loss, ΔPSI
Estimated Total 18-kip Equivalent Single Axle Load (ESAL) Applications, W_{18} (millions)
NOTE: Application of reliability in this chart requires the use of mean values for all the input variables.
T_L
Overall Standard Deviation, S_o
Reliability, R (%)
ΔPSI = 2
R = 95%
S0 = 0.39
Wt = 16 x 10⁶

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14. ABSTRACT
Since 2002, coalition forces have focused on efforts to reconstruct and create new infrastructure in Afghanistan to help stabilize the nation and improve trade and opportunities for a livelihood. The U.S. projects in Afghanistan used common American construction standards. Applying these standards to the projects appeared reasonable, but, in actual practice, these standards have been difficult to implement owing to limited access to technical information, lack of information about local terrain and environment, and limited understanding of local social customs. Afghanistan lacks skilled labor capable of performing quality work. In addition, there are few Afghan testing laboratories for conducting quality control. The quality of available construction materials is unreliable. Logistics are complicated because of the poor state of the transportation infrastructure, and there are no local systems necessary to sustain procurement efforts for major infrastructure projects. Even if a trained engineer is available and willing to work from a set of standards, the information needed to apply the standards may not be available and testing facilities to measure necessary parameters are lacking. This report aims to overcome the lack of availability of actual data for Nangarhar Province. A rational range of values for parameters used in designing and building infrastructure is presented in this report from available geological information and climatic data. The geological data were used to obtain local soil classification and other relevant engineering information. Nangarhar Province was selected as a test case to gather together a pertinent set of design information and useful data with the idea that the process used to generate this report could be replicated for other regions in Afghanistan.

15. SUBJECT TERMS

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