US Army Corps of Engineers
Afghanistan Engineer District South

Afghanistan National Army 215th Brigade Garrison Phase I Camp Garmsir

Helmand Province, Afghanistan

Appendix B -1

AED Design Requirements

MAY 2011

THIS IS A SINGLE PHASE REQUEST FOR PROPOSAL

US Army Corps
of Engineers
Afghanistan Engineer District

AED Design Requirements:
Culverts & Causeways

Various Locations,
Afghanistan

JULY 2009, Version 1.3

TABLE OF CONTENTS

AED DESIGN REQUIREMENTS
FOR
CULVERT /CASUEWAY DESIGN
VARIOUS LOCATIONS,
AFGHANISTAN

1. General

The purpose of this document is to provide requirements to contractors for any project requiring hydraulic design of drainage structures crossing roadways. Culverts, gabion crossings, at-grade concrete wadi crossings or other related structures shall be constructed as required over rivers, dry wadis, canals and other manmade channels that contain water and deep drainages that fill with water during the rainy season. Road sections that cross wide drainages, flood areas or wadis shall be designed and constructed with additional erosion control measures to allow the road to be passable and minimize damage during frequent rain conditions. High erosion areas, such as shallow drainage crossings and wadis, shall be armored with a hard surfaced crossing such as rip rap and provided with debris catchment devices such as STRAFLIN debris barrier or steel debris cage. Culverts smaller than 1m by 1m shall not be constructed; using instead low water crossings (also known as causeways). Causeways shall be built according to Ministry of Rural Rehabilitation and Development (MRRD) standards shown in Reference 1. All existing culverts smaller than 1m by 1m than require replacement shall be replaced with low water crossings; exceptions will be considered by the contracting officer only on a case by case basis.

2. Culverts

2.1 Criteria. Culverts will be used to convey runoff under roads, runways, perimeter walls, or other similar site features in order to prevent the ponding of runoff that may cause a hazardous condition and to prevent damage to site features. The Contractor shall include the following criteria while designing a culvert.

1) All culverts shall be hydraulically designed to pass the peak design flow from the selected design storm. The design storm (return period) selected shall be consistent with the class of road, highway or airfield type.

2) Culvert material selection shall include consideration of service life that includes abrasion and corrosion. Unreinforced concrete culverts are not permitted. Slab culverts constructed to MRRD standards are accepted.

3) Culverts shall be located and designed to present minimum hazard to traffic and people.

4) Culvert length and slope shall be chosen to approximate the existing topography. Culvert invert shall be aligned with the existing channel bottom and the skew angle of the channel to the maximum extent possible.

5) Culverts shall have a minimum of 0.60 meters of cover within the travel way of roads.

6) Allowable headwater is the depth of water that will be allowed to pond at the upstream end of the culvert during the <u>design storm</u> which will be a minimum of 0.45 meters below the edge of the shoulder of the road being crossed. The headwater shall be determined as part of the hydraulic analysis.

7) Maximum velocity at the culvert exit shall be consistent with the velocity in the natural channel or shall be mitigated with channel stabilization or energy dissipation using riprap, gabions or boulder flow stilling designs.

8) Design velocity at the peak design discharge rate determined from the hydrological analysis (see Reference 2) in the culvert shall be greater than 1 meter/second for sediment transport conveyance capacity.

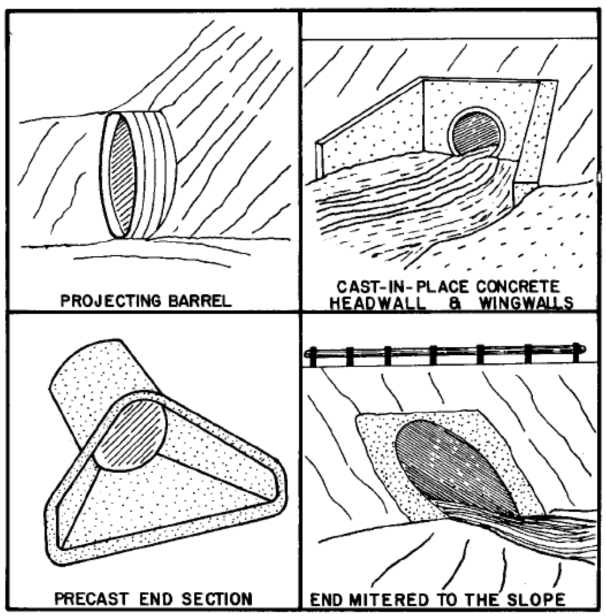
9) Contractors are encouraged to adapt the geometry (i.e. shape and slope) of the standard MRRD culvert and causeway design drawings to site conditions where applicable. For example, reinforced box culvert cross slopes should be adjusted to existing channel grade

rather than provided a flat cross slope that promotes sedimentation inside the culvert and requires future maintenance to obtain design capacity.

2.2. End Treatments

Culvert end treatments are to be provided in specified in the contract technical requirements. Circumstance that require the use of end treatments include construction of the culvert at a severe skew angle to the flow path of the channel which requires a redirection of the flow into the culvert, and where the culvert is covered by a high embankment which requires that erosion around inlet and outlets be minimized used an end treatment. The culvert end treatment type shall be selected based on the diameter of the culvert and the potential hazard to errant vehicles. All culverts larger 1200 mm in diameter and larger should have headwalls and wing walls or shall be mitered to the slope and protected by grouted masonry, and rip rap outlet protection aprons. Where headwalls, wing walls or mitered ends are used, the culvert ends should be extended a sufficient distance from the travel lanes so that there is no hazard to errant vehicles or a traffic barrier (guard rail) should be installed adjacent to the headwall or wing wall. Examples of end treatments are provided in Figure 1. Other examples are found in the MRRD standard drawings (Reference 1).

Figure 1. End Treatment Examples



2.3. Stone Aprons and Cutoff Walls

Approach aprons and or cutoff walls should be used to reduce scour from high headwater depths or from approach velocity in the channel. Approach aprons should be concrete or large diameter rip-rap and shall extend at least one pipe diameter upstream of the culvert entrance. MRRD culvert standard drawings provide details of aprons and cutoff wall design. See MRRD standard reinforced box culvert drawings DCV-05 and 06. Approach aprons should not protrude above the normal streambed elevation. Outlet protection shall be provided at the downstream end of all culverts where the culvert discharge velocity is greater than the natural channel velocity.

2.4. Types of Flow Control That Determine Culvert Capacity

The hydraulic capacity of the culvert depends upon the type of hydraulic control at the design flow rate for the culvert. Type of control refers to the inlet and outlet water surface elevations of the culvert. Culverts with inlet control have a shallow, high velocity flow categorized as "supercritical". For supercritical flow, the control section of the culvert is the upstream end of the barrel (inlet end). Conversely, a culvert flowing under outlet control will have a relatively deep, low velocity flow categorized as "subcritical". The "tail water", that is the natural channel flow depth at the end (or tail) of the culvert limits the flow capacity of the culvert because of natural channel capacity. Typically the flow will be subcritical under this condition. For subcritical flow, the control section of the culvert is either the downstream end of the culvert barrel or the outlet channel section. The tail water depth is either the critical flow depth at the culvert outlet or the downstream channel flow depth, whichever is higher. For all culverts, the type of flow is dependent on all of the factors listed Table 1.

All of the factors influencing the performance of the culvert in inlet control also influence culverts in outlet control. In addition, the barrel characteristics (roughness, area, shape, length and slope) and the tail water elevation affect culvert performance in outlet control. Roughness is a function of the material used to fabricate the barrel. Typical materials include concrete and corrugated metal. The barrel area is the cross-sectional area of the barrel and the barrel shape is the shape of the barrel (circular, square, rectangular, etc.). The barrel length is the total culvert length from the entrance to the exit of the culvert. Because the slope influences the actual length of the barrel, an approximation of the barrel length is usually necessary to begin the design process. The barrel slope is the actual slope of the barrel which is the difference of the inlet and outfall ends of the culvert divided by the length of the culvert.

Table 1. Factors Influencing Culvert Performance

Factor	Inlet Control	Outlet Control
Headwater Elevation	X	X
Inlet Area	X	X
Inlet Edge Configuration	X	X
Inlet Shape	X	X
Barrel Roughness		X
Barrel Area		X
Barrel Shape		X
Barrel Length		X
Barrel Slope	*	X
Tailwater Elevation		X
*Barrel slope affects inlet control performance to a small degree, but may be neglected.		

2.5. Culvert Design Nomographs

The design of culverts is normally achieved using design forms and nomographs for inlet control and outlet control and critical depth charts. The use of the Manning's equation alone is insufficient to establish the principal design parameter the headwater on the culvert at the road embankment. A nomograph is a chart usually containing three parallel scales graduated for different variables so that when a straight line connects values of any two, the related value may be read directly from the third at the point intersected by the line. Numerous inlet control and outlet control nomographs and critical depth charts are available the different shapes and materials of culverts. The design process is explained in further detail in Appendix A. A complete set of nomographs for most commonly used shapes in SI units can be found at the web site shown in Reference 3. A design example is shown in

Appendix B. Once all of the known factors that influence culvert performance and the design storm are known, the process of the culvert design can begin using the design form provided in Appendix B.

3. Causeways

3.1 Criteria. Causeways will be used to convey runoff over reinforced concrete slabs in road sections or other similar site features, for example fenced perimeter channels across wadi where the rate of peak runoff for the design storm is expected to be less than 2.2 cubic meter per second. This flow rate is the approximate hydraulic capacity of most 1m by 1m reinforced concrete box culverts. Causeways will be substituted for existing culverts (1m x 1m or smaller dimension) that because of their deteriorated condition need replacement. Larger causeways can be proposed but should be hydraulically designed because the standard dimensions of the MRRD causeways would need to be site adapted. An example of a causeway plan and elevation that can be used as a basis for site adaption to a specific site is shown in Appendix C.

The Contractor shall include the following criteria while designing a causeway:

1) All causeways shall be equal to or greater than the standard dimensions shown in the MRRD standard drawings. The type of cause way shall be based on the terrain through which the road travels: flat or mountainous. The minimum length of the causeway shall be 10 meters. The causeway shall slope to a low point approximately in the center of the longitudinal alignment. Longitudinal slopes (along the road centerline) shall be approximately10 percent as shown in the example in Appendix C.

2) Causeway embankments, both upstream and downstream in the direction of the overflow shall be protected using heaving stone revetment. The length of revetment depends upon the type of causeway: flat or mountainous type, as shown in the MRRD standard drawings DW-01 and 02.

3) Embankment riprap gradations are provided in the MRRD standard drawings (sheet DSR-01). The gradation selection shall be based upon the average channel velocity calculated for the approach (upstream side) of the causeway embankment. For causeways in flat terrain (channel slope less than 3 percent) the gradation equal to or greater than shown in Table 3 shall be used. For steeper slopes (greater than 3 percent) or causeways for mountainous terrains, the gradation shall be per design based on a hydraulic analysis of the upstream channel velocity. Riprap design information found in Reference 4 is similar to the data shown in the MRRD standard drawings.

4) For causeways crossing irrigation canals or washes with continuous runoff sustained by springs or groundwater, small diameter (100 mm) PVC or HDPE bypass pipes may be provided beneath the compacted backfill of the causeway slab between the upstream and downstream weep holes shown on the MRRD standard drawings in the downstream cutoff walls. The standard spacing is 2 meter centers for the bypass pipes.

5) Causeway approaches shall be provided speed bumps at end side and warning signs to alert pedestrians and vehicles of the possibility of “water over the roadway”.

Table 2. Embankment Riprap Median Stone and Layer Thickness
(Facing Riprap Class)

Flat Causeway Design

(up to 3 % upstream channel slope)

Q, cms	causeway flow depth, m	design velocity, m/s	D_{50}, mm	T, mm
1	0.34	0.85	200	300
1.5	0.4	0.94	233	350
2	0.44	1.02	250	375
2.5	0.48	1.08	257	385
3	0.5	1.14	267	400

Based on Reference 4

3.2. Causeway Debris Control

Upstream debris control for causeways shall be provided based on contract technical requirements stipulate or on a case by case basis determined by the designer. Where the tributary area to the causeway has a visibly large production potential for boulders, cobbles and gravel sediment, an energy dissipation type debris structure shall be designed for the upstream edge of the causeway to reduce the volume of material transported across the road surface.

Contract requirements may specify the use of STRAFLIN or Salerno debris control devices shown in Appendix C. Note these are site adapt drawings that must be adjusted to the size of the culvert.

Appendix D contains details of a potential debris structure that must be site adapted to either a road causeway or perimeter fence wadi crossing to be used. Note these are site adapt drawings that must be adjusted to the size of the causeway.

4. Design Submittal Documentation

Design analysis documentation shall summarize the hydraulic structures designed in the project in tabular form. Design information shall include the following:

- Peak discharge flow rate obtained from hydrologic analysis used as the basis of the design
- downstream channel dimensions and drainage slopes adjacent to the road alignment
- culvert slope
- calculated flow depth in the culvert based on culvert hydraulic design procedures shown in Appendices A & B
- calculated flow velocity in channel
- proposed channel lining material if any
- rip rap layers and gradation for causeway design and culvert outlet protection
- site adapted sizes for debris control devices shown in Appendices C and D

Design variation in structure size, slope and orientation is expected and therefore the results of the design variations shall be conveyed to those constructing the structure by summarize the structure dimension and other design information on schedules shown on the construction drawings.

5. References

1. Ministry of Rural Rehabilitation and Development. Standard Drawings. Revision –I, June 2006
2. USACE-AED Design Requirements Hydrology Studies, 2009
3. U.S. Department of Transportation Federal Highway Administration. Hydraulic Design Series Number 5 – Hydraulic Design of Highway Culverts. Publication No FHWA-NHI-01-020, Revised May 2005. Found at http://www.fhwa.dot.gov/engineering/hydraulics/library_arc.cfm?pub_number=7
4. U.S. Department of Transportation Federal Highway Administration. Hydraulic Design Series Number 11 – Design of Riprap Revetment. Publication No FHWA-IP-89-016, March 1989. Found at http://www.fhwa.dot.gov/engineering/hydraulics/pubs/hec/hec11sI.pdf
5. U.S. Department of Transportation Federal Highway Administration. Hydraulic Engineering Circular 9. Debris Control Structures – Evaluation and Counter Measures FHWA-IF-04-016, October 2005.

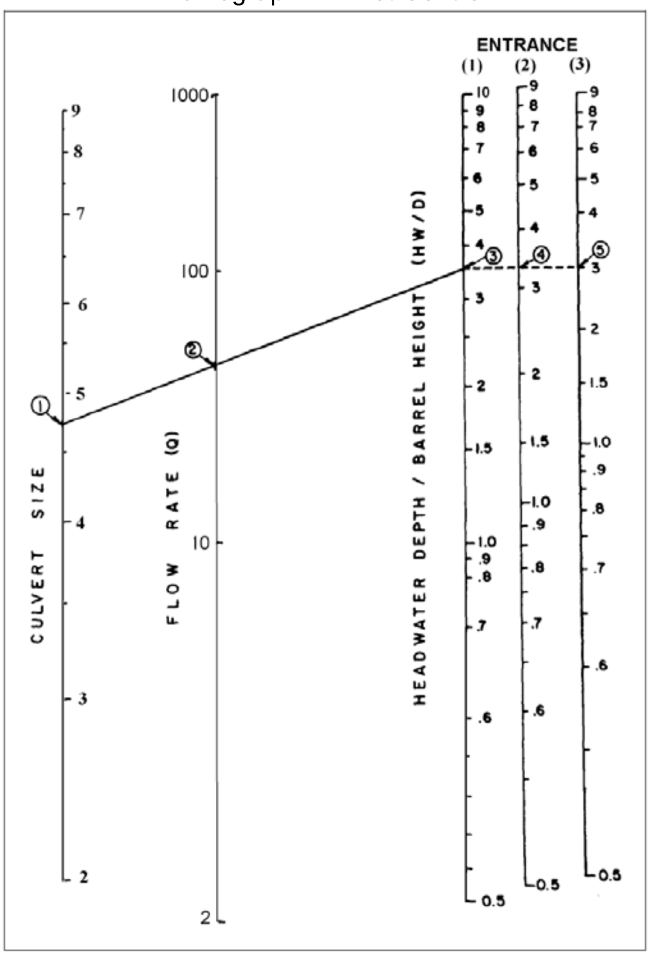
Appendix A – Culvert Design Procedure

1. Inlet Control

The inlet control calculations determine the headwater elevation required to pass the design flow through the selected culvert in inlet control. The designer should begin the design process by summarizing all known data for the culvert at the top of the culvert design form. This information will have been collected or calculated prior to performing the actual culvert design. This information should including the drainage area, return period, design flow, method of determining design flow, culvert inverts, culvert length and slope, tail water depth and controlling roadway elevation. The next step is to select the preliminary culvert material, size shape, and entrance type for the culvert. This preliminary information and the design flow rate are entered under the Culvert Description and Total Flow columns of the culvert design form in the middle of the form. The following steps should be completed to calculate the inlet control design for the culvert.

1) Using the appropriate inlet control nomograph the designer locates the culvert size (point 1) and flow rate (point 2) on the appropriate scales. An example of an inlet control nomograph is provided below.

Nomograph 1. Inlet Control



2) Using a straightedge, carefully extend a straight line from the culvert size (point 1) through the flow rate (point 2) and mark a point on the first headwater/culvert height (HW/D) scale (point 3). The first HW/D scale is also a turning line.

3) If another HW/D scale is required, extend a horizontal line from the first HW/D scale to the desired scale and read the results.

4) Enter the value from the appropriate HW/D scale in the HW_i/D column (column 2) in the middle of the culvert design chart under Inlet Control. Multiply the HW_i/D value by the culvert height to obtain the required headwater (HW_i) from the invert of the control section to the energy grade line. This result is placed in the column to the right of column 2.

5) Calculate the required depression (FALL) of the inlet control section below the stream bed as follows.

   $HW_d=EL_{hd}-EL_{sf}$

   $Fall=HW_i-HW_d$

   HW_d=design headwater depth (m)

   EL_{hd}=design headwater elevation (m)

   EL_{sf}=elevation of the culvert entrance (m)

   HW_i=required headwater depth (m).

   After the FALL has been determined the design should examine that value based on the following criteria.

   If the FALL is negative or zero, set the FALL in column 3 of the culvert design form to zero and proceed to step 6.

   If the FALL is positive, the inlet control section invert must be depressed below the streambed at the face by that amount, if this amount is acceptable proceed to step 6.

   If the FALL is positive and greater than is judged to be acceptable, select another culvert configuration and begin at step 1.

6) Calculate the inlet control section invert elevation as follows:

   $EL_i=EL_{sf}-FALL$

2. Outlet Control

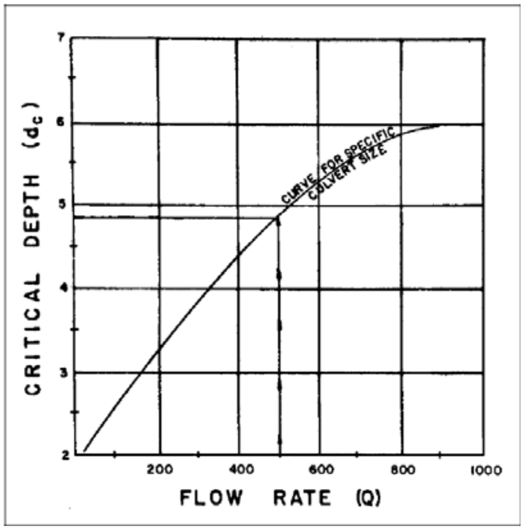
The outlet control calculations result in the headwater elevation required to convey the design discharge through the selected culvert in outlet control. The critical depth charts and outlet control nomographs are used in the outlet control design process. The following steps should be completed to calculate the outlet control design for the culvert.

1) Determine the tailwater (TW) depth above the outlet invert at the design flow rate. This is obtained from backwater or normal depth calculations. This information is entered in the TW column (column 5) in the middle of the culvert design chart under Outlet Control.

2) Enter the appropriate critical depth chart with the flow rate and culvert size and read the critical depth (d_c). The critical depth cannot exceed the diameter of the culvert. This

information is entered on the d_c column in the middle of the culvert design chart under Outlet Control. An example of a critical depth chart is shown below.

Chart 1 – Critical Depth



3) Calculate $(d_c+D)/2$, where D is the culvert diameter and enter this information in the appropriate column in the middle of the culvert design chart under Outlet Control.

4) Determine the depth from the culvert outlet invert to the hydraulic grade line (h_o) and enter this information in column 6 in the middle of the culvert design chart under Outlet Control.

   H_o=TW or $(d_c+D)/2$, whichever is larger.

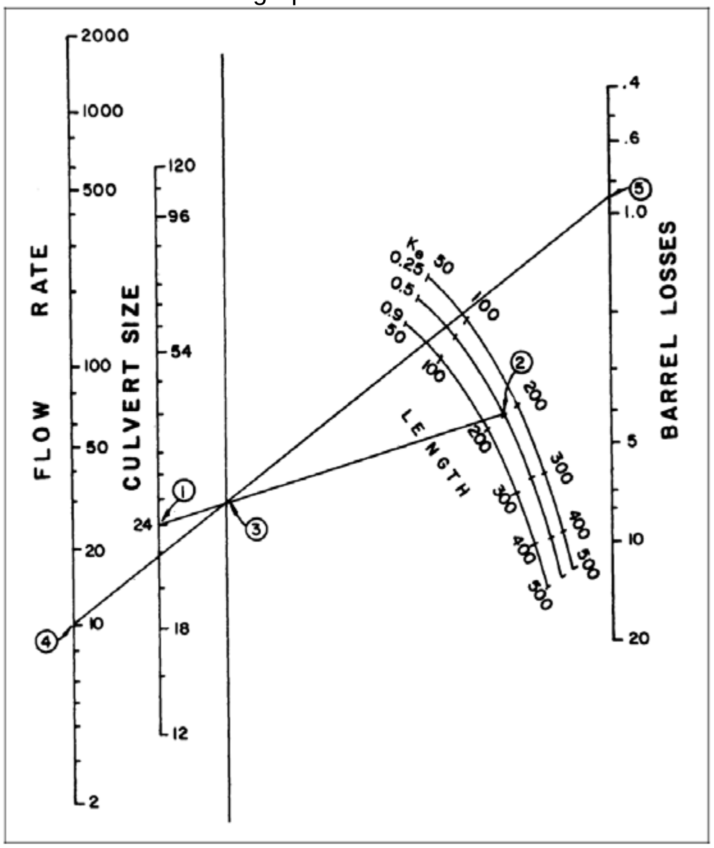
5) Determine the appropriate entrance loss coefficient, k_e, for the culvert inlet configuration. An entrance loss coefficient table is shown below.

Table 2. Entrance Loss Coefficients

Type of Structure and Design of Entrance	Coefficient K_e
• Pipe, Concrete	
Projecting from fill, socket end (groove-end)	0.2
Projecting from fill, sq. cut end	0.5
Headwall or headwall and wingwalls	
Socket end of pipe (groove-end	0.2
Square-edge	0.5
Rounded (radius = D/12	0.2
Mitered to conform to fill slope	0.7
*End-Section conforming to fill slope	0.5
Beveled edges, 33.7^0 or 45^0 bevels	0.2
Side- or slope-tapered inlet	0.2
• Pipe, or Pipe-Arch, Corrugated Metal	
Projecting from fill (no headwall)	0.9
Headwall or headwall and wingwalls square-edge	0.5
Mitered to conform to fill slope, paved or unpaved slope	0.7
*End-Section conforming to fill slope	0.5
Beveled edges, 33.7^0 or 45^0 bevels	0.2
Side- or slope-tapered inlet	0.2
• Box, Reinforced Concrete	
Headwall parallel to embankment (no wingwalls)	
Square-edged on 3 edges	0.5
Rounded on 3 edges to radius of D/12 or B/12 or beveled edges on 3 sides	0.2
Wingwalls at 30^0 to 75^0 to barrel	
Square-edged at crown	0.4
Crown edge rounded to radius of D/12 or beveled top edge	0.2
Wingwall at 10^0 to 25^0 to barrel	
Square-edged at crown	0.5
Wingwalls parallel (extension of sides)	
Square-edged at crown	0.7
Side- or slope-tapered inlet	0.2

6) Determine the losses through the culvert barrel, H, using the appropriate outlet control nomograph. An example of an outlet control nomograph is shown below.

Nomograph 2. Outlet Control



If the Manning's n value given in the outlet control nomograph is different that the Manning's n for the culvert, adjust the culvert length using the formula:

$L_1=L(n_1/n)^2$

L_1 is the adjusted culvert length in meters.

L is the actual culvert length in meters.

n_1 is the desired Manning's n value.

n is the Manning's n value from the outlet control chart.

The, use L_1 rather than the actual culvert length when using the outlet control nomograph.

a) Using a straightedge, connect the culvert size (point 1) with the culvert length on the appropriate k_e scale (point 2). This defines a point on the turning line (point 3).

b) Using a straightedge, extend a line from the discharge (point 4) through the point on the turning line (point 3) to the Head Loss (H) scale (point 5). Head Loss is the energy loss through the culvert, including entrance, friction and outlet losses. Enter the Head Loss in the H column (column 7) in the middle of the culvert design chart under Outlet Control.

7) Calculate the required outlet control headwater elevation.

   $EL_{ho}=EL_o+H+h_o$

   Where EL_o is the invert elevation at the outlet

8) If the outlet control headwater elevation exceeds the allowable headwater elevation, a new culvert configuration must be selected and the process repeated. Generally, an enlarged barrel will be necessary since inlet improvements are of limited benefit in outlet control.

9. Evaluation of Results

Compare the headwater elevations calculated for inlet and outlet control. The higher of the two is designated the controlling headwater elevation. The culvert can be expected to operate with the higher headwater for at least part of the time. Enter the controlling headwater elevation in the appropriate column in the middle of the culvert design chart. The outlet velocity is calculated as follows.

If the controlling headwater is based on inlet control, determine the normal depth and velocity in the culvert barrel. The velocity at normal depth is assumed to be the outlet velocity.

If the controlling headwater is in outlet control, determine the area of flow at the outlet based on the barrel geometry and the following:

1) Critical depth if the tail water is below critical depth.

2) Tail water depth if the tail water is between critical depth and the top of the barrel.

3) Height of the barrel if the tail water is above the top of the barrel.

Repeat the design process until an acceptable culvert configuration is determined. Once the barrel is selected it must be fitted into the roadway cross section. The culvert barrel must have adequate cover, the length should be close to the approximate length, and the headwalls and wing walls must be dimensioned.

Appendix B – Culvert Design Example

Culvert Design Example

Reference: *U.S. Department of Transportation Federal Highway Administration.*
Hydraulic Design Series Number 5 – Hydraulic Design of Highway Culverts.
Publication No FHWA-NHI-01-020, Revised May 2005.

Given Data

10-year design flow=	1	m^3/s	from result of the USACE-AED Hydrology Study Appendix A example
Design Head Elevation, ELhd=	33.528	m	
Road shoulder elev=	34.595	m	
Channel invert levation,Eli=	30.48	m	
Number of barrels	1		
Stream bed slope, So=	0.02	m/m	
Approx Culvert length, La=	76.2	m	
Fall=	0	m	
Outlet elev, ELo=	28.956	m	
S=So-(Fall/La)=	0.02	m/m	
Hwi=Elhd-Eli=	3.048	m	
Box Mannings n=	0.013	concrete	

Tailwater variation table	Flow, m^3/s	Tailwater elevation, m	
			Tailwater variation determined by
	0.5	0.3	using cross sections upstream and
	1	0.51	downstream of culvert to compute
	1.5	0.64	water surface profile using standard
Based on channel n value=		0.035	water surface profile calculation
			such as standard step method

Trial #1 Box width, B=	1	m	Area of box=	1.00	sq m
Trial #1 Box height, D=	1	m			
Inlet type=	square	90 o wingwall			

<u>Technical Notes per Design Chart Explanation========></u> See CULVERT DESIGN FORM

(1)	Q /barrel=Q/(N*B)=	1.00 m^3/s-m	divide design Q by number barrels*width)		
(2.a)	Hwi/D	0.800 Chart 8A	Use result from (1) to read across chart 8A to second line (90 o wingwall) - read 0.8		
(2.b)	Hwi	0.800 m	multiply result from (2) * D - obtain 0.8		
(3)	Fall	0	fall - the depression of the inlet below the stream bed - is zero for culverts on grade		
(4)	Elhi=Hwi+Eli	31.28 m	elevation of headwater in inlet control		
(5.a)	TW	0.51 m	tailwater depth as detrmined for design flow from tailwater variation table		
(5.b)	dc	0.45 Chart 14A	critical depth for rectangular cross section from chart 14 A - read 0.45 for Q/B=1		
(5.c)	(dc+D)/2	0.725 m	(0.45+1)/2=0.725		
(6.a)	greater TW or (dc+D)/2,Ho	0.725 m	select greater value between (5.a) and (5.c)		
(6.b)	ko=	0.5	loss coefficient for outlet		
(7)	H=	0.16 m	H=(1+ko+(19.63*n^2)/R^1.33)*v^2/(2*9.81)) (sum of inlet loss, friction loss, and velocity head)		
(8)	Elho=Elo+H+He	29.84 m	sum of H + Head$_{exit}$ +exit elev.	R=A/P=	0.25
control headwater elev		31.28 m	greater value of (8) or (4)	Inlet controls culvert flow capacity	
full box outlet velocity		1.00 m/s	Outlet V if full flow occured	He, exit velocity head=	0.47 m
outlet critical depth velocity	width*TW depth=	2.22 m/s	Inlet V if partial flow occured	flow depth=	2.5 ft
normal depth velocity		1.31 m/s		from normal depth calc	0.762 m

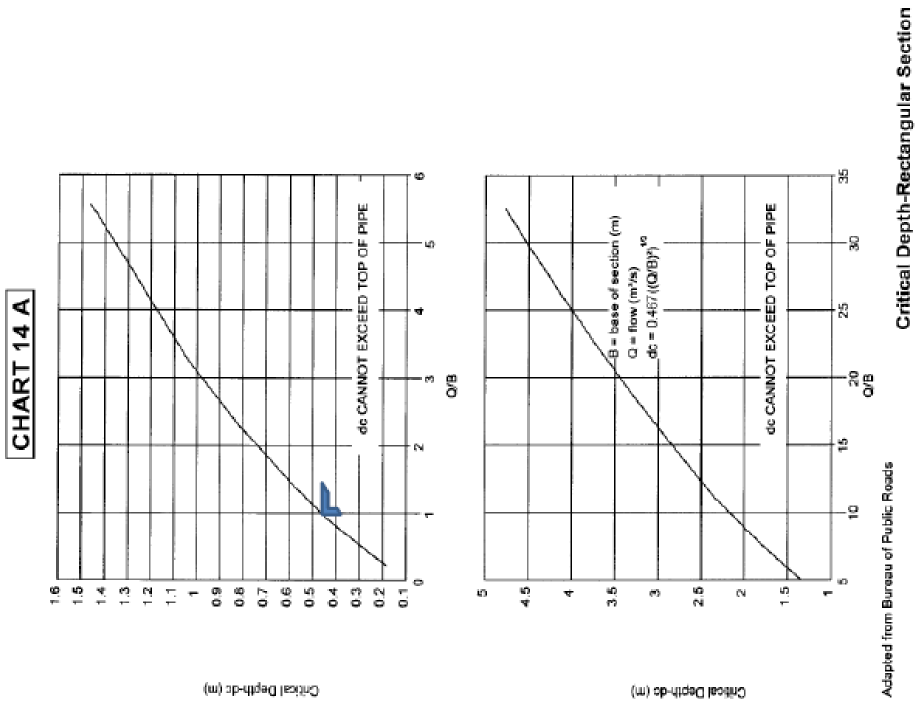
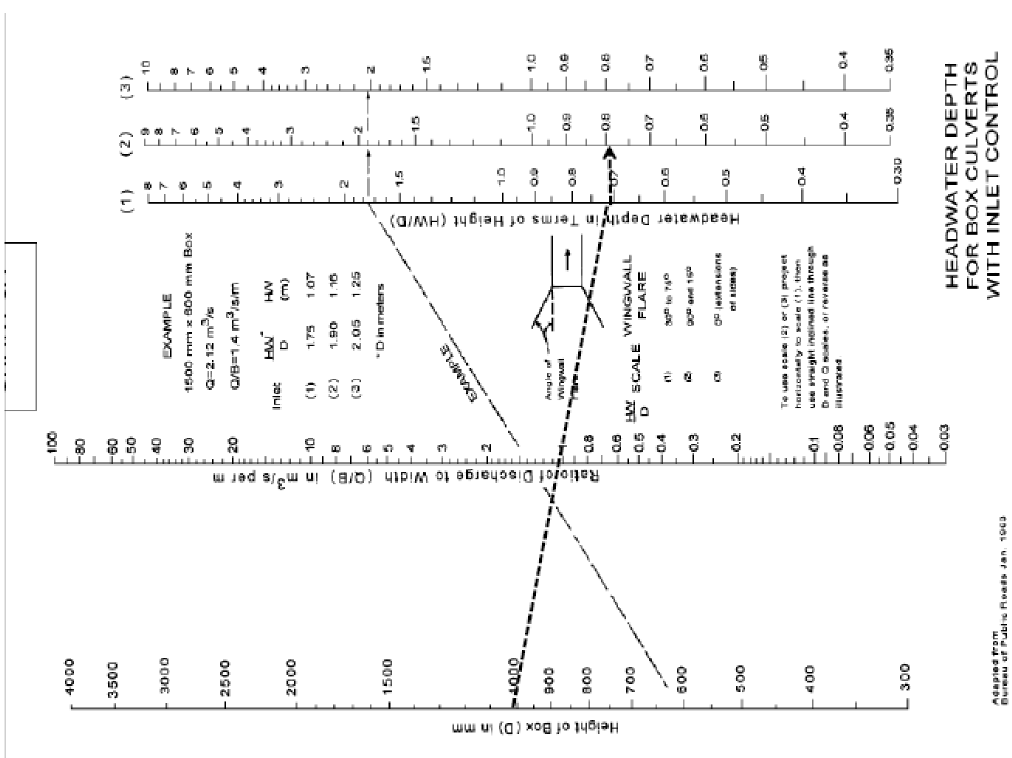
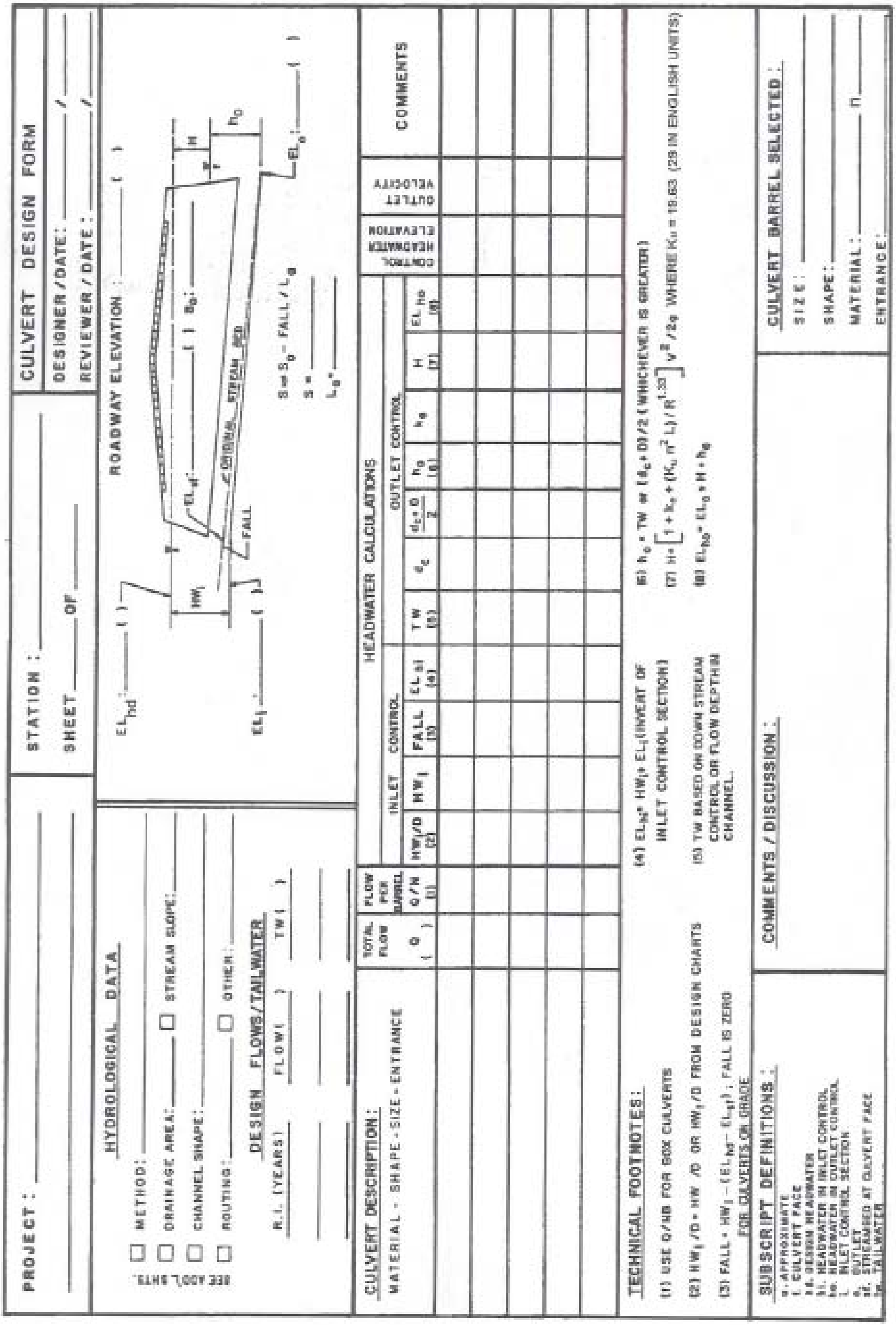


CHART 14 A
Critical Depth-dc (m)
dc CANNOT EXCEED TOP OF PIPE
Q/B
B = base of section (m)
Q = flow (m³/s)
dc = 0.467((Q/B)²)^1/3
Adapted from Bureau of Public Roads
Critical Depth-Rectangular Section


Height of Box (D) in mm
Ratio of Discharge to Width (Q/B) in m³/s per m
Headwater Depth in Terms of Height (HW/D)
EXAMPLE
1500 mm x 800 mm Box
Q=2.12 m³/s
Q/B=1.4 m³/s/m
Inlet HW/D HW (m)
(1) 1.75 1.07
(2) 1.90 1.16
(3) 2.05 1.25
* D in meters
HW/D SCALE WINGWALL FLARE
(1) 30° to 75°
(2) 90° and 15°
(3) 0° (extensions of sides)
To use scale (2) or (3) project horizontally to scale (1), then use straight inclined line through D and Q scales, or reverse as illustrated.
Angle of Wingwall Flare
Adapted from Bureau of Public Roads Jan. 1963
HEADWATER DEPTH FOR BOX CULVERTS WITH INLET CONTROL

Culvert Design Form



CULVERT DESIGN FORM

PROJECT: ______

STATION: ______

SHEET ____ OF ____

DESIGNER / DATE: ______ / ______

REVIEWER / DATE: ______ / ______

HYDROLOGICAL DATA

SEE ADD'L SHTS.

- ☐ METHOD: ______
- ☐ DRAINAGE AREA: ______ ☐ STREAM SLOPE: ______
- ☐ CHANNEL SHAPE: ______
- ☐ ROUTING: ______ ☐ OTHER: ______

DESIGN FLOWS/TAILWATER

R.I. (YEARS)	FLOW ( )	TW ( )

CULVERT DESCRIPTION: MATERIAL - SHAPE - SIZE - ENTRANCE	TOTAL FLOW Q ( )	FLOW PER BARREL Q/N (1)	HEADWATER CALCULATIONS: INLET CONTROL HW_i/D (2)	HW_i	FALL (3)	EL_{hi} (4)	OUTLET CONTROL TW (5)	d_c	$\frac{d_c+D}{2}$	h_o (6)	k_e	H (7)	EL_{ho} (8)	CONTROL HEADWATER ELEVATION	OUTLET VELOCITY	COMMENTS

TECHNICAL FOOTNOTES:

(1) USE Q/NB FOR BOX CULVERTS

(2) $HW_i/D = HW/D$ OR HW_i/D FROM DESIGN CHARTS

(3) $FALL = HW_i - (EL_{hd} - EL_{sf})$; FALL IS ZERO FOR CULVERTS ON GRADE

(4) $EL_{hi} = HW_i + EL_i$ (INVERT OF INLET CONTROL SECTION)

(5) TW BASED ON DOWN STREAM CONTROL OR FLOW DEPTH IN CHANNEL.

(6) $h_o = TW$ or $(d_c + D)/2$ (WHICHEVER IS GREATER)

(7) $H = \left[1 + k_e + (K_u n^2 L)/R^{1.33}\right] V^2/2g$ WHERE K_u = 19.63 (29 IN ENGLISH UNITS)

(8) $EL_{ho} = EL_o + H + h_o$

SUBSCRIPT DEFINITIONS:

- a. APPROXIMATE
- f. CULVERT FACE
- hd. DESIGN HEADWATER
- hi. HEADWATER IN INLET CONTROL
- ho. HEADWATER IN OUTLET CONTROL
- i. INLET CONTROL SECTION
- o. OUTLET
- sf. STREAMBED AT CULVERT FACE
- tw. TAILWATER

COMMENTS / DISCUSSION:

CULVERT BARREL SELECTED:

SIZE: ______

SHAPE: ______

MATERIAL: ______ n ______

ENTRANCE: ______

۱- د اوبو عمق معملوکړی

۲- وخیم عمق معلوم کړی dc=[(Q/D)^2/ g]^1/3

۳- (dc+D)/2

۴- ho= TW= (dc+D)/2

۵ – انتخاب کړی د دخول ضریب (ke) له جدول څخه

۶- د پلچک ضایعات معلوم کړی

H= [1+Ke+(1963*n^2)/R^1/3]*(V^2/2g)

V= Q/A R= A/P P=wetted parameter

۷- د اوبو دخروج د ضرورت وړ ارتفا ع معلومه کړی

$EL_o= El_i- (S_o*L)$

۸- د اوبو دډیزاین جګه ارتفاع معلومه کړی

$EL_{hi}= HW_i+EL_i$

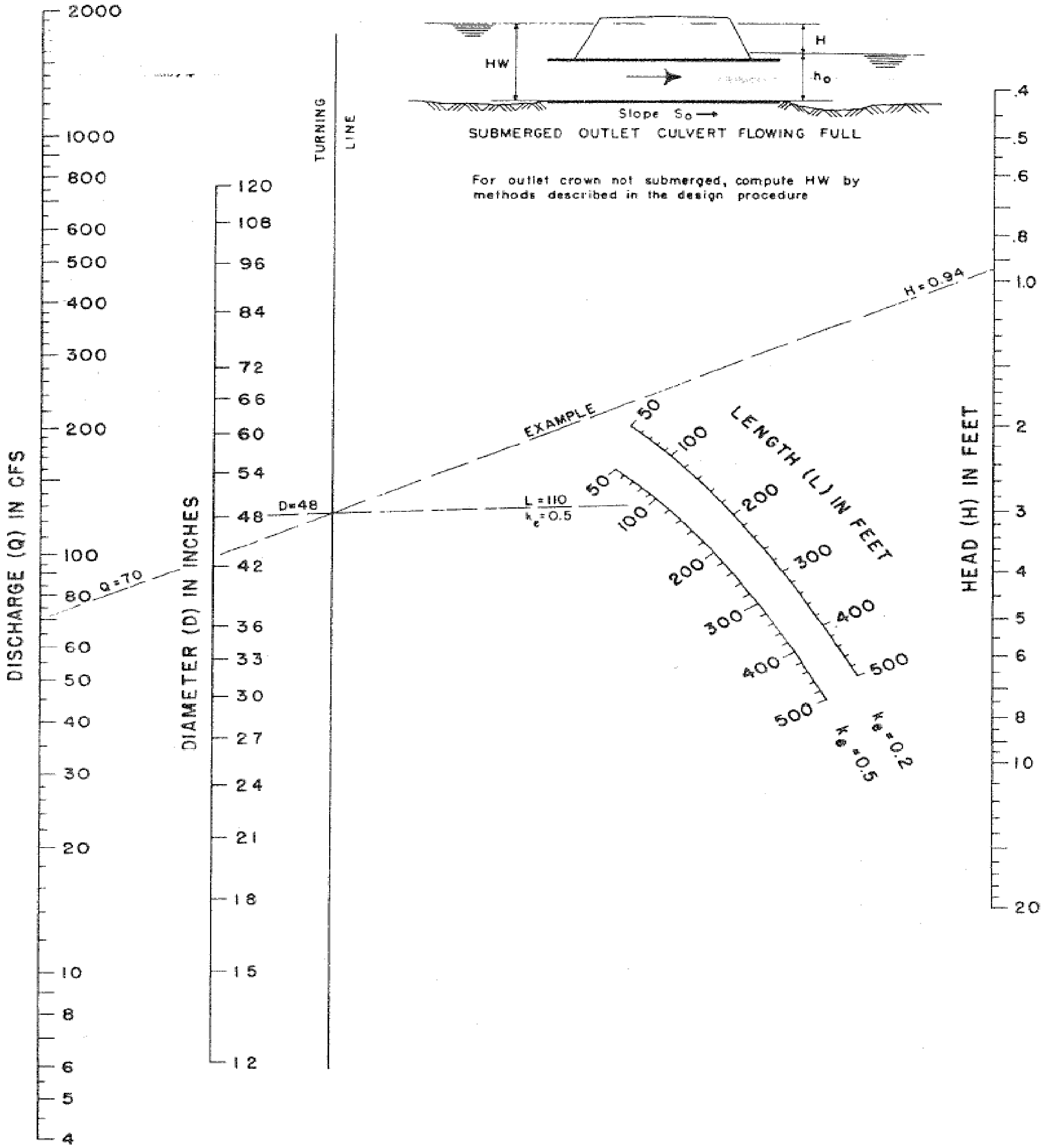
$HW_i= D[HW/D]$

HW/D= C[Q/AD^0.5]^-2 +Y+Z

94		designe steps				
95	step 1	determin tailwater depth wich is abtain form normal depth calculation				
96						
97	step2	critical depth =	dc= ((Q/D)^2/g)^1/3			
98	step 3	$(d_c+D)/2$				
99	step 4	ho= TW or (dc+D)/2				
100	step 5	slect ke form table 16.2.3			entrace loos coeffeceint	
101	step 6	determine head looses through the barrel =H= [1+Ke+(19.63*n^2)/R^1.33]*(v^2/2g)				
102		v= Q/A	R=A/P	P= enviroment of culvert		
103	step 7	determine required outlet elevation			$EL_0=EL_i - S_0 *L$	
104	step 8	determine the design head water elevation				$EL_{hi}= Hw_i+EL_i$
105		Hwi= D[HW/D]		HW/D= C[Q/AD^0/5]^-2+Y+Z		

74							
75		given data					
76	د لسو کلونو د جریان دیزاین	10-year design folw =		3.23928	cms	from abave caculation		
77	د سرک د اوږو ارتفاع	road sulder elevation		34.595	m	form survey		
78		Design Head Elevation, ELI		33.528	m	from survey		
79		Channel invert levation,Eli=		30.48	m	from survey		
80	تعداد د بیلر (پلچک)	Number of barrels		1		from survey		
81	د پلچک د تخت میلان	Stream bed slope, So=		0.458	m/m	from survey		
82	تقریبی طول د پلچک	Approx Culvert length, La=		76.2	m	from survey		
83	د اوبو کښینا ستل	Fall=Hw_i- HW_d		0	m			
84		Outlet elev, ELo=		-4.4196	m			
85		S=So-(Fall/La)=		0.458				
86		Hwi=Elhd-Eli=		3.048	m			
87		Box Mannings n=		0.013				
88								
89								
90								
91		Trial #1 Box width, B=		1	m			
92		Trial #1 Box height, D=		1	m			
93		Area of box=		1	sqm			
94		R=A/P		0.25			P=(2*B)+(2*D)	A=B*D
95		Inlet type=	squre					

1	Q /barrel=Q/(N*B)=	3.26928	cm/sm	N= nose of barrel B= wide of barrel	
2.a	Hwi/D	3.048		from chart A8 acurdance to Q/B(sqm/sca)	
2.b	Hwi	3.048	m		
3	Fall	0			
4	Elhi=Hwi+Eli	33.528	m	inlet control head water elevation	
5.a	TW	0.51	m		
5.b	dc	1	m	chart 14A acurdance to Q/B(sqm/sca	
5.c	(dc+D)/2	1	m		
6.a	greater TW or (dc+D)/2,Ho=He		1	m	*Reference: U.S. Department of Transportation Federal Highway Administration.*
6.b	ko=	0.5		haydrolic	
7	H=	28.87	m	by this (H=[1+Ke+(ku*n^2*L/R^1.33)]*(V^2/2g) farmula	
8 outlet control elvation	Elho=Elo+H+He	25.45	m		
inletecontrol headwater elevation		33.528			
full box outlet velocity	Q/(B*D)	3.26928	m/s		
inlet control outlet velocity	width*TW depth=	0.51	m/s		



HEAD FOR
CONCRETE PIPE CULVERTS
FLOWING FULL
$n = 0.012$

BUREAU OF PUBLIC ROADS JAN. 1963

Design of Culverts

Table 4-2 Inlet Coefficients

Type of Structure and Design of Entrance	Coefficient K_e
Pipe, Concrete	
Projecting from fill, socket end (grove-end)	0.2
Projecting from fill, square cut end	0.5
Headwall or headwall and wingwalls	
Socket end of pipe (groove-end)	0.2
Square-edge	0.5
Rounded [radius = 1/12(D)]	0.2
Mitered to conform to fill slope	0.7
*End-Section conforming to fill slope	0.5
Beveled edges, 33.7° or 45° bevels	0.2
Side- or slope-tapered inlet	0.2
Pipe, or Pipe-Arch, Corrugated Metal	
Projecting from fill (no headwall)	0.9
Headwall or headwall and wingwalls square-edge	0.5
Mitered to fill slope, paved or unpaved slope	0.7
*End-Section conforming to fill slope	0.5
Beveled edges, 33.7° or 45° bevels	0.2
Side- or slope-tapered inlet	0.2
Box, Reinforced Concrete	
Headwall parallel to embankment (no wingwalls)	
Square-edged on 3 edges	0.5
Rounded on 3 edges to radius of [1/12(D)] or beveled edges on 3 sides	0.2
Wingwalls at 30° to 75° to barrel	
Square-edged at crown	0.4
Crown edge rounded to radius of [1/12(D)] or beveled top edge	0.2
Wingwalls at 10° or 25° to barrel	
Square-edged at crown	0.5
Wingwalls parallel (extension of sides)	
Square-edged at crown	0.7
Side- or slope-tapered inlet	0.2

* Note: End Sections conforming to fill slope, made of either metal or concrete, are the sections commonly available from manufacturers. From limited hydraulic tests they are equivalent in operation to a headwall in both inlet and outlet control. Some end sections incorporating a closed taper in their design have a superior hydraulic performance.

Source: HDS:5

4.5.14 Manning's n Values

For culvert selection, only reinforced concrete pipe is allowed within City street right-of-way except for driveway culverts. For culverts equal to or greater than 60 inches in diameter, corrugated metal pipe is allowed if it is bituminous coated with a concrete-poured invert. Table 4-3 gives recommended Manning's n values.

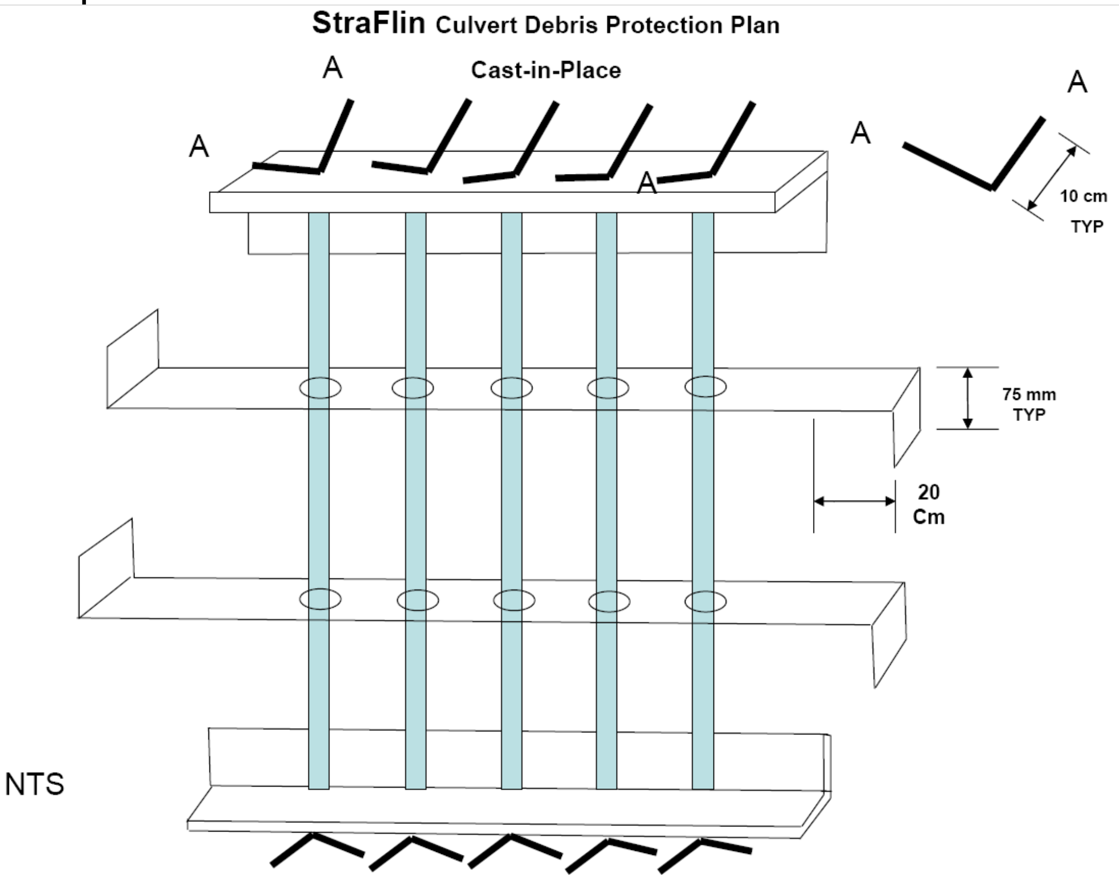
Table 4-3 Manning's n Values

Type of Conduit	Wall & Joint Description	Manning's n
Concrete Pipe	Good joints, smooth walls	0.011-0.013
	Good joints, rough walls	0.014-0.016
	Poor joints, rough walls	0.016-0.017
Concrete Box	Good joints, smooth finished walls	0.014-0.018
	Poor joints, rough, unfinished walls	0.014-0.018
Corrugated Metal Pipes and Boxes, Annular Corrugations	2 2/3 by 1/2-inch corrugations	0.027-0.022
	6 by 1-inch corrugations	0.025-0.022
	5 by 1-inch corrugations	0.026-0.025
	3 by 1-inch corrugations	0.028-0.027
	6 by 2-inch structural plate	0.035-0.033
	9 by 2 1/2-inch structural plate	0.037-0.033
Corrugated Metal Pipes, Helical Corrugations, Full Circular Flow	2 2/3 by 1/2-inch corrugated 24-inch plate width	0.024-0.012
Spiral Rib Metal Pipe	3/4 by 3/4-inch recesses at 12-inch spacing, good joints	0.012-0.013

Note: For further information concerning Manning n values for selected conduits, consult Hydraulic Design of Highway Culverts, Federal Highway Administration, HDS No. 5, page 163.

Appendix C – Culvert Debris Protection Devices

Site Adapt STRAFLIN



StraFlin Culvert Debris Protection Plan

Cast-in-Place

Bill of Material

A. 75mm X 75mm X 9mm ASTM A36 Grade B, Angle, Hot Dip Galvanized, Cut to Length (2 Pieces)

B. 50 mm X 9mm ASTM A36 Grade B, Flat Bar, Hot Dip Galvanized, Cut to Length (2 Pieces)

C. 20 mm ASTM A36 Grade B, Cold Rolled Round Bar, Hot Dip Galvanized, Cut to Length, 5 Pieces

D. Reinforcing Bar, Number 10, ASTM A615/A615M-05a , Cut to Length, (10 Pieces)

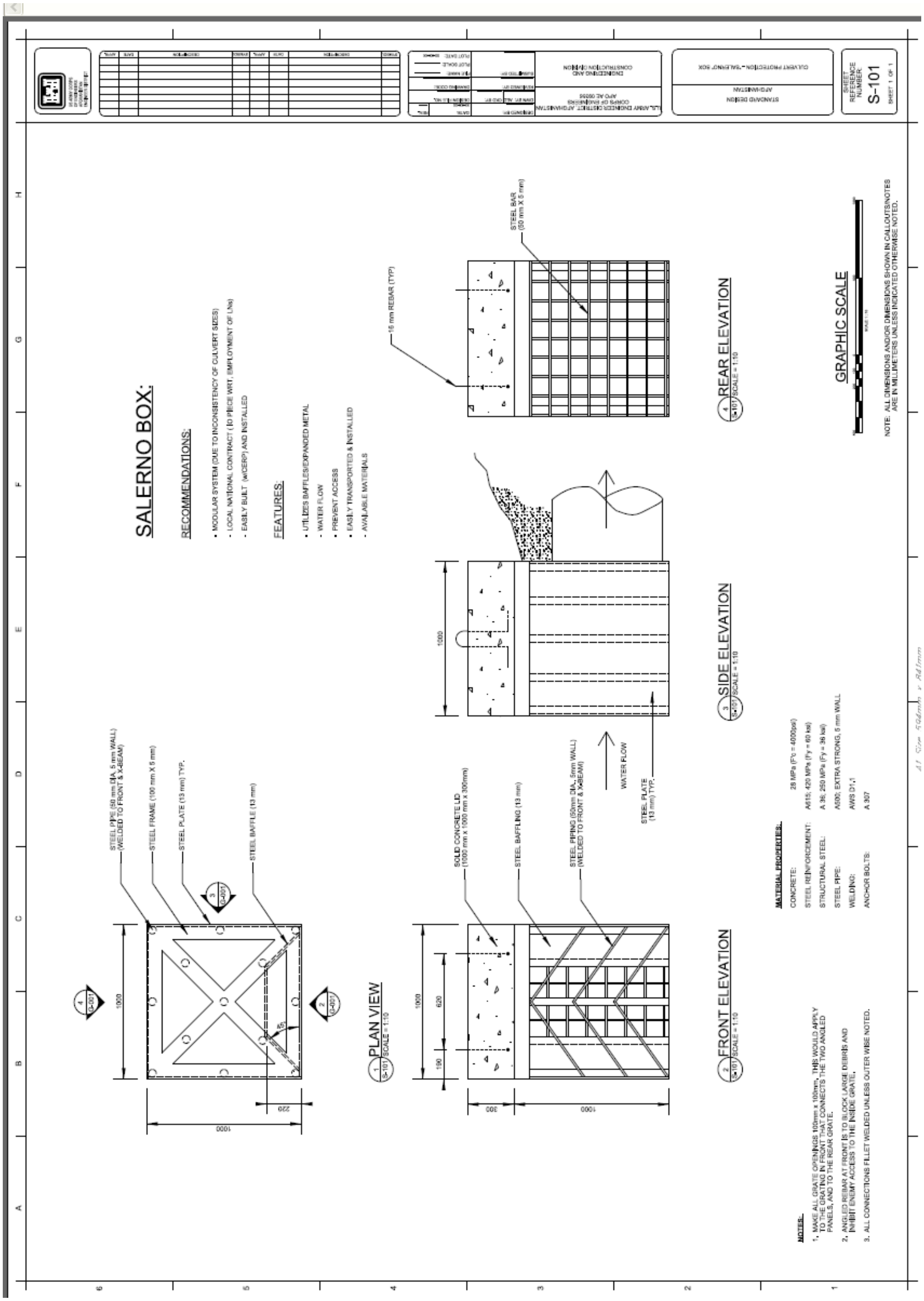
Designed by:

William Stratton

CPT Darrell Flinn

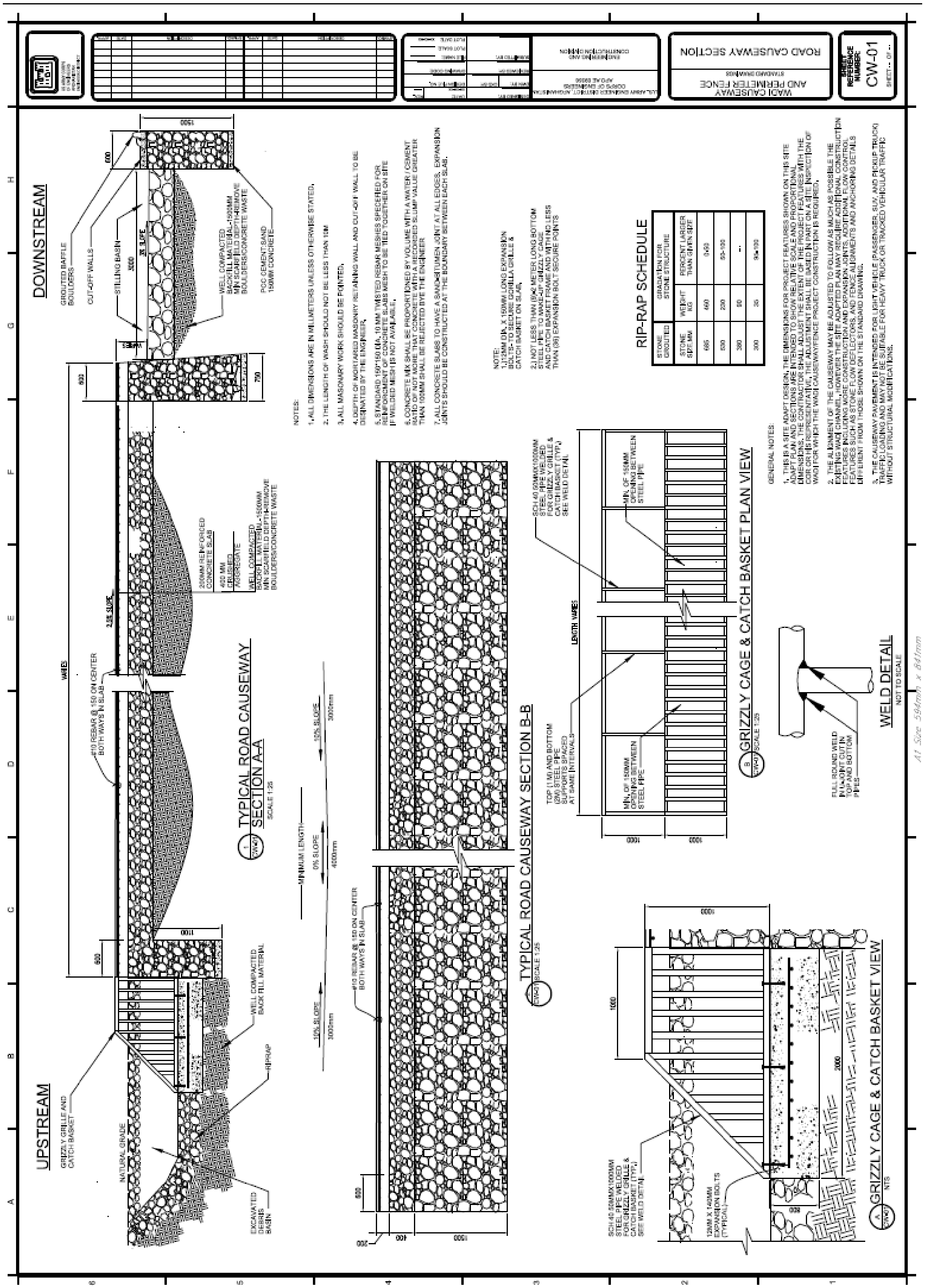
Mehtarlam PRT

Site Adapt SALERNO BOX

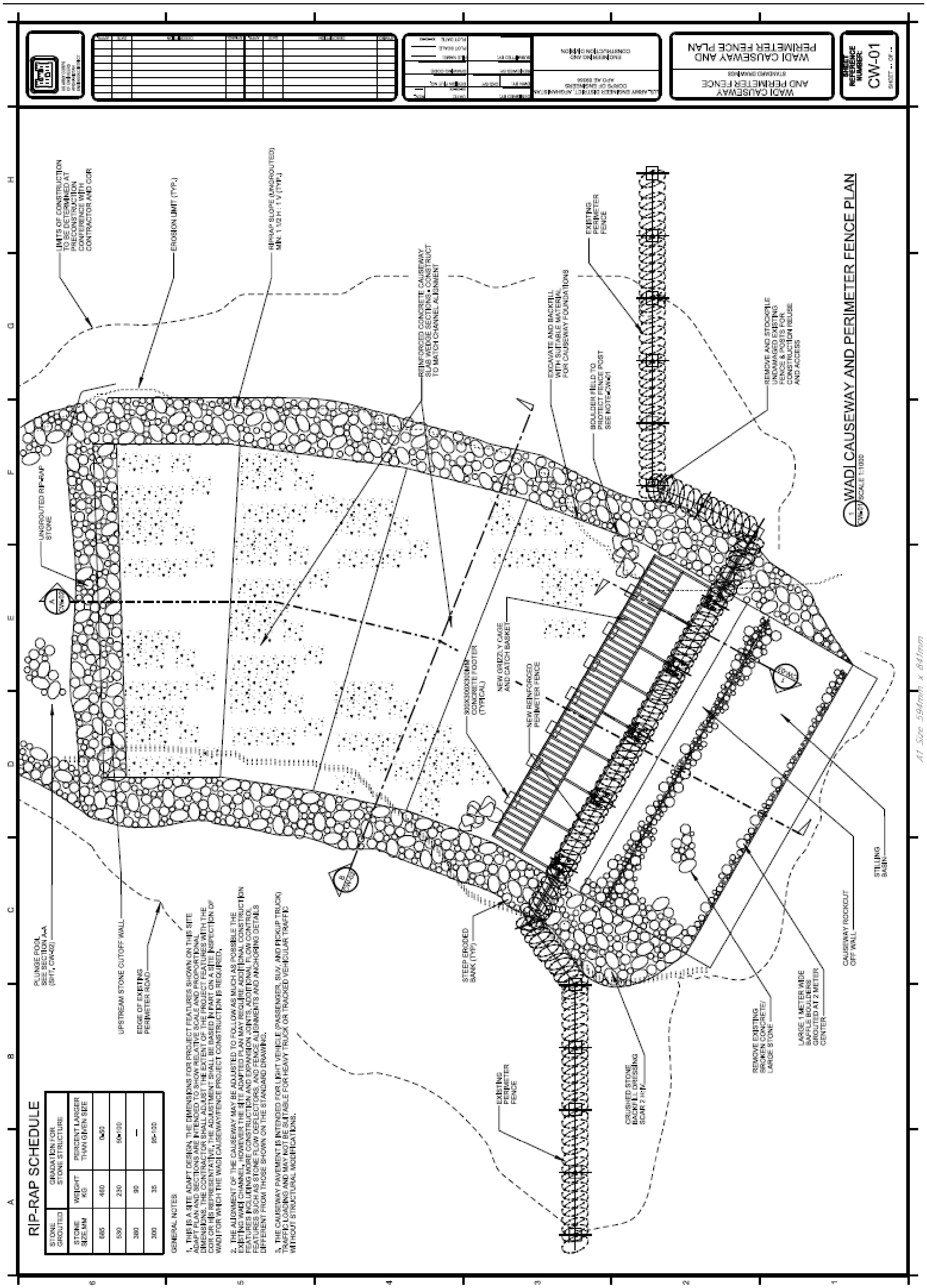


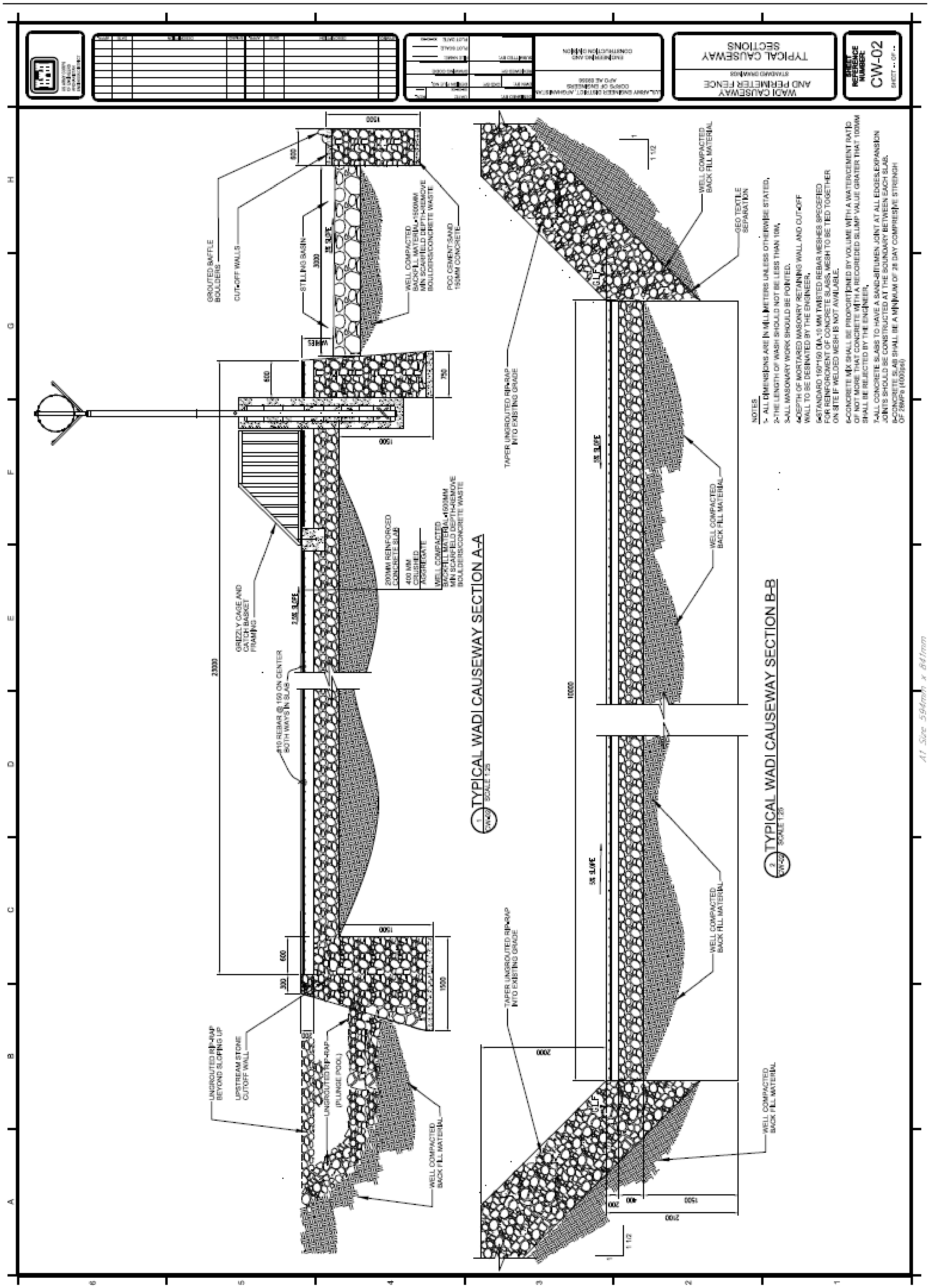
Appendix D – Causeway Debris Protection Devices

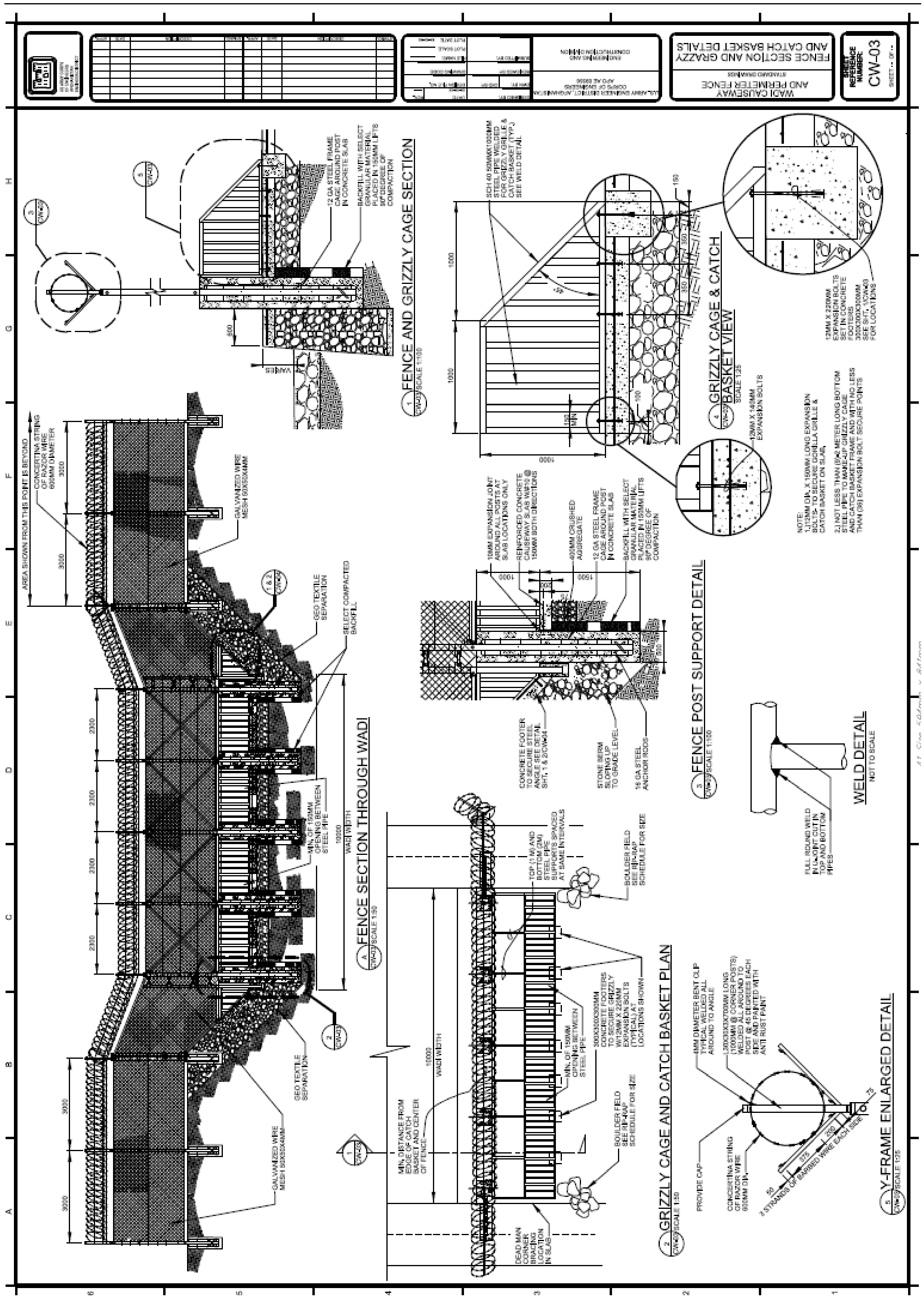
Site Adapt Roadway Causeway with Grizzly Cage

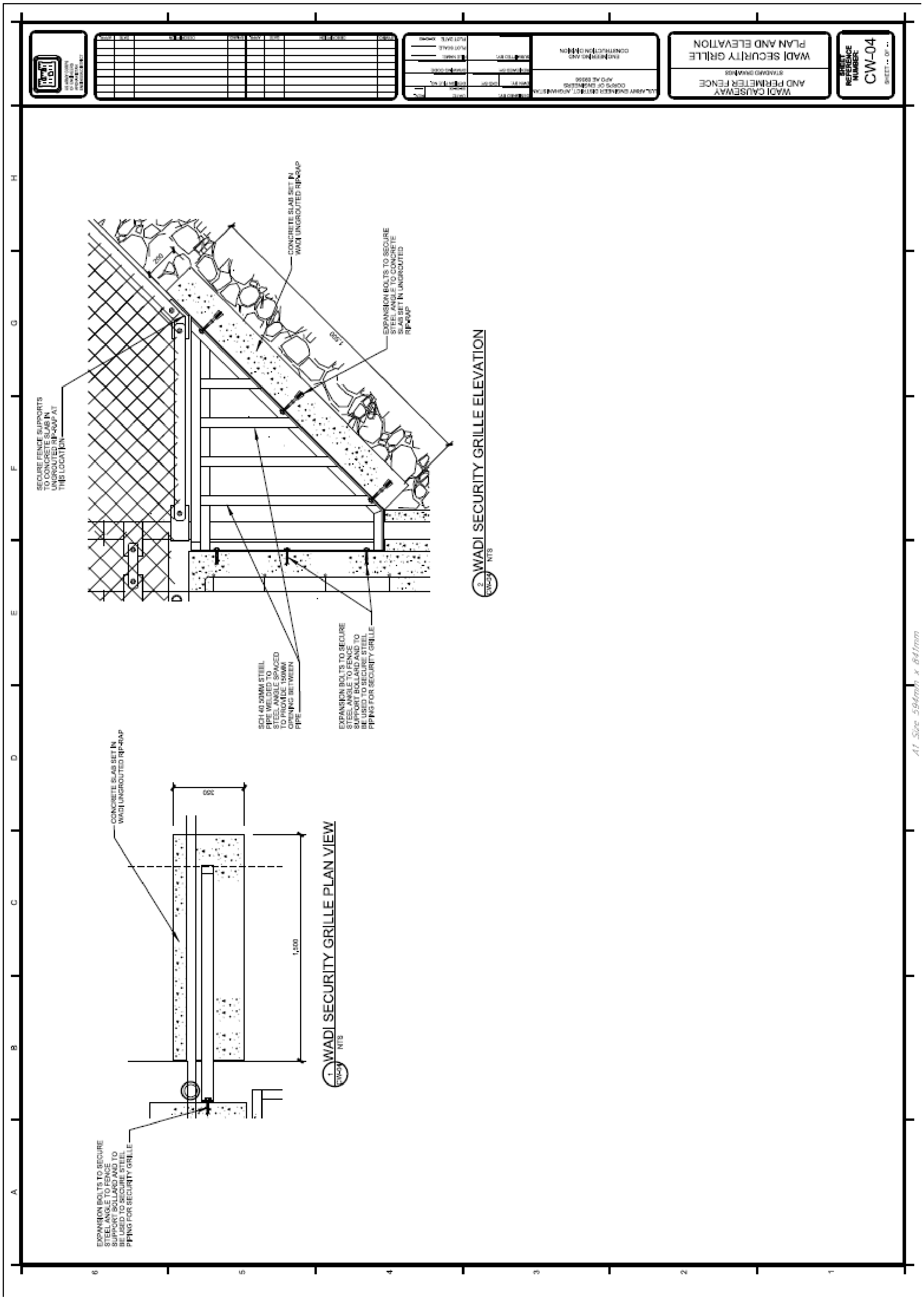

DOWNSTREAM
UPSTREAM
TYPICAL ROAD CAUSEWAY SECTION A-A
TYPICAL ROAD CAUSEWAY SECTION B-B
RIP-RAP SCHEDULE
GRIZZLY CAGE & CATCH BASKET PLAN VIEW
GRIZZLY CAGE & CATCH BASKET VIEW
WELD DETAIL
ROAD CAUSEWAY SECTION
CW-01

Site Adapt Perimeter Fence Wadi Causeway with Grizzly Cage




TYPICAL WADI CAUSEWAY SECTION A-A
TYPICAL WADI CAUSEWAY SECTION B-B
WADI CAUSEWAY AND PERIMETER FENCE
TYPICAL CAUSEWAY SECTIONS
CW-02


WADI CAUSEWAY
AND PERIMETER FENCE
FENCE SECTION AND GRAZZY
AND CATCH BASKET DETAILS
CW-03
FENCE AND GRIZZLY CAGE SECTION
GRIZZLY CAGE & CATCH
BASKET VIEW
FENCE SECTION THROUGH WADI
FENCE POST SUPPORT DETAIL
WELD DETAIL
GRIZZLY CAGE AND CATCH BASKET PLAN
Y-FRAME ENLARGED DETAIL


WADI CAUSEWAY AND PERIMETER FENCE
WADI SECURITY GRILLE PLAN AND ELEVATION
CW-04
CONCRETE SLAB SET IN WADI UNGROUTED RIP-RAP
EXPANSION BOLTS TO SECURE STEEL ANGLE TO CONCRETE SLAB SET IN UNGROUTED RIP-RAP
SECURE FENCE SUPPORTS TO CONCRETE SLAB IN UNGROUTED RIP-RAP AT THIS LOCATION
SCH 40 50MM STEEL PIPE WELDED TO STEEL ANGLE SPACED TO PROVIDE 150MM OPENING BETWEEN PIPE
EXPANSION BOLTS TO SECURE STEEL ANGLE TO FENCE SUPPORT BOLLARD AND TO BE USED TO SECURE STEEL PIPING FOR SECURITY GRILLE
1,500
WADI SECURITY GRILLE ELEVATION
NTS
WADI SECURITY GRILLE PLAN VIEW
NTS
A1 Size 594mm x 841mm

US Army Corps
of Engineers
Afghanistan Engineer District

AED Design Requirements: Geotechnical Investigations (Provisional)

Various Locations,
Afghanistan

January 2010, Version 1.0

TABLE OF CONTENTS

AED DESIGN REQUIREMENTS
FOR
GEOTECHNCIAL INVESTIGATIONS
VARIOUS LOCATIONS,
AFGHANISTAN

1.0 GENERAL

This design guide presents guidance for planning, performing, and reporting soil investigations. All information presented within this document is summarized from Unified Facilities Criteria (UFC) and Engineering Manuals (EM). It is not a comprehensive textbook on soil sampling; the treatment of this subject cannot be substituted for actual experience. Rather, it is a summary of commonly accepted soil sampling practices and procedures which are intended to assist geotechnical personnel performing actual field operations or those personnel functioning as contracting officers' representatives. Soils investigations for USACE-AED projects are intended to provide the design engineer various design parameters including:

a. Revealing adverse subsurface conditions that could lead to construction difficulties, excessive maintenance, or failure of the structure

b. Depth required for satisfactory bearing material for foundations

c. Bearing capacity of soil for building or other structural foundations

d. Potential range of settlements of buildings

e. Allowable California Bearing Ratio (CBR) for road bed design

f. Optimum moisture content at optimum compaction

g. Depth of groundwater for dewatering and buoyancy design considerations

h. Minimum embankment side slope and benching required for cuts and fills

i. Erosion protection requirements

j. Expected long term settlements of high embankments and placement and compaction procedures to minimize adverse impact to project functions

k. Active and passive pressures for retaining walls and revetments

l. Internal friction angles and cohesive strengths of soil for analyses

m. Water-soluble sulfate in soil (ASTM C 1580). Percolation test results (U.S. Environmental Protection Agency guidance is available from AED).

Additional design parameters may be requested in the contract technical requirements.

This design guide is intended to provide contractors information on the expected investigations and methods required for geotechnical reports for USACE-AED projects. It should provide USACE-AED with standardized sets of information from which a reliable data base can be maintained for future use in project development. It is intended to eliminate submittal of information that USACE neither requests in the contract technical requirements nor uses in its evaluation of project designs; thereby also reducing costs for supplying this information from contractors. Insufficient geotechnical investigations, faulty interpretation of results, or failure to portray results in a clearly understandable manner may contribute to inappropriate designs, delays in construction schedules, costly construction modifications, use of substandard borrow material, environmental damage to the site, post construction remedial work, and even failure of a structure and subsequent litigation.

2.0 PLANNING OF SITE GEOTECHNICAL INVESTIGATION

In order to properly characterize the sub surface soils at any site for any project a geotechnical investigation plan is crucial to accommodate any situation that may arise once the investigation has started. Due to security issues here in Afghanistan, proper planning will reduce risks to life, limb and equipment by ensuring only one site visit is necessary to capture site specific information.

2.1 SITE SPECIFIC INFORMATION

The logical and necessary first step of any field investigation is the compilation of all pertinent information on geological and soil conditions at and in the vicinity of the site or sites under consideration, including previous excavations, material storage, and buildings. A geotechnical investigation plan also involves collection of information such as project type, location, and purpose of the structure or facility. The project type will control the depth and number of boreholes that are required. For instance, a road construction project will require numerous boreholes at a shallow depth where a vertical construction project will require boreholes that are drilled to depths sufficient to characterize the soils within the zone of influence and below.

2.2 DEPTH, LOCATION, AND SPACING OF EXPLORATION

Depths of explorations shall be determined by the type of structure to be constructed on site. Major structures shall have a minimum of 3 borings within the footprint and 5 for variable soil conditions and earthquake zone. The minimum depth shall be the greater of 6 meters or twice the height of the structure. Minor structures shall have 1 to 3 test pits, 1 test pit for every 225 square meters. The test pits shall be a minimum of 3 meters in depth. Major structures shall be defined as reinforced concrete structures greater than 1000 square meters, steel frame buildings greater than 3000 square meters, structures with a height greater than or equal to 1.5 stories, and steel or concrete water tanks greater than 350 square meters. Minor structures are defined as all smaller structures than Major structures. As a minimum for airfield pavements, the contractor shall excavate 3 test pits for pavements less than or equal to 5,000 square meters and 1 test pit for each additional 5,000 square meters of pavement or fraction thereof. As a minimum for all other pavements, except roads, the contractor shall excavate 3 test pits for pavements less than or equal to 7,500 square meters and 1 test pit for each additional 5,000 square meters of pavement or fraction thereof. As a minimum for roads, the contractor shall excavate 3 test pit for pavements less than or equal to 200 linear meters and 1 test pit for each additional 200 linear meters or fraction thereof.

2.3 MINIMUM SAMPLING AND TESTING

a. Unified soil classification system (USCS) tests shall be performed for every sample including all accompanying tests to determine soil type. SPT blow counts and USCS shall be performed every 0.75 meters.

b. Direct shears for ML, SC, SP, SW, and SC shall be conducted; 3 for each soil type encountered from samples taken at preferably 1 meter in depth.

c. 3 unconfined compressive strength and consolidation tests shall be performed for each fine grained soil type encountered at the site, preferably at 1 meter of depth.

d. 6 percolation tests distributed over the proposed absorption field at 0.5 meters.

e. Minimum 1 soluble sulfate test to a maximum of 4 per site at 0.5 meters.

2.3.1 ROADS

In determining subgrade conditions, borings will be carried to the depth of frost penetration, but no less than 1.8 meters (6 feet) below the finished grade. In the design of some high fills, it may be necessary to consider settlement caused by the weight of the fill. This results in the need for borings deeper than 1.8 meters to determine soil characteristics where compressive soils may or are suspected to exist at greater depth. Specific needs for pavement design will be directed by the designer-of-record. For flexible pavement design, soaked California bearing ratio (ASTM D 1883) will likely need to be determined as a function of soil dry density for each subgrade soil type. For rigid pavement design, modulus of subgrade reaction will likely need to be determined by plate bearing tests (ASTM D 1196). These values are correction for saturated soil conditions, in accordance with CRD-C 655 (USACE Standard Test Method).

2.4 WATER TOWERS

The geotechnical characteristics of the soils at any location water towers will be constructed must be adequate to support the deep loadings and extended zone of influence generated by the tower and water tank. This will require a borehole exploration. Test pits will not be allowed for collection of subsurface data for the water tower. The minimum depth of borehole will be 16.5 meters (54 feet) below the finished grade in the center of the proposed tower location. A borehole log and results of testing defined below will be recorded and submitted as part of the geotechnical report.

2.4.1 MINIMUM SAMPLING AND TESTING FOR WATER TOWERS

a. At least one SPT test should be recorded for each 0.75 meters elevation change down to 16.5 meters. Visual classification shall be performed at this time (in accordance with ASTM D 2488) and recorded on the boring log. The field boring log shall be included in the geotechnical report submittal. The depth of water table shall also be recorded if found and reported on the log.

b. Unified soil classification system (USCS) tests shall be performed for every sample including all accompanying tests to determine soil type.

c. Atterberg Limits (PL, LL, PI) tests shall be determined and reported for every sample.

d. Moisture content shall be determined and reported for every sample.

e. Unit weights of samples shall be determined and reported for every sample.

3.0 GEOTECHNICAL SITE INVESTIGATION

There is no set standard for a geotechnical investigation as every site is different and every project is different. The site investigation for the most part should follow the geotechnical investigation plan unless conditions arise that require alteration of the plan in the field. All changes and the logic used to adjust the plan shall be reported in the final geotechnical report.

3.1 SAMPLING

Depending on the material expected to be encountered at the project site from the information gathered in the compilation of pertinent data the geotechnical investigation plan shall provide a minimum number of disturbed samples to be collected. If cohesive soils are expected to be encountered during the investigation then the minimum number of undisturbed samples shall be collected. Samples shall be taken at not more than 0.75 meter (2.5 feet) intervals. The number of samples shall be determined by taking the required amount of borings, multiplying the total depths of each boring, and dividing by 0.75 meters.

3.2 UNDISTURBED SAMPLES

Undisturbed samples have been subjected to relatively little disturbance and may be obtained from borings using push-type or rotary-core samplers. High-quality undisturbed samples may be obtained by hand trimming block samples from test pits and trenches. Undisturbed samples are useful for strength, compressibility, and permeability tests of the foundation materials. Undisturbed sampling should be conducted in a manner to minimize: (a) changes of void ratio and water content, (b) mechanical disturbance of the soil structure, and (c) changes of stress conditions. Efforts should also be undertaken to eliminate other causes of disturbance, such as freezing and chemical changes, caused by prolonged storage in metal containers. Any method of taking and removing a sample that results in a stress change, possible pore water change, and some structure alteration because of displacement effects of the sampler is not acceptable. Careful attention to details and use of proper equipment can reduce disturbance to a tolerable amount. Sample disturbance is related to the area ratio A_r, through which the sample passes (commonly the cutting edge is swedged to a lesser diameter than the inside tube wall thickness to reduce friction) defined as follows:

$$A_r = (D_o^2 – D_1^2 / D_1^2) \times 100 \text{ percent} \qquad \text{Eq. 2}$$

Where:

D_0 = outside diameter of sampler tube, mm

D_1 = internal diameter of the cutting shoe, mm

The area ratio should be less than 10 percent for undisturbed sampling. Undisturbed samples are commonly taken by thin-wall seamless steel tubing from 50 to 75 mm (2 to 3 inches) in diameter and lengths from 0.61 to 1.2 meters (2 to 4 feet). Undisturbed samples for shear, unconfined compression, and consolidation testing are commonly 75 mm (3 inches) in diameter, but 125 mm (5 inches) diameter samples are much preferred. An indication of sample quality is the recovery ratio, Lr , defined as follows:

$$Lr = \frac{\text{Length of recovered sample}}{\text{Length sample tube pushed}}$$

A value for Lr < 1 indicates that the sample was compressed or lost during recovery, and Lr > 1 indicates that the sample expanded during recovery or the excess soil was forced into the sampler.

The following table shall be used to determine the minimum size of sample necessary for an undisturbed sample:

Table 1 Minimum sample sizes

Test Sample that is Collected for:	**Minimum Sample Diameter, mm (in.)**
Unit weight	75 (3)
Consolidation	125 (5)
Unconfined compression	75 (3)
Direct shear	125 (5)

Undisturbed samples must be handled and preserved in a manner to preserve stratification or structure, water content, and in situ stresses, to the greatest extent possible. Once the sample has been removed from the borehole, it must be either sealed within the sampling tube or extruded and sealed within another suitable container prior to shipment to the laboratory for testing. In general, carbon steel tubes should not be used if the samples are to be stored in the tubes for an extended period of time (i.e. more than 48 hours) because the tubes will rust or corrode and may contaminate the sample. If extended storage is required, containers made of alternative metals or wax-coated cardboard tubes should be considered. Samples for water content determination must be sealed to prevent changes of soil moisture. If glass jars are used, the gasket and the sealing edge of the container must be clean to ensure a good seal. Guidance for preservation and shipment of samples is given in ASTM Standard D 4220-83 (ASTM 1993). The most common test method for undisturbed samples in this country is the unconfined compression test (ASTM D 2166). Specimens shall have a minimum diameter of 30 mm (1.3 in.) and the largest particle contained within the test specimen shall be smaller than one tenth of the specimen diameter. For specimens having a diameter of 72 mm (2.8 in.) or larger, the largest particle size shall be smaller than one sixth of the specimen diameter.

For each building structure, samples should be obtained at 3 locations and at 2 to 3 depths for each location: 1 m below surface, 3 m below surface and at the depth of any different soil layer between 1 and 3 m.

3.2.1 ADVANCING THE BOREHOLE

Position the drill rig over the sampling location, chock (secure) the wheels on the drill rig, and adjust the mast to a vertical position. Place the drill rods and related equipment at a convenient location for use with respect to the drill rig. The drill rods, sampling equipment, casing, etc., may be placed about 5 m (15 ft) from the drill rig. The inspector's work station, areas for sample storage, power units, support vehicles, and other equipment can be located at greater distances, depending on the type of drilling and/or sampling operations, site topography, weather conditions, logistics, etc.

After a vertical pilot hole has been established, attach the drill bit or auger to the drill rod and lower the string into the borehole. Attach more drilling rods or auger flights, as necessary. The depth of the borehole is advanced to the desired depth by rotating the auger or bit and applying a downward pressure from the drill rig or by gravity feed as required to achieve a satisfactory penetration rate. After the hole has been advanced to the desired sampling depth, remove the excess cuttings from the bottom of the hole before the drill string is withdrawn. As the string is withdrawn, disconnect the sections of rod and lay aside. Repeat until the cutting head is retrieved.

When lowering the equipment into the borehole to a new sampling depth, repeat the above procedures in reverse order. Count the number of rods to determine the depth of the borehole. Carefully monitor and record the depth of the hole for use during the sampling operations. All trips up and down the borehole with the drill string should be made without rotation.

In order to obtain an undisturbed soil sample, a clean, open borehole of sufficient diameter must be drilled to the desired sample depth. The sample should be taken as soon as possible after advancing the hole to minimize swelling and/or plastic deformation of the soil to be sampled.

3.2.2 DIAMETER OF THE BOREHOLE

The diameter of the borehole should be as small as practical. If casing is not used, a borehole 6 to 19 mm (1/4 to 3/4 in.) greater in diameter than the outside diameter of the sampler should be sufficient. In soils containing irregular hard and soft pockets that cause deviation of the drill bit. In soils that tend to squeeze, a slightly larger clearance, i.e., 12 to 19 mm (1/2 to 3/4 in.), may be required. When casing is used, the hole should be drilled 6 to 25 mm (1/4 to 1 in.) larger than the outside diameter (OD) of the casing.

3.2.3 METHODS OF ADVANCE

Boreholes for undisturbed samples may be advanced by rotary drilling methods or with augers. Augers are discussed in Chapters 3, 7, and 8 of EM 1110-1-1804 “GEOTECHNICAL INVESTIGATIONS”. **Displacement and percussion methods for advancing boreholes are not acceptable for undisturbed sampling operations.**

3.3 DISTURBED SAMPLES AND QUANTITIES

Boreholes for disturbed soil samples, obtained by split-barrel samplers, may be advanced in the same manner as those procedures used for boreholes for undisturbed soil samples. Other acceptable procedures for advancing boreholes are listed in ASTM D 1586. It is not permissible to advance the boring for subsequent insertion of the sampler solely by means of previous sampling with the split-barrel sampler. Disturbed samples are primarily used for moisture content, Atterberg limits, specific gravity, sieve analysis or grain-size distribution, and compaction characteristics. The amount of sample material required is shown in Table 2. Strength and deformation tests may be conducted on reconstituted (remolded) specimens of the disturbed materials. Samples can be obtained by means of auger or drive-sampling methods. Thick-wall, solid, or split-barrel drive samplers can be used for all but gravelly soils. Samples taken with a drive sampler should be not less than 50 mm (2 inches), and preferably 75 mm (3 inches) or more in diameter. Where loose sands or soft silts are encountered, a special sampler with a flap valve or a plunger is usually required to hold the material in the barrel. A bailer can be used to obtain sands and gravel samples from below the water table. Split-spoon samples should be used to obtain representative samples in all cases where the density of cohesionless materials must be estimated.

Table 2 Minimum sample size for disturbed samples

Test	Minimum Sample Required for soils with all material passing the No. 4 sieve. kg (lb)	Minimum Sample Required for all other soils. kg (lb)
Water content	0.227 (0.5)	5 (11)
Atterberg limits	0.091 (0.2)	0.2 (0.44)
Specific gravity	0.091 (0.2)	0.2 (0.44)
Grain-size analysis	0.227 (0.5)	70 (154)
Standard compaction	13.61 (30)	29 (64)
Direct shear	0.907 (2)	8 (17.6)
4-in.-diam consolidation	0.907 (2)	

3.4 STANDARD PENETRATION TESTS

This test is performed in conjunction with split-barrel sampling. It provides a rough approximation of the relative density or consistency of foundation soils and should always be made when piles are to be driven. For a single test results, the split spoon is driven a total of 457 mm; the penetration resistance in blows per foot (or N-value) is based on the last 305 mm; the first 152 mm being to seat the sampler in undisturbed soil at the bottom of the boring. "Refusal" is usually taken at a blow count of 50 per 6 inches. This test must be performed in accordance with ASTM D 1586; that is, the split-barrel sampler must conform to specified dimensions and it must be driven by a 63.5-kg hammer, which drops 0.76 m to impose a “blow.” On large projects, the SPT data should be used in conjunction with tests on undisturbed samples.
At least one SPT test should be recorded for each 0.75 m elevation change down to 4.5 m. Below 4.5 m depth, at least one SPT test is needed for every 1.5 m elevation change

3.5 TEST PITS AND TEST TRENCHES

Test pits and trenches may be excavated by hand or by conventional earth-moving equipment. Shallow pits dug to depths up to 1.2 m (4 ft) in stable soil usually require no shoring; for deeper excavations or pits in unstable soil, shoring must be used (EM 385-1-1). Excavations extending below the water table require control of the groundwater. In impervious or relatively impervious soils, groundwater can be controlled by pumping directly from a sump or drainage ditch in the pit. If the pit extends below the water table in sand or silt, dewatering by means of a well-point system may be necessary to ensure dryness and stability of the pit. If seepage forces are great, blowouts in the bottom or sides of the pit can result.

Test pits are commonly used for exposing and sampling foundation and construction materials. The test pit must be large enough to permit detailed examinations of the material in situ to be conducted or to obtain large, undisturbed samples as required by the investigation. Typically, the plan view of the pit will be square, rectangular, or circular. The minimum dimensions of the pit are on the order of 0.9 by 1.5 m (3 by 5 ft) or 1.2 by 1.8 m (4 by 6 ft); it should be noted that these dimensions are net dimensions at the bottom of the excavation and do not include the space required for shoring or sloping the walls of the excavation in unstable or soft materials or for deep excavations.

Test pits may be dug by hand or by machine. Power excavating equipment, such as backhoes may be used for rough excavation of test pits to a distance of about 0.6 m (2 ft) from the proposed sample. The final excavation of samples must be carefully made with hand tools, such as picks, shovels, trowels, and buckets. Deeper pits must be started with sufficient dimensions to allow for shoring or sloping of the sides to prevent caving. The depth of the pit and the type and condition of the soil generally dictate the type of support system, such as sheeting, sheet piling, bracing, shoring, or cribbing, which is needed. Shoring must be installed progressively as the pit is deepened. The space between the walls of the pit and the support system should be kept to a minimum.

Care must be exercised in excavating the area near the intended sample or test. The limits of the sample should be outlined with a pick and shovel. The material near the proposed sample should be excavated to a depth about 25 to 50 mm (1 to 2 in.) below the bottom of the intended sample. The excavated zone should be trimmed relatively level and be sufficiently large to allow adequate working space for obtaining the sample. A pedestal of soil, roughly the shape of the sample and about 25 mm (1 in.) larger in each dimension, should be left undisturbed for final trimming.

Excavated material should be placed at a horizontal distance from edge of the pit not less than the anticipated maximum depth of the pit. Excavated material should be placed in orderly fashion around the pit to facilitate logging of the material. Wooden stakes can be used to mark the depth of the excavated material. Samples for water content determination should be obtained in a timely manner to prevent drying of the material.

Test trenches can be used to perform the same function as test pits but offer one distinct advantage, i.e., trenches provide a continuous exposure of the continuity and character of the subsurface material along a given line or section. Test trenches can be excavated with ditching machines, backhoes, bulldozers, or pans, depending upon the required size and depth of the trench. The minimum bottom width of a trench is about 0.6 to 0.9 m (2 to 3 ft), although this dimension is sometimes greater because of the use of power equipment, such as bulldozers and pans. As the trench is deepened, the sides must be sloped, step cut, or shored to prevent caving, similar to the procedures that must be used for excavating deep test pits. Final excavation in the vicinity of the intended sample must be performed carefully by hand.

3.5.1 DISTURBED SAMPLES

Disturbed samples may be obtained from test pits or trenches. Disturbed, representative samples of soil are satisfactory for certain laboratory tests including classification, water content determination, and physical properties tests. For certain soils such as very soft clays or gravelly soils, undisturbed samples may be impossible to obtain. Provided that the unit weight and moisture content of the soil in place can be estimated or are known, it may be permissible to perform certain laboratory tests on specimens remolded from samples of disturbed material.

To sample a particular stratum, remove all weathered and mixed soil from the exposed face of the excavation. Place a large tarpaulin or sheet of plastic on the bottom of the test pit or accessible boring. With a knife or shovel, trench a vertical cut of uniform cross section along the full length of the horizon or stratum to be sampled. The width and depth of the cut should be at least six times the diameter of the largest soil particle sampled. Collect the soil on the tarpaulin. All material excavated from the trench should be placed in a large noncorrosive container or bag and preserved as a representative sample for that stratum. An alternative sampling procedure consists of obtaining a composite sample of two or more soil strata; if samples from certain strata are omitted, an explanation must be reported under .Remarks. on the log form. Samples obtained for determination of water content may be placed in pint glass or plastic jars with airtight covers; the sample should fill the container. Take care to ensure that overburden or weathered material is not included as a portion of the sample.

3.5.2 UNDISTURBED SAMPLES

Undisturbed samples are taken to preserve as closely as possible the in-place density, stress, and fabric characteristics of the soil. Although excavating a column of soil may relieve in situ stresses to some degree, it has been demonstrated that hand sampling of certain soils, such as stiff and brittle soils, partially cemented soils, and soils containing coarse gravel and cobbles, is perhaps the best and sometimes the only method for obtaining any type of representative sample. Large block samples of these materials are suitable for certain laboratory tests, although smaller samples should be used whenever the size of the sample does not adversely affect the test results. During handling and shipping of undisturbed samples, it is important to minimize all sources of disturbance including vibration, excessive temperature changes, and changes of water content.

3.5.2.1 BLOCK OR CUBE SAMPLES

To obtain a cube or block sample, prepare the surface of the soil to be sampled. Excavate a pedestal of soil that is slightly larger than the dimensions of the box or container into which the sample is to be placed. A knife, shovel, trowel, or other suitable hand tools should be used to carefully trim the sample to about 25 mm (1 in.) smaller than the inside dimensions of the box. As the sample is trimmed to its final dimensions, cover the freshly exposed faces of the sample with cheesecloth and paint with melted wax to prevent drying and to support the column of soil. After the block of soil has been trimmed but before it has been cut from the underlying material, place additional layers of cheesecloth and wax to form a minimum of three layers, as presented in ASTM D 4220-83 (ASTM 1993). A 1:1 mixture of paraffin and microcrystalline wax is better than paraffin for sealing the sample. A sturdy box should be centered over the sample and seated. Loose soil may be lightly tamped around the outside of the bottom of the box to align the box with respect to the soil sample and to allow packing material such as styrofoam, sawdust, or similar material to be placed in the voids between the box and the soil sample. Hot wax should not be poured over the sample. After the packing material has been placed around and on top of the sample and the top cover for the box has been attached, cut or shear the base of the sample from the parent soil and turn the sample over. After the sample has been trimmed to about 12 mm (1/2 in.) inside the bottom of the box, the bottom of the sample should be covered with three alternating layers of cheesecloth and wax. The space between the bottom of the sample and the bottom of the box should be filled with a suitable packing material before the bottom cover is attached. The top and bottom of the box should be attached to the sides of the box by placing screws in predrilled holes. The top and bottom should never be attached to the sides of the box with a hammer and nails because the vibrations caused by hammer blows may cause severe disturbance to the sample.

3.5.2.2 PUSH SAMPLERS

Several hand-operated open- or piston-samplers are available for obtaining undisturbed samples from the ground surface as well as from the walls and bottom of pits, trenches, or accessible borings. The hand-operated open sampler consists of a thin-wall sampling tube affixed to a push rod and handle. The piston sampler is similar to the open sampler except a piston is incorporated into the design of the device. The procedures for operating these samplers are similar to the procedures for open samplers or piston samplers in rotary drilling operations. Hand operated push samplers may be used to obtain samples in soft-to-medium clays, silts, and peat deposits at depths of 6 to 9 m (20 to 30 ft) or more.

3.6 BORING AND SAMPLING RECORDS

After the soil samples have been removed from the sampling apparatus, visually identified according to the procedures and methods which are presented in ASTM D 2488, and sealed in appropriate sample containers, the sample containers should be identified and labeled and the boring logs should be updated.

All tubes and samples should be labeled immediately to ensure correct orientation and to accurately identify the sample. The information on the sample identification tag should include project title and location, boring and sample number, depth and/or elevation interval, type of sample, recovery length, trimmed sample length, sample condition, visual soil classification (ASTM D 2488), date of sampling, and name of inspector. All markings should be made with waterproof, nonfading ink. Pertinent boring information and sample data, must be recorded in the boring log.

In addition to the aforementioned data which were placed on the sample identification tag, clear and accurate information which describes the soil profile and sample location should be documented in the boring logs. Record any information that may be forgotten or misplaced if not recorded immediately, such as observations which may aid in estimating the condition of the samples, the physical properties of the in situ soil, special drilling problems, weather conditions, and members of the field party.

3.7 SHIPMENT OF SAMPLES

The most satisfactory method of transporting soil samples is in a vehicle that can be loaded at the exploration site and driven directly to the testing laboratory. This method helps to minimize sample handling and allows the responsibility of the samples to be delegated to one person. In general, jar samples from the bottom of the tube samples can usually be packaged in containers furnished by the manufacturer, although special cartons may be required if considerable handling is anticipated. Undisturbed sample tubes should be packed in an upright orientation in prefabricated shipping containers or in moist sawdust or similar packing materials to reduce the disturbance due to handling and shipping. For certain cases, special packing and shipping considerations may be required. Regardless of the mode of transportation, the soil samples should be protected from temperature extremes and exposure to moisture. If transportation requires considerable handling, the samples should be placed in wooden boxes. Additional guidance is presented in ASTM D 4220-83, .Preserving and Transporting Soil Samples. (ASTM 1993).

3.8 BACKFILLING BOREHOLES AND EXCAVATIONS

All open boreholes, test pits or trenches, and accessible borings, including shafts or tunnels, must be covered or provided with suitable barricades, such as fences, covers, or warning lights, to protect pedestrians, livestock or wild animals, or vehicular traffic from accidents (EM 385-1-1). After the excavations have served their intended purposes, the sites should be restored to their original state as nearly as possible. Boreholes or excavations which are backfilled as a safety precaution may be filled with random soil. The quality of the backfill material should be sufficient to prevent hazards to persons or animals and should prevent water movement or collapse, particularly when drilling for deep excavations or tunnels. The soil should be tamped to minimize additional settlement which could result in an open hole at some later time.

4.0 ROCK CORING

Core drilling, if carefully executed and properly reported, can produce invaluable subsurface information. Details of coring log requirements are provided in EM1110-1-1804. Each feature logged shall be described in such a way that other persons looking at the core log will recognize what the feature is, the depth at which it occurred in the boring, and its thickness or size. They should also be able to obtain some idea of the appearance of the core and an indication of its physical. Each lithologic unit in the core

shall be logged. The classification and description of each unit shall be as complete as possible. A simple and widely used measure of the quality of the rock mass is provided by the Rock Quality Designation (RQD), which incorporates only sound, intact pieces 10 cm (4 in.) or longer in determining core recovery. In practice, the RQD is measured for each core run. See EM1110-1-1804 for details.
Backfilling Boreholes and Excavations

5.0 GEOTECHNICAL REPORT CONTENT AND ORGANIZATION.

Geotechnical reports submitted for review with design analysis shall be organized to present the key findings and conclusions at the beginning of the report in a clear, concise written narrative that allows the reader easy access to the main design parameters that have been determined. The organization of the report is important. Report conclusions shall not be "buried" in an obscure part of the report. Details of the field investigation and laboratory testing shall be provided as appendices. General dissertations on geotechnical engineering are neither helpful nor welcome. For the most part, USACE-AED projects do not require seismicity information and it is not required as part of the usual geotechnical report. Under special circumstance where it is required it will be identified as a requirement in the contract section 01015.

5.1 MINIMUM CONTENT OF THE GEOTECHNICAL REPORT

5.1.1 REPORT SUMMARY AND CONCLUSIONS -

a. Summary paragraph of the scope of work as stated in the contract documents.

b. Summary of site location and description of the project site, indicating principal topographic features in the vicinity including site photos showing subsurface investigation methods at the project site (not generic illustrations).

c. A plan map that shows the surface contours, the location of the proposed structure, and the location of all borings or test pits.

d. Tables that summarize the geotechnical findings for easy reference by designers and reviewers including:

   i. Number of soil sample and location relative to foundations or road segments.

   ii. Soil classification for samples.

   iii. Summary of design parameters (see example in Appendix B).

e. Summary of settlement analysis with calculations and references provided. Calculations shall consist of the equation utilized, definition of all variables, values used for all variables, and the calculation (substitution of values for the variables). Every step of the analysis shall be included for verification. If assumptions are made, then those assumptions shall be clearly stated and supported with verifiable information.

f. Summary of allowable bearing capacity analysis with calculations and references provided. Calculations shall consist of the equation utilized, definition of all variables, values used for all variables, and the calculation (substitution of values for the variables). Every step of the analysis shall be included for verification. If assumptions are made, then those assumptions shall be clearly stated and supported with verifiable information contained within the report. Note soils laboratories shall report the allowable bearing capacity values. These values will be provided to the design engineer based on the supporting strength test data provided by the laboratory. In each case the design shall be approved and stamped by a registered professional engineer. USACE-AED will require minimum design standards that insure uniformity in project design throughout Afghanistan and sufficient conservatism in design to insure that uncertainties in sampling and testing are not a significant risk to the integrity of the project functionality or a life safety issue for the users. Contract technical requirements will state these minimum design parameters for the designer in the request for proposal RFP for the project. The designer shall insure, with the assistance of geotechnical investigations and tests based on the approved standard methods cited herein that the minimum is sufficiently conservative as stated in the technical requirements, a basis for a selection of a more conservative design parameter.

g. The Factor of Safety (FS) utilized for analysis shall be clearly stated within the report. It shall be obvious that the Factor of Safety was implemented in the calculations reported. Due to the nature of projects USACE-AED is contracting and the seismology of Afghanistan the minimum Factor of Safety utilized for analysis shall be as follows in Table 3.

Table 3 Minimum foundation design factors of safety

Structure	FS
Retaining	
walls	3
excavations	> 2
Bridges	3.5
Buildings	
Warehouses	> 3
Offices	3
Industrial, Public	3.5
Footings	3
Mats	> 3

5.1.2 REPORT APPENDICES

5.1.2.1 SURFACE INVESTIGATIONS

a. Difference in topographic elevation over the site; in other words the highest and lowest elevation, examples of steep and flat existing slope across the project site, major drainage channel within the site, and presence of exposed bedrock.

b. Evidence of in situ soil performance as indicated by localized subsidence, existing building deformations and settlements, landslide scars

c. Evidence of seasonal high surface and ground water for example related to proximity to river channels and springs

d. Potential sources of construction material including quarries, borrow areas, and river gravel and sand deposits

e. Results of plate-load bearing tests (ASTM D 1194) if applicable.

5.1.2.2 SUBSURFACE INVESTIGATIONS

a. Detailed geologic profiles describing by depth of strata the soil types, visual observations and photographic record

b. Ground water elevations if encountered during subsurface explorations

c. Depths at which cores or samples were taken in relation to important structures as shown on a site plan drawing

d. Boring boring logs shall be provided for all borings (see Appendix A). The logs shall have a depth scale starting at the ground surface and scaled vertically in meters below ground surface. Show visual descriptions (e.g. light brown Silt with trace of gravel) and soil classifications (e.g. SM) and make clear the elevation of transitions between soil types. Classifications shall be based on Unified Soil Classification System. Record blow counts (for each 152 mm) and N-values. Show ground water symbol level at time of drilling and 24 hours later. Also, record sampling information as shown in Appendix A.

5.1.2.3 LABORATORY TEST RESULTS

a. water content (ASTM D 2216)

b. specific gravity of soil solids (ASTM D 854)

c. Atterberg limits (ASTM D 4318)

d. Grain size analysis (ASTM D 422)

e. Density and unit weight of soil in place using sand cone method (ASTM D 1556)

f. Soil classification according to Unified Soil Classification System (ASTM D 2487)

g. Laboratory compaction characteristics using modified effort (ASTM D 1557)

h. Unconfined compressive strength of cohesive soil (ASTM D 2166)

i. CBR (ASTM D 1883)

Note variations in the above list of required tests will occur between projects that contain vertical and horizontal construction features.

5.1.2.4 ADDITIONAL ADVANCED LABORATORY TESTS

a. One dimensional consolidation properties of soils (ASTM D 2435)

b. Direct shear test of soils under consolidated drained conditions (ASTM D 3080)

6.0 USACE-AED CERTIFICATION REQUIREMENT

All geotechnical investigations as well as laboratory and field methods of testing shall be performed by laboratories that have been certified by USACE-AED Quality Assurance Branch. A roster of certified laboratories and test certified to be performed is maintained by the branch. Failure to maintain certification shall be grounds for rejection of the geotechnical report.

A copy of the current certification shall be included with each geotechnical report.

7.0 REFERENCES

1. UFC 3-220-10N Soil Mechanics, 2005
2. UFC 3-220-01N Geotechnical Engineering Procedures for Foundation Design of Buildings and Structures, 2004
3. UFC 3-220-03FA Soils and Geology Procedures for Foundation Design of Buildings and Structures, 2004
4. UFC 3-220-07 Foundations in Expansive Soils, 2004
5. EM 1110-1-1804 Geotechnical Investigations, 2001
6. EM 1110-1-1904 Settlement Analysis, 1990
7. EM 1110-1-1905 Bearing Capacity of Soils, 1992
8. EM 1110-2-1902 Slope Stability, 2003
9. Department of the Navy Soil Dynamics, Deep Stabilization, and Special Geotechnical Construction, 1983

Appendix A
EXAMPLE FIELD BORING LOG

<table>
<tr><td colspan="4">Field Boring Log</td><td colspan="3">Project:</td><td>Boring #:</td><td>Page of</td></tr>
<tr><td colspan="4">County:</td><td colspan="3">Site:</td><td colspan="2">Location:</td></tr>
<tr><td colspan="4">Drilled by:</td><td colspan="2">Start Date:</td><td>Completion Date:</td><td colspan="2">Total Depth:</td></tr>
<tr><td colspan="7">Drilling Method & Equipment:</td><td colspan="2">Logged by:</td></tr>
<tr><td rowspan="2">DEPTH (m)</td><td rowspan="2">SPT Blow Count</td><td colspan="3">SAMPLE</td><td rowspan="2">Litho logy Strip</td><td colspan="3" rowspan="2">Visual Description/Other Field Test</td></tr>
<tr><td>% Recovery</td><td>Type</td><td>#</td></tr>
<tr><td></td><td></td><td></td><td></td><td></td><td></td><td colspan="3"></td></tr>
</table>

Appendix B
EXAMPLE OF DATA SUMMARY

Summary Table of Geotechnical Tests for Vertical Construction

Project Name & Contract		Date of Investigation		Site Surface Description (attach photos)	
Project Location		Investigation Supervsior			
Sample Location		Date of this Report			

			Particle Size (ASTM D422)			Atterberg Limits (ASTM D 4318)			Soil Class ASTM D 2487			Compression (ASTM D 2166)		ASTM D 2216	ASTM D 1556	ASTM D 1586	ASTM D 2435		ASTM D 3080			
Sample No	Sample Location	Sample Depth	Gravel (4.75 - 87.2mm)	Sand (0.076 - 4.75 mm)	Silt & Clay (<0.075 mm)	LL	PL	PI	USCS Group Symbol	USCS Name	Type of soil by AASHTO	C	compressive strength,qu Kpa	Natural Water content, %	Dry density, kg/m^3	SPT Blow Counts	Compression index, C_c	Recompression index, Cr	Shear stress-1, Kpa	Shear stress-2, Kpa	Shear stress-3, Kpa	Peak friction angle, phi, o

Summary Table of Geotechnical Tests for Horizontal Construction

Project Name & Contract		Date of Investigation		Site Surface Description (attach photos)
Project Location		Investigation Supervsior		
Sample Location		Date of this Report		

Sample No	Sample Location	Sample Depth	Particle Size (ASTM D422)			Atterberg Limits (ASTM D 4318)			Soil Class ASTM D 2487			Compression (ASTM D 2166)		ASTM D 2216	ASTM D 1556	ASTM D 1883	ASTM D 2435		ASTM D 3080			
			Gravel (4.75 - 87.2mm)	Sand (0.076 - 4.75 mm)	Silt & Clay (<0.075 mm)	LL	PL	PI	USCS Group Symbol	USCS Name	Type of soil by AASHTO	C	compressive strength, qu Kpa	Natural Water content, %	Dry density, kg/m^3	CBR	Compressi on index, C_c	Recompres ssion index, Cr	Shear stress-1, Kpa	Shear stress-2, Kpa	Shear stress-3, Kpa	Peak friction angle, phi, o

Appendix C
EXAMPLE OF DATA PRESENTATION

Step 1: Plot graphs.
See Fig. E5.5.

Step 2: Determine whether the sand is dense or loose.
The sand appears to be dense—it shows a peak horizontal force and dilated.

Step 3: Extract the required values.

Cross-sectional area of sample: $A = 10 \times 10 = 100 \text{ cm}^2 = 10^{-2} \text{ m}^2$

(c1) $\tau_p = \dfrac{(P_x)_p}{A} = \dfrac{1005 \text{ N}}{10^{-2}} \times 10^{-3} = 100.5 \text{ kPa}$

(c2) $\tau_{cs} = \dfrac{(P_x)_{cs}}{A} = \dfrac{758 \text{ N}}{10^{-2}} \times 10^{-3} = 75.8 \text{ kPa}$

(c3) $\alpha_p = \tan^{-1}\left(\dfrac{-\Delta z}{\Delta x}\right) = \tan^{-1}\left(\dfrac{0.1}{0.8}\right) = 7.1°$

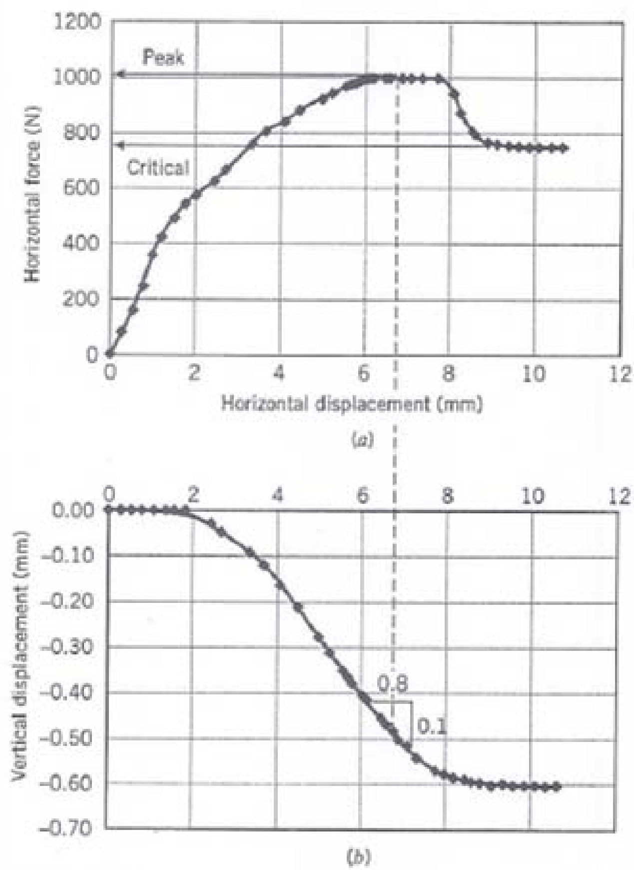


FIGURE E5.5

Soil	Test number	Vertical force (N)	Horizontal force (N)
A	Test 1	250	150
	Test 2	500	269
	Test 3	750	433
B	Test 1	100	98
	Test 2	200	175
	Test 3	300	210
	Test 4	400	248

Determine the following:

(a) ϕ'_{cs}

(b) ϕ'_p at vertical forces of 200 N and 400 N for sample B

(c) The dilation angle at vertical forces of 200 N and 400 N for sample B

Strategy To obtain the desired values, it is best to plot a graph of vertical force versus horizontal force.

Solution 5.3

Step 1: Plot a graph of the vertical forces versus failure horizontal forces for each sample. See Fig. E5.3.

Step 2: Extract ϕ'_{cs}.
All the plotted points for sample A fall on a straight line through the origin. Sample A is a nondilatant soil, possibly a loose sand or a normally consolidated clay. The effective friction angle is $\phi'_{cs} = 30°$.

Step 3: Determine ϕ'_p.
The horizontal forces at 200 N and 400 N for sample B do not lie on the straight line corresponding to ϕ'_{cs}. Therefore, each of these forces has a ϕ'_p associated with it.

$$(\phi'_p)_{200\text{ N}} = \tan^{-1}\left(\frac{175}{200}\right) = 41.2°$$

$$(\phi'_p)_{400\text{ N}} = \tan^{-1}\left(\frac{248}{400}\right) = 31.8°$$

Step 4: Determine α_p.

$$\alpha_p = \phi'_p - \phi'_{cs}$$

$$(\alpha)_{200\text{ N}} = 41.2 - 30 = 11.2°$$

$$(\alpha)_{400\text{ N}} = 31.8 - 30 = 1.8°$$

Note that as the normal force increases α_p decreases.

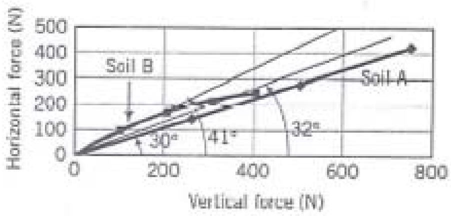


FIGURE E5.3

APPENDIX D
GEOTECHNICAL REPORT CHECKLIST

A. General

1. Has the appropriate geotechnical engineer reviewed the report to ensure that the design and construction recommendations have been incorporated as intended and that the subsurface information has been presented correctly? **This is absolutely necessary**.
   (*Yes / No / Unknown or N/A*)

2. Are the finished profile exploration logs and locations included in the plans?
   (*Yes / No / Unknown or N/A*)

3. Have the following common pitfalls been avoided:

   a. Has an adequate site investigation been conducted (reasonably meeting or exceeding the minimum criteria given in the AED Design Guide)?
      (*Yes / No / Unknown or N/A*)

   b. Has the use of "subjective" subsurface terminology (such as relatively soft rock or gravel with occasional boulders) been avoided?
      (*Yes / No / Unknown or N/A*)

   c. Have multiple soil classifications for a single sample in a soil horizon been reduced to only those permitted using ASTM D 2487
      (*Yes / No / Unknown or N/A*)

   d. If alignment has been shifted, have additional subsurface explorations been conducted along the new alignment?
      (*Yes / No / Unknown or N/A*)

   e. Do you think the wording of the geotechnical special provisions are clear, specific and unambiguous?
      (*Yes / No / Unknown or N/A*)

B. Centerline Cuts and Embankments

1. Where excavation is required, are excavation limits and description of unsuitable organic soils shown on the plans?
   (Yes / No / Unknown or N/A)

2. Are plan details and special provisions provided for special drainage details, such as lined surface ditches, drainage blanket under sidehill fill, interceptor trench drains, etc.?
   (Yes / No / Unknown or N/A)

3. Are special provisions included for fill materials requiring special treatment, such as nondurable shales, lightweight fill, etc.?
   (Yes / No / Unknown or N/A)

4. Are special provisions provided for any special rock slope excavation and Stabilization measures called for in plans, such as controlled blasting, wire mesh slope protection, rock bolts, shotcrete, etc.?
   (Yes / No / Unknown or N/A)

C. Embankments Over Soft Ground

1. Where subexcavation is required, are excavation limits and description of unsuitable soils clearly shown on the plans?
   (*Yes / No / Unknown or N/A*)

2. If instrumentation will be used to control the rate of fill placement, do special provisions clearly spell out how this will be done and how the readings will be used to control the contractor's

operation?

(*Yes* / *No* / *Unknown or N/A*)

D. Retaining Structures

1. Are select materials specified for wall backfill with gradation and compaction requirements covered in the specification?
   (*Yes* / *No* / *Unknown or N/A*)
2. Are excavation requirements specified, e.g., safe slopes for excavations, need for sheeting, etc.?
   (*Yes* / *No* / *Unknown or N/A*)

E. Structure Foundations - Spread Footings

1. Where spread footings are to be placed on natural soil, is the specific bearing strata in which the footing is to be founded clearly described, e.g., placed on Br. Sandy GRAVEL deposit, etc.?
   (*Yes* / *No* / *Unknown or N/A*)
2. Where spread footings are to be placed in the bridge end fill, are gradation and compaction requirements, for the select fill and backfill drainage material, covered in the special provisions, standard specifications, or standard structure sheets?
   (*Yes* / *No* / *Unknown or N/A*)

F. Ground Improvement Techniques

1. For fill, are minimum/maximum densities, gradation, lift thickness, and method of compaction specified?
   (*Yes* / *No* / *Unknown or N/A*)

US Army Corps of Engineers
Afghanistan Engineer District

AED Design Requirements:
Superelevation Road Design

Various Locations, Afghanistan

MARCH 2009

TABLE OF CONTENTS

AED DESIGN REQUIREMENTS
FOR
SUPERELEVATION ROAD DESIGN
VARIOUS LOCATIONS,
AFGHANISTAN

1. General

The purpose of this document is to provide requirements to Contractors for any project requiring the design and construction of superelevation road design.

2. Superelevation

Superelevation of a road is required to offset the centripetal acceleration that acts toward the center of curvature on a horizontal curve. For paved roads the rate of superelevation (e) is a function of the radius of curvature (R) of the road and the vehicle speed (V). The Islamic Republic of Afghanistan, Ministry of Public Works, Interim Road and Highway Standards, paragraph IX (c) limits the maximum superelevation rate (e_{max}) for all roads is 10%. Table 1 shown below will be used to determine the superelevation rates for horizontal curves of a known radius, a known vehicles speed and a maximum superelevation rate of 10%. If a horizontal curve has a sufficiently large radius and low vehicle speed so that no e is identified on Table 1, no superelevation is required for the curve. Additionally the minimum radius of a curve can be obtained by the vehicle speed and the maximum e.

Table 1. Minimum Radii for Design Superelevation Rates, Design Speed and e_{max}=10%

e (%)	V_d = 20 km/h R (m)	V_d = 30 km/h R (m)	V_d = 40 km/h R (m)	V_d = 50 km/h R (m)	V_d = 60 km/h R (m)	V_d = 70 km/h R (m)	V_d = 80 km/h R (m)	V_d = 90 km/h R (m)	V_d = 100 km/h R (m)	V_d = 110 km/h R (m)	V_d = 120 km/h R (m)	V_d = 130 km/h R (m)
1.5	197	454	790	1110	1520	2000	2480	3010	3690	4250	4960	5410
2.0	145	333	580	815	1120	1480	1840	2230	2740	3160	3700	4050
2.2	130	300	522	735	1020	1340	1660	2020	2480	2860	3360	3680
2.4	118	272	474	669	920	1220	1520	1840	2260	2620	3070	3370
2.6	108	249	434	612	844	1120	1390	1700	2080	2410	2830	3110
2.8	99	229	399	564	778	1030	1290	1570	1920	2230	2620	2880
3.0	91	211	368	522	720	952	1190	1460	1790	2070	2440	2690
3.2	85	196	342	485	670	887	1110	1360	1670	1940	2280	2520
3.4	79	182	318	453	626	829	1040	1270	1560	1820	2140	2370
3.6	73	170	297	424	586	777	974	1200	1470	1710	2020	2230
3.8	68	159	278	398	551	731	917	1130	1390	1610	1910	2120
4.0	64	149	261	374	519	690	866	1060	1310	1530	1810	2010
4.2	60	140	245	353	490	652	820	1010	1240	1450	1720	1910
4.4	56	132	231	333	464	617	777	953	1180	1380	1640	1820
4.6	53	124	218	315	439	586	738	907	1120	1310	1560	1740
4.8	50	117	206	299	417	557	703	864	1070	1250	1490	1670
5.0	47	111	194	283	396	530	670	824	1020	1200	1430	1600
5.2	44	104	184	269	377	505	640	788	975	1150	1370	1540
5.4	41	98	174	256	359	482	611	754	934	1100	1320	1480
5.6	39	93	164	243	343	461	585	723	896	1060	1270	1420
5.8	36	88	155	232	327	441	561	693	860	1020	1220	1370
6.0	33	82	146	221	312	422	538	666	827	976	1180	1330
6.2	31	77	138	210	298	404	516	640	795	941	1140	1280
6.4	28	72	130	200	285	387	496	616	766	907	1100	1240
6.6	26	67	121	191	273	372	476	593	738	876	1060	1200
6.8	24	62	114	181	261	357	458	571	712	846	1030	1170
7.0	22	58	107	172	249	342	441	551	688	819	993	1130
7.2	21	55	101	164	238	329	425	532	664	792	963	1100
7.4	20	51	95	156	228	315	409	513	642	767	934	1070
7.6	18	48	90	148	218	303	394	496	621	743	907	1040
7.8	17	45	85	141	208	291	380	479	601	721	882	1010
8.0	16	43	80	135	199	279	366	463	582	699	857	981
8.2	15	40	76	128	190	268	353	448	564	679	834	956
8.4	14	38	72	122	182	257	339	432	546	660	812	932
8.6	14	36	68	116	174	246	326	417	528	641	790	910
8.8	13	34	64	110	166	236	313	402	509	621	770	888
9.0	12	32	61	105	158	225	300	386	491	602	751	867
9.2	11	30	57	99	150	215	287	371	472	582	731	847
9.4	11	28	54	94	142	204	274	354	453	560	709	828
9.6	10	26	50	88	133	192	259	337	432	537	685	809
9.8	9	24	46	81	124	179	242	316	407	509	656	786
10.0	7	19	38	68	105	154	210	277	358	454	597	739

3. Superelevation Transition

A fully superelevated road section is obtained using a superelevation transition (T) from the normal crown of the road. The superelevation transition is composed of a superelevation runoff (L_r) and tangent runout (L_t). The superelevation runoff is the length of the roadway required to change the outside (superelevated) lane cross slope from a flat cross slope (0%) to a fully superelevated section. The length of the superelevation runoff is obtained from Equation 1 as shown below:

Equation 1 $L_r=[((w*n_1)*e_d)*b_w]/\Delta$

Where:

L_r=minimum length of superelevation runoff (m)
w=width of one traffic lane (m)
n_1=number of lanes rotated
e_d=design superelevation rate (%)

b_w=adjustment factor for number of lanes rotated=$[1+0.5(n_1-1)]/n_1$
Δ=maximum relative gradient (%) from Table 2

Table 2. Maximum Relative Gradients

Metric		
Design speed (km/h)	Maximum relative gradient (%)	Equivalent maximum relative slope
20	0.80	1:125
30	0.75	1:133
40	0.70	1:143
50	0.65	1:154
60	0.60	1:167
70	0.55	1:182
80	0.50	1:200
90	0.47	1:213
100	0.44	1:227
110	0.41	1:244
120	0.38	1:263
130	0.35	1:286

The tangent runout is the length of the roadway required to change the outside (superelevated) lanes from a normal cross crown cross slope to a flat cross slope (0%). The length of the tangent runout is obtained from equation 2 as shown below:

Equation 2 $L_t=(e_{NC}/e_d)L_r$

Where:
L_t=minimum length of tangent runout (m)
e_{NC}=normal cross slope rate (%)
e_d=design superelevation rate (%)
L_r=minimum length of superelevation runoff (m)

4. Superelevation Transition Placement

The proper placement of the superelevation transition (superelevation runoff and tangent runout) in relationship to the beginning of the curve (PC) or end of curve (PT) may have an effect of the safety and driver comfort along the curve. The placement of the superelevation runoff shall be with 1/3 of the runoff length on the curve and 2/3 of the runoff length on the tangent. The tangent runout will be immediately prior to the superelevation runoff when entering a curve and immediately after the superelevation runoff when exiting a curve.

5. Traveled Way Widening

Traveled way widening for horizontal curves may be required to make the operating conditions on the curve similar to those on the tangents. The traveled way widening values for two-lane highway with the specified roadway widths, curve radii and a WB-15 truck are obtained from Table 3 shown below.

Table 3. Calculated and Design Values for Travel Way Widening

Metric																		
Radius of curve (m)	Roadway width = 7.2 m Design Speed (km/h)						Roadway width = 6.6 m Design Speed (km/h)						Roadway width = 6.0 m Design Speed (km/h)					
	50	60	70	80	90	100	50	60	70	80	90	100	50	60	70	80	90	100
3000	0.0	0.0	0.0	0.0	0.0	0.0	0.2	0.2	0.3	0.3	0.3	0.3	0.5	0.5	0.6	0.6	0.6	0.6
2500	0.0	0.0	0.0	0.0	0.0	0.0	0.2	0.3	0.3	0.3	0.3	0.3	0.5	0.6	0.6	0.6	0.6	0.6
2000	0.0	0.0	0.0	0.0	0.0	0.1	0.3	0.3	0.3	0.3	0.3	0.4	0.6	0.6	0.6	0.6	0.6	0.7
1500	0.0	0.0	0.1	0.1	0.1	0.1	0.3	0.3	0.4	0.4	0.4	0.4	0.6	0.6	0.7	0.7	0.7	0.7
1000	0.1	0.1	0.1	0.2	0.2	0.2	0.4	0.4	0.4	0.5	0.5	0.5	0.7	0.7	0.7	0.8	0.8	0.8
900	0.1	0.1	0.2	0.2	0.2	0.3	0.4	0.4	0.5	0.5	0.5	0.6	0.7	0.7	0.8	0.8	0.8	0.9
800	0.1	0.2	0.2	0.2	0.3	0.3	0.4	0.5	0.5	0.5	0.6	0.6	0.7	0.8	0.8	0.8	0.9	0.9
700	0.2	0.2	0.2	0.3	0.3	0.4	0.5	0.5	0.5	0.6	0.6	0.7	0.8	0.8	0.8	0.9	0.9	1.0
600	0.2	0.3	0.3	0.3	0.4	0.4	0.5	0.6	0.6	0.6	0.7	0.7	0.8	0.9	0.9	0.9	1.0	1.0
500	0.3	0.3	0.4	0.4	0.5	0.5	0.6	0.6	0.7	0.7	0.8	0.8	0.9	0.9	1.0	1.0	1.1	1.1
400	0.4	0.4	0.5	0.5	0.6	0.6	0.7	0.7	0.8	0.8	0.9	0.9	1.0	1.0	1.1	1.1	1.2	1.2
300	0.5	0.6	0.6	0.7	0.8	0.8	0.8	0.9	0.9	1.0	1.1	1.1	1.1	1.2	1.2	1.3	1.4	1.4
250	0.6	0.7	0.8	0.8	0.9		0.9	1.0	1.1	1.1	1.2		1.2	1.3	1.4	1.4	1.5	
200	0.8	0.9	1.0	1.0			1.1	1.2	1.3	1.3			1.4	1.5	1.6	1.6		
150	1.1	1.2	1.3	1.3			1.4	1.5	1.6	1.6			1.7	1.8	1.9	1.9		
140	1.2	1.3					1.5	1.6					1.8	1.9				
130	1.3	1.4					1.6	1.7					1.9	2.0				
120	1.4	1.5					1.7	1.8					2.0	2.1				
110	1.5	1.6					1.8	1.9					2.1	2.2				
100	1.6	1.7					1.9	2.0					2.2	2.3				
90	1.8						2.1						2.4					
80	2.0						2.3						2.6					
70	2.3						2.6						2.9					

Notes: Values shown are for WB-15 design vehicle and represent widening in meters. For other design vehicles, use adjustments in Exhibit 3-48.
Values less than 0.6 m may be disregarded.
For 3-lane roadways, multiply above values by 1.5.
For 4-lane roadways, multiply above values by 2.

Travel lane widening for alternative vehicle types can be obtained by adding the values defined in Table 4 for the appropriate curve radius and vehicle type to the value obtained from Table 3.

Table 4. Adjustments for Traveled Way Widening Values

Metric							
Radius of curve (m)	Design vehicle						
	SU	WB–12	WB–19	WB–20	WB–20D	WB–30T	WB–33D
3000	–0.3	–0.3	0.0	0.0	0.0	0.0	0.1
2500	–0.3	–0.3	0.0	0.0	0.0	0.0	0.1
2000	–0.3	–0.3	0.0	0.0	0.0	0.0	0.1
1500	–0.4	–0.3	0.0	0.1	0.0	0.0	0.1
1000	–0.4	–0.4	0.1	0.1	0.0	0.0	0.2
900	–0.4	–0.4	0.1	0.1	0.0	0.0	0.2
800	–0.4	–0.4	0.1	0.1	0.0	0.0	0.2
700	–0.4	–0.4	0.1	0.1	0.0	0.0	0.3
600	–0.5	–0.4	0.1	0.1	0.0	0.1	0.3
500	–0.5	–0.4	0.1	0.2	0.0	0.1	0.4
400	–0.5	–0.4	0.2	0.2	0.0	0.1	0.5
300	–0.6	–0.5	0.2	0.3	–0.1	0.1	0.6
250	–0.7	–0.5	0.2	0.3	–0.1	0.1	0.8
200	–0.8	–0.6	0.3	0.4	–0.1	0.2	1.0
150	–0.9	–0.7	0.4	0.6	–0.1	0.2	1.3
140	–0.9	–0.7	0.4	0.6	–0.1	0.2	1.4
130	–1.0	–0.7	0.5	0.6	–0.2	0.2	1.5
120	–1.1	–0.8	0.5	0.7	–0.2	0.3	1.6
110	–1.1	–0.8	0.6	0.8	–0.2	0.3	1.7
100	–1.2	–0.9	0.6	0.8	–0.2	0.3	1.9
90	–1.3	–0.9	0.7	0.9	–0.2	0.3	2.1
80	–1.4	–1.0	0.8	1.1	–0.2	0.4	2.4
70	–1.6	–1.1	0.9	1.2	–0.3	0.5	2.8

Notes: Adjustments are applied by adding to or subtracting from the values in Exhibit 3-47.
Adjustments depend only on radius and design vehicle; they are independent of roadway width and design speed.
For 3-lane roadways, multiply above values by 1.5.
For 4-lane roadways, multiply above values by 2.0.

Very little benefit is gained from small amount of widening. Therefore the minimum widening will be 0.6 meters with widening amounts less than 0.6 meters being disregarded. Widening should be applied on the inside edge of the curve only and the widening should transition over the superelevation runoff length with 2/3 of the transition length along the tangent and 1/3 of the transition length along the curve. The edge of the traveled way through the widening transition should be a smooth curve with the transition ends avoiding an angular break at the pavement edge.

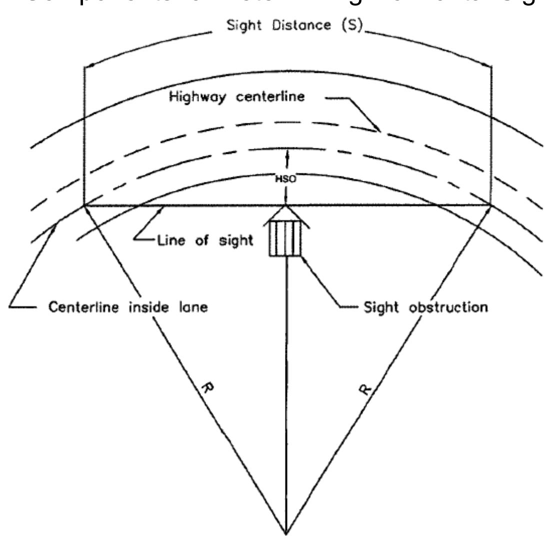
6. Stopping Sight Distance

Required stopping sight distance for various design speeds is presented in Table 5. The horizontal sightline offset in meters from the center of the inside travel lane is obtained from Equation 3 as shown in Exhibit 1.

Table 5. Stopping Sight Distance

Metric				
			Stopping sight distance	
Design speed (km/h)	Brake reaction distance (m)	Braking distance on level (m)	Calculated (m)	Design (m)
20	13.9	4.6	18.5	20
30	20.9	10.3	31.2	35
40	27.8	18.4	46.2	50
50	34.8	28.7	63.5	65
60	41.7	41.3	83.0	85
70	48.7	56.2	104.9	105
80	55.6	73.4	129.0	130
90	62.6	92.9	155.5	160
100	69.5	114.7	184.2	185
110	76.5	138.8	215.3	220
120	83.4	165.2	248.6	250
130	90.4	193.8	284.2	285

Exhibit 1. Components for Determining Horizontal Sight Distance



Equation 3 $HSO=R[1-\cos(28.65S/R)]$

Where:

HSO=horizontal sightline offset (m)
S=stopping sight distance (m)
R=radius of curve (m)

Where sufficient stopping sight distance is not available due to sight obstructions, alternative designs such as increasing the offset to the obstruction, increasing the radius or reducing the design speed should be considered for safety and economic reasons. The selected alternative should not include shoulder widths on the inside of the curve in excess of 3.6 meters to eliminate the chance of drivers using the shoulder as a passing or travel lane.

7. Passing Sight Distance

The minimum passing sight distance for a two-lane road is approximately four times as great as the minimum stopping sight distance at the same design speed. This greater distance may result is the sight line extending beyond the normal road right-of-way. For these reasons, passing sight distance should be limited to tangents and very flat curves.

8. Design Considerations

The following design considerations, in addition to the criteria listed above, should be reviewed for all horizontal curves to endure a safe design.

For a given design speed the minimum radius of curvature for that speed should be avoided wherever practical. The designer should attempt to use the largest radius possible saving the minimum radius curves for the most critical conditions.

Sudden sharp curves should not be introduces at the end of a long tangent section or large radius curves. Where a sharp curve is necessary, it should be preceded by a series of successively sharper curves.

For small deflection angles, curves should be sufficiently long to avoid the appearance of a kink. The minimum length for a horizontal curve on a main highway should be three times the design speed in km/h.

Compound curves should be avoided wherever possible. If the use of compound curves is unavoidable the radius of the flatter curve should not be more than 50 percent greater than the radius of the sharper curve.

Reverse curves should be avoided. The distance between reverse curves should be the sum of the superelevation runoff lengths and the tangent runout lengths.

Short tangent sections of roadway between two curves in the same direction should be avoided except where very unusual topographic or fight-of-way conditions make other alternatives impractical. A single large radius curve or two curves of smaller radius resulting is a longer tangent section should be investigates.

9. As-Builts

Upon completion of construction of the roadway, The Contractor shall submit editable CAD format As-Built drawings. The drawing shall show the final product as it was installed in the field, with the exact dimensions, locations, materials used and any other changes made to the original drawings. Refer to Contract Sections 01335 and 01780A of the specific project for additional details.

**US Army Corps
of Engineers
Afghanistan Engineer District**

AED Design Requirements: Hydrology Studies (Provisional)

Various Locations,
Afghanistan

January 2010, Version 1.6

TABLE OF CONTENTS

AED DESIGN REQUIREMENTS
FOR
HYDROLOGY STUDY
VARIOUS LOCATIONS, AFGHANISTAN

1. General

The purpose of this document is to illustrate the technical requirements contractors shall show in design analyses for projects requiring hydrology analysis of storm drainage components that are part of USACE-AED projects. The guidance is provisional – meaning it serves for the time being only until permanently replaced. The development of hydrologic statistics in Afghanistan is an ongoing process and as new data and analyses become available they will be incorporated into this design guide. A companion design guide discusses technical requirements for the design of culverts and road causeways - two hydraulic structures that require hydrologic analysis as the basis of the design.

2. Hydrology

Hydrology studies include a careful appraisal of factors affecting storm runoff to insure the development of a drainage system or road crossing culverts are capable of providing the required flow conveyance at the specified annual flood frequency of protection in the contract technical requirements. If the design flood frequency is not specified, the engineer shall base the selection of design storm magnitudes not only on the protection sought but also on the type of construction contemplated and the consequences of storms of greater magnitude than the design storm as specified in References 1 and 9.

Hydrologic studies for USACE-AED projects are generally concerned with the estimate of peak flow rates for use in the hydraulic design of channels, culverts, and erosion control and energy dissipation structures. In limited situations where ponding capacity is required, such as detention or infiltration facilities, runoff volume estimation is required. General USACE design information is provided in Reference 1.

Two hydrologic methods are preferred for use on USACE-AED projects: the Rational Method and the unit hydrograph method. The Rational Method shall be used when the catchment area draining to the structure or other point of concentration is less than one (1) square kilometer (247 acres). The Rational Method is generally limited to the calculation of the peak flow rate. The unit hydrograph method is required for drainage areas greater than one square kilometer. The theory and assumptions involved with these methods are well documented in design manuals and hydrologic engineering texts; two references which can be obtained from U.S. Government internet sources are included in References 2 and 3. The intent of this guide is to provide standardized data and assumptions in the use of these methods to simplify design and review of projects.

3. Design Conditions

Ground conditions affecting runoff must be selected to be consistent with existing and anticipated development and also with the characteristics and seasonal time of occurrence of the design rainfall.

Design conditions for the Rational Method consist of the runoff coefficient (C), the rainfall intensity-duration-frequency relationship, and the time of concentration. The runoff coefficient is a single parameter that considers soil type, land use cover (bare, vegetation, or pavement) and slope. There are several sources for C values that are acceptable provided they are accompanied by a complete reference in the design analysis. Generally the more information that is used in the C-value evaluation, the more accurate the flow estimation will be. A suggested chart is included in the next section that has compiled C values from several references.

In the majority of areas such as military, industrial, and cantonment areas, the design storm will normally be based on rainfall of 10-year frequency. This is equivalent to an annual probability of being equaled or exceeded equal to ten percent each year (Probability=1/10= 0.1, or 10 percent expresses as a percent). Potential damage or operational requirements may warrant a more severe criterion which shall usually be stated in the contract technical requirements. A lesser criterion may also be employed in regions where storms of an appreciable magnitude are infrequent and either damages or operational capabilities are such that large expenditures for drainage are not justified. The design of roadway culverts will normally be based on 10-year rainfall. Examples of conditions where greater than 10-year rainfall may be used are areas of steep slope in which overflows would cause severe erosion damage; high road fills that impound large quantities of water; and primary diversion structures, important bridges, and critical facilities where uninterrupted operation is imperative.

4. Runoff Computation Methods

The design procedures for drainage facilities involve computations to convert the rainfall intensities expected from the design storm into runoff rates which can be used to size the various elements of the storm drainage system. As previously stated, there are two basic approaches: direct estimates of the proportion of the average rainfall intensity which will appear as the peak rate of runoff (Rational Method) and unit hydrograph methods which account for losses such as infiltration and for the effects of flow over the surface to the point of design. The Rational Method approach can be used successfully by experienced designers for drainage areas up to 100 hectares in size and is discussed first. **For watershed sizes greater than one square kilometer a second approach shall be used to compute peak runoff that includes techniques to generate hydrographs, or calculation of a continuous flow rate over time, for surface runoff where studies of large drainage areas or complex conditions of storage require hydrographs are required.**

4.1. Rational Method

To compute peak runoff using the Rational Method the following equation is used.

$$Q=kCIA$$

Where

- Q=peak flow (m^3/sec.)
- k=0.278 (dimensionless)
- C=runoff coefficient (dimensionless)
- I=rainfall intensity (mm/hr)
- A=drainage area (km^2)

The k value in the above equation is a conversion factor to convert the peak flow into units of m^3/second.

a) Runoff Coefficient. The runoff coefficient (C) is a variable of the Rational Method that requires significant judgment and understanding on the part of the designer. The coefficient must account for all the factors affecting the relation of peak flow to average rainfall intensity other than area and response time. A range of C-values is typically offered to account for slope, condition of cover, soil moisture condition prior to the storm, and other factors that may influence runoff quantities. Good engineering judgment must be used when selecting a C-value for design and peak flow values because a typical coefficient represents the integrated effects of many drainage basin parameters. When available, design and peak flows should be checked against observed flood data. The following discussion considers only the effects of soil groups, land use, and average land slope.

As the slope of the drainage basin increases, the selected C-value should also increase. This is because as the slope of the drainage area increases, the velocity of overland and channel flow will increase, allowing less opportunity for water to infiltrate the ground surface. Thus, more of the rainfall will become runoff from the drainage area. The lowest range of C-values should be used for flat areas where the majority of grades and slopes are less than 2 percent. The average range of C-values should be used for intermediate areas where the majority of grades and slopes range from 2 to 5 percent. The highest range of C-values should be used for steep areas (grades greater than 5 percent), for impervious areas, and for development in clay soil areas.

It is often desirable to develop a composite runoff coefficient based on the percentage of different surface types in the drainage area. The composite procedure can be applied to an entire drainage area or to typical "sample" blocks as a guide to selection of reasonable values of the coefficient for an entire area. Impervious areas such as roadways, need to be accounted for in actual design. An example table of runoff coefficient values is provided Table 1.

Table 1. Runoff Coefficient Values (10-year storm frequency)

Rational Method Runoff C Coefficients

Type of Cover	Soil Type	Flat	Rolling 2% to 10%	Mountains over 10%
Buildings and roofs		0.90	0.90	0.90
Concrete pavedsurfaces		0.80	0.90	0.95
Asphalt pavedsurfaces		0.70	0.80	0.90
Earth embankments	bare & compacted	0.60	0.60	0.60
Gravel road shoulders		0.50	0.55	0.60
Sidewalks		0.80	0.82	0.85
Grassed areas	sandy	0.10	0.15	0.20
Grassed areas	clay	0.15	0.20	0.30
Farmed land	sand & gravel	0.25	0.30	0.35
Farmed land	clay & loam	0.50	0.55	0.60
steppe forest	sandy	0.10	0.15	0.20
semi desert land	bare & loose	0.10	0.20	0.30

Source: References 7 and 8

Other values that might be more appropriate for specific projects may be used provided they are completely referenced in the design analysis.

b) Rainfall Intensity. The rainfall intensity in millimeters/hour is generally determined from **I**ntensity-**D**uration-**F**requency (IDF) curves, if available. IDF curves are developed for regional areas as opposed to using one value for the entire sections of the country due to the wide fluctuations in rainfall over a large area. Sufficient information is available from sources in Afghanistan that merit the use of local data rather than attempts to derive IDF relationships from other countries. Data obtained from The Ministry of Energy and Water (MEW). The data was developed into IDF curves shown in Appendix B. The curves were developed as follows:

- Maximum annual 24-hour rainfall total depth measurements were compiled and fit to the Log Pearson Type III probability distribution using the Corps of Engineers computer program FFA (Reference 4); 10-, 20- , and 50- year 24-hour peak rainfall intensities were calculated
- The peak 24-hour intensities for each frequency were multiplied by ratios to obtain hour, one-hour, 30-minute, and 15 minute rainfall intensities for each time duration. The ratios were based on regional rainfall intensity durations curves obtained from MEW.
- The calculated rainfall intensity data were plotted on charts using log-log abscissa and ordinate scales

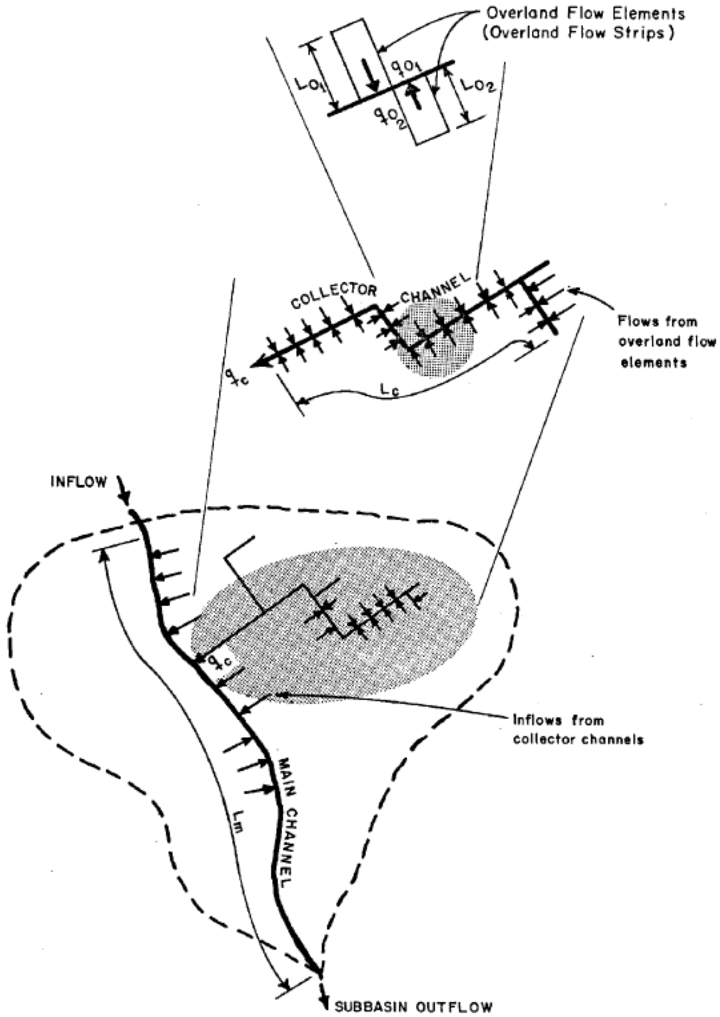
In regions where no I-D-F curve is available, the rainfall intensity may be calculated by the following formula:

$$I=(R/24)*(24/T_c)^K$$

Where

I=rainfall intensity (mm/hr)
R=maximum daily rainfall for design frequency (mm)
T_c=time of concentration (hr)
K = a regional coefficient whose approximate value is 0.722 for Afghanistan for 10-year storm

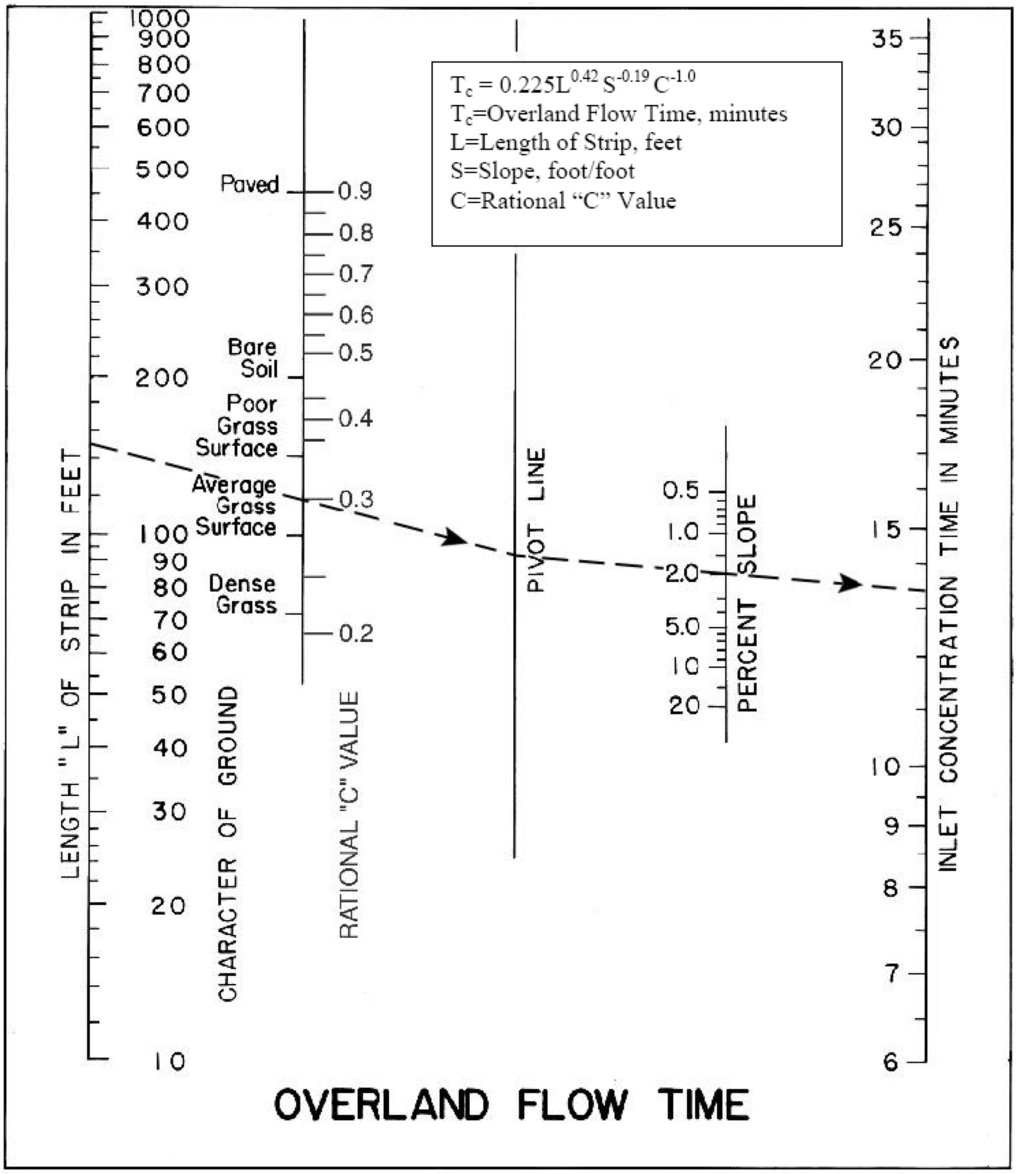
c) Time of Concentration. Time of concentration is the time for runoff to travel from the most hydraulically distant point in the watershed to the point of interest within the watershed. The time of concentration is the sum of the overland flow time, the shallow concentrated flow time and the channel flow time. For almost all drainage areas the maximum length of the overland flow will be approximately 100 meters. Overland flow will normally occur at the upper ends of the drainage or installation catchment area and will occur over relatively smooth surfaces such as parking areas and flat slopes. In areas where shallow ditches occur, the runoff will not be overland flow but will concentrate into shallow channels. Farther downstream the shallow channels such as gutters and surface swales further concentrate into open channel drainages. The following figure illustrates the concept of these flow components.



Source: Reference 3.

The overland flow time of concentration may be determined by the following nomograph in Figure 1. A nomograph is a chart usually containing three parallel scales graduated for different variables so that when a straight line connects values of any two, the related value may be read directly from the third at the point intersected by the line. Notice that the units on this nomograph are U.S. customary units and the result should be obtained first in this unit system and converted to SI units because the nomograph is based on the equation shown on it that uses empirical constants developed in this unit system.

Figure 1. Nomograph for Overland Flow Time of Concentration

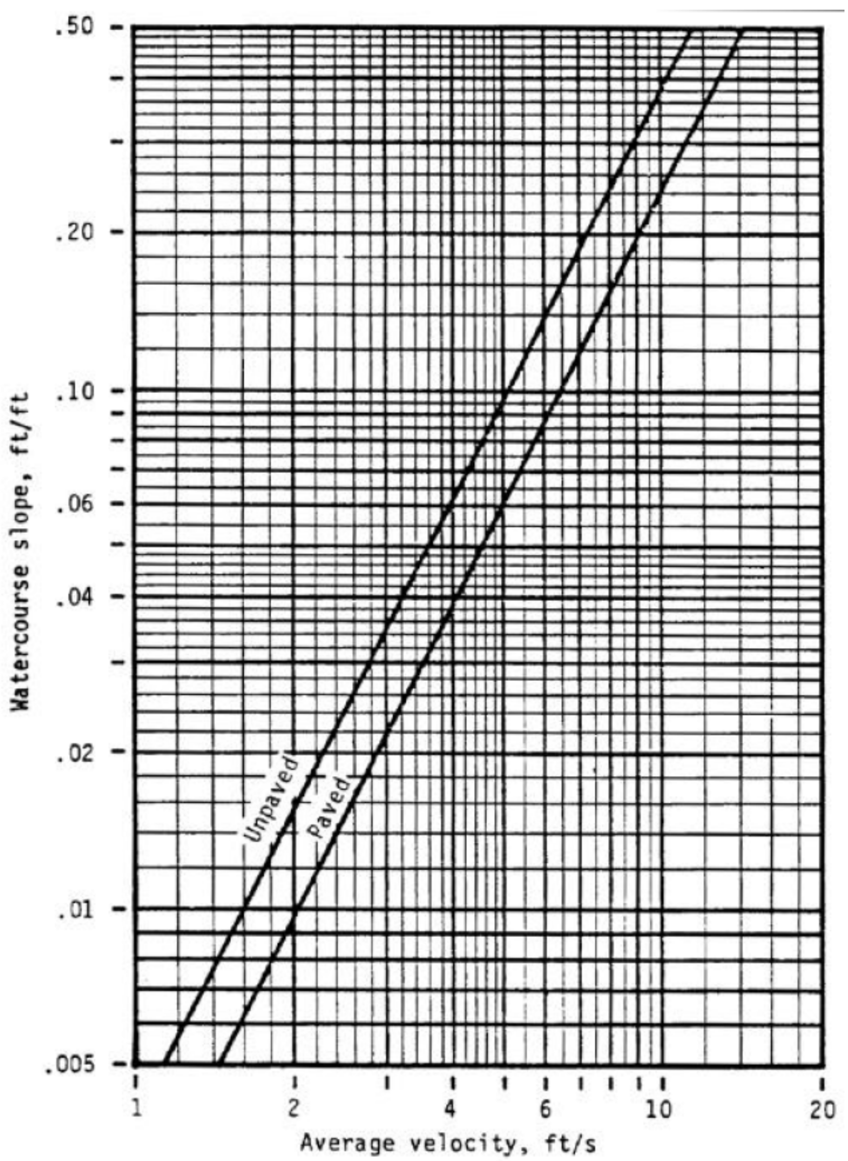


The overland flow time of concentration is determined by drawing a straight line through the flow length of the overland flow and the surface type or rational runoff coefficient and extending this line to the pivot line in the center of the nomograph. A line is then drawn from the intersecting point of the first line and the pivot line through the overland flow slope and extending this line to the concentration time line. Alternately the equation given in the top of Figure 1 can be used to calculate the time of concentration.

Shallow concentrated flow will occur after the maximum length of overland flow; generally within a distance of 100 to 150 meters such as in the depressions on the side of a slope or mountain. The designer should use topographic maps to determine where the shallow concentrated flow will begin and end such as a shallow watercourse. Topographic maps can be obtained from project survey drawings or for areas not within the project limits from Reference 6. The map scale for Afghanistan topographic maps is 1:250,000, and therefore will generally be used in conjunction with a CAD program to enlarge, scale and compute the area from an image file.

Shallow concentrated flow time of concentration is determined by dividing the flow length by the flow velocity. The flow velocity is determined by the following nomograph:

Figure 2. Nomograph for Shallow Concentrated Flow Velocity



Enter the nomograph using the slope of the shallow concentrated flow path and extend a line horizontally until the diagonal line of the appropriate surface type is intersected. From this point, extend the line straight down to determine the average velocity. The average velocity of the above nomograph is expressed in ft/sec. which is converted to m/min by multiplying by 18.29. The equations for these surfaces are:

Unpaved surface:	$y = 16.441 * x^{0.5063}$	where:	x = slope, ft/ft
Paved surface:	$y = 19.794 * x^{0.4896}$		y = average velocity, ft/s

Total time of concentration (T) is the sum of the overland and shallow travel time (To), plus the concentrated channel travel time (Tc), if any. An example is provided showing the steps necessary to calculate T using the previous concepts and equations.

Channel flow will occur in swales, ditches or underground culverts that have a sufficient volume to adequately convey the flow. Channel flow time of concentration is determined by dividing the flow length by the flow velocity. The channel flow velocity is determined by Manning's formula as shown below.

$$V = \frac{1.0}{n} * R^{2/3} * S^{1/2}$$

V = flow velocity (m/sec)

n = roughness coefficient
R = hydraulic radius, cross sectional flow area/wetted perimeter
S = channel slope (m/m)

$$V = \frac{1.49}{n} * R^{2/3} * S^{1/2}$$

V = flow velocity (ft/sec)

Manning's roughness coefficients (n) for channel surfaces are provided below in Table 2.

When designing channels for drainage collection and conveyance, stone or concrete lined trapezoidal channels with 120 degree interior angles are strongly recommended since they are compact, and can collect and efficiently convey significant quantities of water. Earthen swales with side slopes of 3 Horizontal to 1 Vertical may be used in flat areas, but these channels erode quickly when subject to high flows. Channels on AED projects shall be lined and trapezoidal when possible. Channel design and safety factors are described below.

Size channels in the following manner: Calculate the required flow area. Add a safety factor of 33% to both the horizontal and vertical channel dimensions to allow for sediment buildup and trash in the bottom. Add an additional 150mm freeboard height to the channel configuration which already includes the safety factor. More freeboard may be required at bends in the collection system.

Channels shall not make sharp 90 degree bends, but shall have smooth curves for turns. Conveyance systems with combinations of channels and culverts shall provide equal flow area for both styles of conveyance to prevent creating flow restrictions and potential flooding. Energy dissipation devices shall be provided where grades create high velocities. Where a 90 degree bend cannot be avoided due to site constraints, a covered, manhole type structure may be necessary to prevent spillage out of the channel.

NOTE: Even though flow calculations may indicate that smaller channels will adequately convey flows, the smallest channel allowed on AED projects shall have a bottom width of 400mm and a side height of 400mm. Final channel configuration shall include the required flow area plus the 33% safety factor plus the required freeboard.

Table 2. Manning's Channel Roughness Coefficient

Type of Channel and Description	Minimum	Normal	Maximum
LINED CHANNELS (Selected linings)			
a. Concrete			
1. Trowel finish	0.011	0.013	0.015
2. Float finish	0.013	0.015	0.016
3. Gunite, good section	0.016	0.019	0.023
b. Asphalt			
1. Smooth	0.013	0.013	-
2. Rough	0.016	0.016	-
EXCAVATED OR DREDGED			
a. Earth, straight and uniform			
1. Clean, recently completed	0.016	0.018	0.020
2. Clean, after weathering	0.018	0.022	0.025
3. Gravel, uniform section, clean	0.022	0.025	0.030
4. With short grass, few weeds	0.022	0.027	0.033
b. Earth, winding and sluggish			
1. No vegetation	0.023	0.025	0.030
2. Grass, some weeds	0.025	0.030	0.033
3. Dense weeds or aquatic plants in deep channels	0.030	0.035	0.040
4. Earth bottom and rubble sides	0.025	0.030	0.035
5. Stony bottom and weedy sides	0.025	0.035	0.045
6. Cobble bottom and clean sides	0.030	0.040	0.050
c. Dragline excavated or dredged			
1. No vegetation	0.025	0.028	0.033
2. Light brush on banks	0.035	0.050	0.060
d. Rock cuts			
1. Smooth and uniform	0.025	0.035	0.040
2. Jagged and irregular	0.035	0.040	0.050
e. Channels not maintained, weeds and brush uncut			
1. Dense weeds, high as flow depth	0.050	0.080	0.120
2. Clean bottom, brush on sides	0.040	0.050	0.080
3. Same, highest stage of flow	0.045	0.070	0.110
4. Dense brush, high stage	0.080	0.100	0.140
NATURAL STREAMS			
1. Minor streams (top width at flood stage <100 ft)			
a. Streams on Plain			
1. Clean, straight, full stage, no rifts or deep pools	0.025	0.030	0.033
2. Same as above, but more stones/weeds	0.030	0.035	0.040
3. Clean, winding, some pools/shoals	0.033	0.040	0.045
4. Same as above, but some weeds/stones	0.035	0.045	0.050
5. Same as above, lower stages, more ineffective slopes and sections	0.040	0.048	0.055
6. Same as 4, but more stones	0.045	0.050	0.060
7. Sluggish reaches, weedy, deep pools	0.050	0.070	0.080
8. Very weedy reaches, deep pools, or floodways with heavy stand of timber and underbrush	0.075	0.100	0.150
b. Mountain streams, no vegetation in channel, banks usually steep, trees and brush along banks submerged at high stages			
1. Bottom: gravels, cobbles and few boulders	0.030	0.040	0.050
2. Bottom: cobbles with large boulders	0.040	0.050	0.070
2. Floodplains			
a. Pasture, no brush			
1. Short grass	0.025	0.030	0.035
2. High grass	0.030	0.035	0.050
b. Cultivated area			
1. No crop	0.020	0.030	0.040
2. Mature row crops	0.025	0.035	0.045
3. Mature field crops	0.030	0.040	0.050
c. Brush			
1. Scattered brush, heavy weeds	0.035	0.050	0.070
2. Light brush and trees, in winter	0.035	0.050	0.060
3. Light brush and trees, in summer	0.040	0.060	0.080
4. Medium to dense brush, in winter	0.045	0.070	0.110
5. Medium to dense brush, in summer	0.700	0.100	0.160
d. Trees			
1. Dense Willows, summer, straight	0.110	0.150	0.200
2. Cleared land with tree stumps, no sprouts	0.030	0.040	0.050
3. Same as above, but with heavy growth of sprouts	0.050	0.060	0.080
4. Heavy stand of timber, a few down trees, little undergrowth, flood stage below branches	0.080	0.100	0.120
5. Same as above, but with flood stage reaching branches	0.100	0.120	0.160
3. Major Streams (top width at flood stage > 100 ft)			
The n-value is less than that for minor streams of similar description, because banks offer less effective resistance.			
a. Regular section with no boulders or brush	0.025	-	0.060
b. Irregular and rough section	0.035	-	0.100

Several alternative methods are available for estimation of the time of concentration that are based on empirical relationships for specific geographic areas and caution should be exerted in their application to a specific site. For example, another method for overland time of concentration is Kirpich's formula. It is based on analysis of data for watersheds in the state of Tennessee in the United States and has not been validated for Afghanistan. Note the units are customary US units are used in some of these methods which should be used to provide the results because the formula is based on empirical coefficients derived in that unit system.

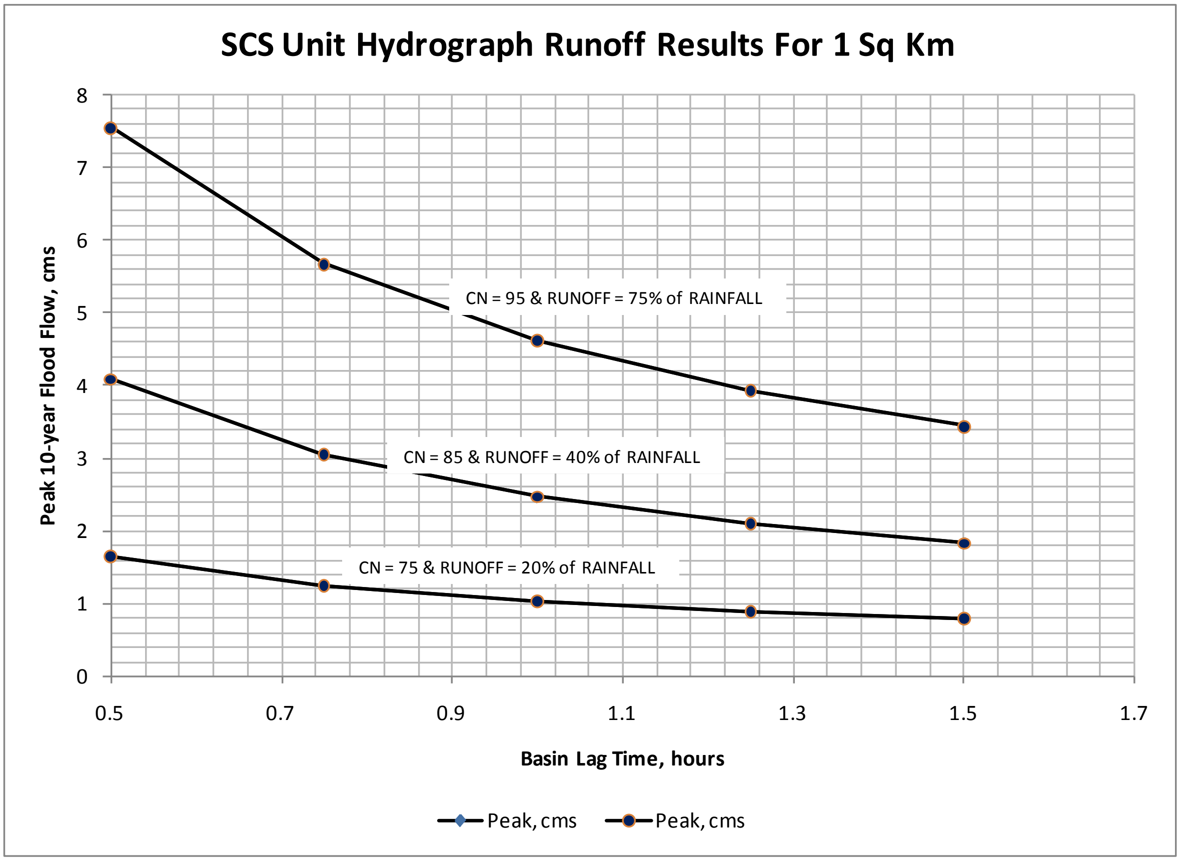
4.2. Soil Conservation Service (SCS) Unit Hydrograph Method

As the drainage basin size increases, the Rational Method becomes less accurate for a number of reasons. The principal reason is that the underlying concept behind the method, namely that the peak rainfall intensity duration is equal to the time of concentration no longer is a reasonable assumption. The water courses in larger basins have flood plains that will reduce larger flood flow rates because they will store flood water in their overbank areas thereby reducing the peak discharge rate. Large basins have more varied topography which correlates to varying times of concentration. Because of these and other limitations, numerous methods for developing unit hydrographs for selected watersheds have developed. **The Rational Method may significantly overestimate the peak discharge rate for larger basins and therefore the limit of 1 square kilometer for it has evolved from engineering experience as a useful upper limit.**

A unit hydrograph is defined as the direct runoff hydrograph (flow rate versus time relationship) from one unit of excess rainfall (usually 1 cm in SI units) generated uniformly over the drainage area at a constant rate for an effective duration of time. References 2 and 3 provide information on the theory and calculation details.

Because the solution to the total hydrograph computation involves successive convolutions of unit hydrographs for the period of the storm, the method is suitable to the use of computer programs for execution. Several programs are available from US Government agencies that are based on a particular unit hydrograph shape known as the SCS unit dimensionless hydrograph; reference 3 shows the location for one such program HEC-1 Flood hydrograph package. This program allows the user to employ the SCS unit hydrograph method with a hypothetical rainfall pattern constructed from data obtained from the intensity-duration-frequency curves (previously described in the rational method) to compute runoff hydrographs. In order to simplify the use of the SCS method for drainage areas in the range of one to two square kilometers which are common on road projects in Afghanistan, graphs have been prepared of peak discharge and runoff volumes have been computed for use shown in Figure 3. Results for other basins sizes can be obtained by the ratio of the drainage areas multiplied times the values from the figure. For large basins (greater than 10 sq km) a factor shown in Appendix C may be applicable.

Figure 3. SCS Unit Hydrograph Results for 1 Sq Km Drainage Area



Caution should be used applying data in Figure 3 for basins greater than two square kilometers in size because the floodplain attenuation effects in larger watersheds are neglected. Use of the curves requires calculating the basin lag defined as 0.6 times the time of concentration and the approximate runoff ratio based on the rational method or from data to develop curve numbers described below. For basins larger than 2 square kilometers or other flood frequency (than 10-year) storms, the design should be based on calculating the runoff using a computer model that supports the SCS method. If more than one basin analysis is required, a computer model should be used to perform hydrologic routing of the individual basins; Figure 3 curves do not include these affects and should not be used to combine more than one basin results.

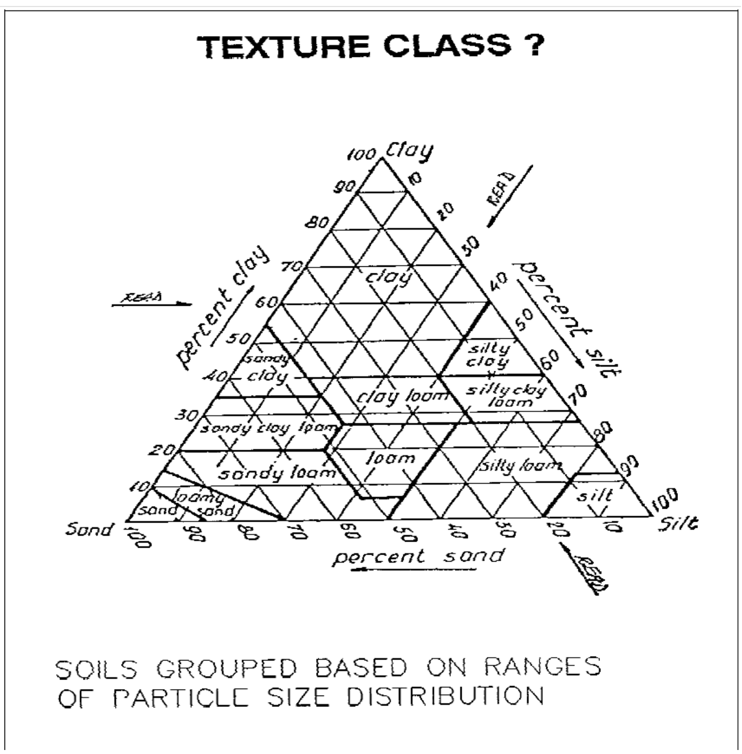
The SCS unit hydrograph technique is described in reference 2. There are two basic parameters required to use this method: basin lag and basin runoff curve number (CN) value. Basin lag is defined by the method as approximately 0.6 times the time of concentration (previously described in the rational method). The curve number is a dimensionless number that is an empirical function of soils slope, and land cover. It is used in the SCS method to determine the amount of rainfall retention over time that the watershed can hold. The excess becomes runoff. The curve numbers were derived empirically for non urban areas in the United States following a long program of collecting measurements at Soil Conservation Service hydrologic field stations of stream flow, precipitation, land cover and soil moisture. Tables of CN values for different hydrologic soil groups and land use are published in several sources.

Soil types are defined as follows:

Group A:	deep sands, deep loess, aggregated silts
Group B:	shallow loess, sandy loam
Group C:	clay loams, shallow sandy loams, and soils high in clay
Group D:	soils that swell when wet, plastic clays, and certain saline soils

Soil classification is defined in the SCS soil classification textural triangle shown in Figure 4. The availability of soil classification in regions of Afghanistan can be determined using internet sources (see References 5 and 6). Surface soil classification from geotechnical reports for project foundation design can also be used as a source of soil information provided the top soil horizon is used for determination of the runoff curve number.

Figure 4. SCS Soil Classification for Soil Groups



The following Table 3 contains an example from Reference 2 of curve number values.

Table 3. SCS Curve Number for Hydrologic Soil Groups

Table 6-5
Runoff CN's for Hydrologic Soil-Cover Complexes
(Antecedent runoff condition II, and $I_a = 0.2S$)

Cover			Hydrologic Soil Group			
Land use	Treatment or practice	Hydrologic Condition	A	B	C	D
Fallow	Straight row		77	86	91	94
Row crops	Straight row	Poor	72	81	88	91
		Good	67	78	85	89
	Contoured	Poor	70	79	84	88
		Good	65	75	82	86
	Contoured and terraced	Poor	66	74	80	82
		Good	62	71	78	81
Small grain	Straight row	Poor	65	76	84	88
		Good	63	75	83	87
	Contoured	Poor	63	74	82	85
		Good	61	73	81	84
	Contoured and terraced	Poor	61	72	79	82
		Good	59	70	78	81
Close-seeded legumes[1] or rotation meadow	Straight row	Poor	66	77	85	89
		Good	58	72	81	85
	Contoured	Poor	64	75	83	85
		Good	55	69	78	83
	Contoured and terraced	Poor	63	73	80	83
		Good	51	67	76	80
Pasture or range		Poor	68	79	86	89
		Fair	49	69	79	84
		Good	39	61	74	80
	Contoured	Poor	47	67	81	88
		Fair	25	59	75	83
		Good	6	35	70	79
Meadow		Good	30	58	71	78
Woods		Poor	45	66	77	83
		Fair	36	60	73	79
		Good	25	55	70	77
Farmsteads			59	74	82	86
Roads (dirts)[2]			72	82	87	89
(hard surface)[2]			74	84	90	92

[1] Closed-drilled or broadcast.
[2] Including right-of-way.

5. Design Submittal Documentation

Design analysis reports shall summarize the results of the calculations in a tabular form. The contents of the table shall include the following information:

- basin name/ or culvert number
- drainage area
- calculated time of concentration
- rainfall intensity for design storm
- runoff coefficient in rational method for the basin
- reduction for total area factor
- peak flow rate at the point of concentration (for structure design)
- channel sizing calculations showing equations used, assumptions, Manning coefficients, channel dimensions including sketches with bottom, side and freeboard dimensions, slopes, velocities, maximum capacities and anticipated conveyance due to site-specific storm requirements.

Submittal shall include drawings or sketches that identify the catchments areas used n the calculations.

6. References

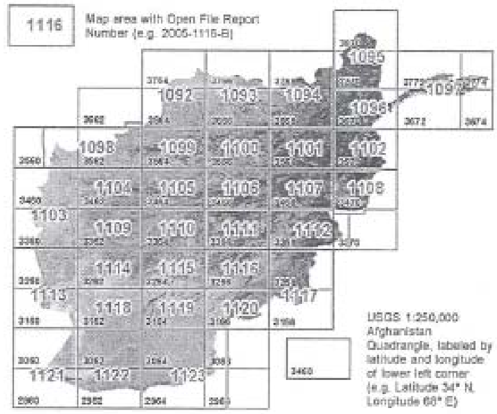
1. UFC 3 230 17FA Drainage for Areas Other than Airfields. Department of Defense, January 2004.
2. U.S. Army Corps of Engineers. Engineering and Design Flood hydrograph Analysis, EM 110-2-1417, August 1994. Found at http://140.194.76.129/publications/eng-manuals/em1110-2-1417/toc.htm
3. U.S. Army Corps of Engineers, Hydrologic Engineering Center, HEC-1 Flood Hydrograph Package. 1998. Found at http://www.hec.usace.army.mil/software/legacysoftware/hec1/hec1.htm
4. U.S. Army Corps of Engineers, Hydrologic Engineering Center, Flood Frequency Analysis (FFA) Program. 1994.
5. European Commission Land Management & Natrual Hazards Unit. Found at http://eusoils.jrc.ec.europa.eu/result.cfm?form.criteria=afghanistan%20and%20soil
6. US Geological Survey Open-File Report 2005-1103 Series of topographic maps found at http://pubs.usgs.gov/of/2005/
7. Civil Engineering Reference Manual, Michael Lindenburg, Profesional Publications, Inc. 2008
8. Washington State Department of Transportation, Hydraulics Manual, March 2005. Found at http://www.wsdot.wa.gov/Design/Hydraulics/
9. UFC 3 230 15FA Surface Drainage Facilities for Airfields and Heliports. Department of Defense, January 2004.

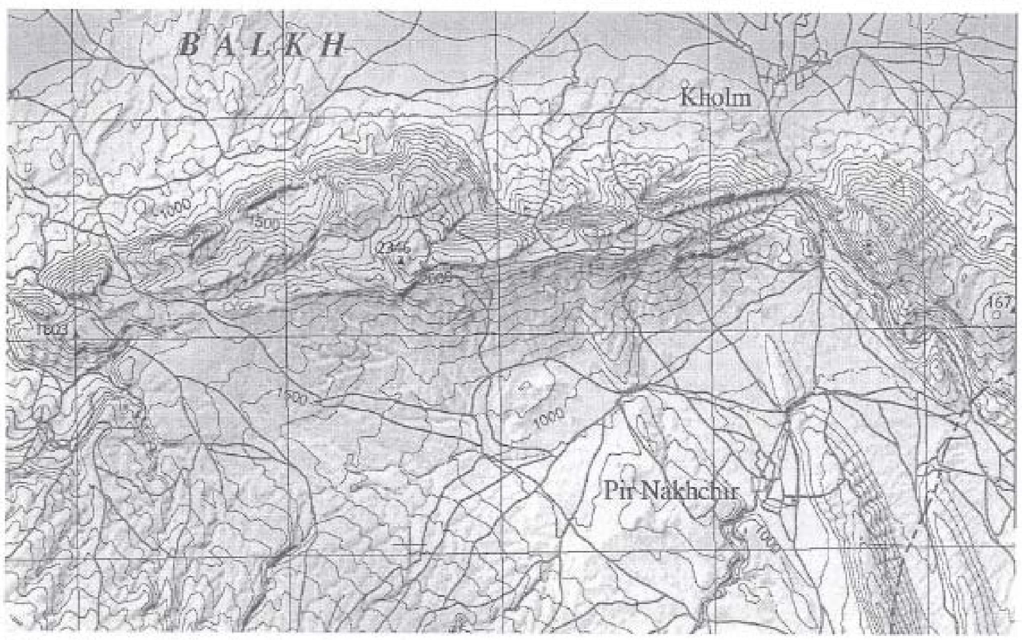
Topographic Map Index Information for

USGS Afghanistan Topographic Maps

The U.S. Geological Survey (USGS) has used TNTmips to prepare a series of 32 topographic maps of Afghanistan at a scale of 1:250,000, in cooperation with the Afghan Geological Survey and Afghanistan Geodesy and Cartography Head Office. These maps provide complete coverage of the country and are published electronically in the USGS Open File Report series as PDF map files that can be printed to scale. Each map covers a quadrangle 1° latitude by 2° longitude in size, or portions of several quadrangles along the border of the country. The complete map references are listed on the reverse side of this page along with a download link, and the index map to the right shows the map areas with report numbers. A companion series of geologic maps has also been published (see the Technical Guide entitled *USGS Afghanistan Geologic Maps*).

The map compilers at the USGS used the Surface Modeling process in TNTmips to generate topographic contours for the maps from Shuttle Radar Topography Mission (SRTM) 85-m digital elevation data. The contours are overlaid on a color shaded-relief image also derived from the SRTM data using the TNT Slope, Aspect, Shading process (now the Topographic Properties process). The stream lines are selected flow path lines generated from the SRTM data using the TNTmips Watershed process. This process automatically generates stream order attributes for the flow paths, and a selection query on the Horton stream order value was used to extract an appropriate density of stream lines for each map. (over)





Portion of *Topographic Map of Quadrangles 3666 and 3766, Balkh (219), Mazar-I-Sharif (220), Qarqin (213), and Hazara Toghai (214) Quadrangles, Afghanistan*, U.S. Geological Survey Open File Report 2005-1093-B, compiled by Robert G. Bohannon. Extract is shown at the published scale of 1:250,000.

MicroImages, Inc. • 11th Floor - Sharp Tower • 206 South 13th Street • Lincoln, Nebraska • 68508-2010 • USA
Voice (402)477-9554 • FAX (402)477-9559 • email info@microimages.com • web www.microimages.com • June 2007

All of the 32 Afghanistan Topographic maps listed below are available for free download as PDF files from: http://pubs.usgs.gov/of/2005. See the index map on the reverse side of this page for locations.

Topographic Map of Quadrangles 3764 and 3664, Jalajin (117), Kham-Ab (118), Char Shangho (123), and Sheberghan (124) Quadrangles, Afghanistan. U.S. Geological Survey Open File Report 2005-1092-B, compiled by Robert G. Bohannon.

Topographic Map of Quadrangles 3666 and 3766, Balkh (219), Mazar-I-Sharif (220), Qarqin (213), and Hazara Toghai (214) Quadrangles, Afghanistan. U.S. Geological Survey Open File Report 2005-1093-B, compiled by Robert G. Bohannon.

Topographic Map of Quadrangles 3768 and 3668, Imam-Saheb (215), Rustaq (216), Baghlan (221), and Taloqan (222) Quadrangles, Afghanistan. U.S. Geological Survey Open File Report 2005-1094-B, compiled by Robert G. Bohannon.

Topographic Map of Quadrangles 3870 and 3770, Maymayk (211), Jamarj-I-Bala (212), Faydz-Abad (217), and Parkhaw (218) Quadrangles, Afghanistan. U.S. Geological Survey Open File Report 2005-1095-B, compiled by Robert G. Bohannon.

Topographic Map of Quadrangle 3670, Jarm-Keshem (223) and Zebak (224) Quadrangles, Afghanistan. U.S. Geological Survey Open File Report 2005-1096-B, compiled by Robert G. Bohannon.

Topographic Map of Quadrangles 3772, 3774, 3672, and 3674, Gaz-Khan (313), Sarhad (314), Kol-i-Chaqmaqtin (315), Khandud (319), Deh-Ghulaman (320), and Ertfah (321) Quadrangles, Afghanistan. U.S. Geological Survey Open File Report 2005-1097-B, compiled by Robert G. Bohannon.

Topographic Map of Quadrangles 3560, 3562, and 3662, Sir Band (402), Khawja-Jir (403), Bala-Murghab (404), and Darah-I-Shor-I-Karamandi (122) Quadrangles, Afghanistan. U.S. Geological Survey Open File Report 2005-1098-B, compiled by Robert G. Bohannon.

Topographic Map of Quadrangle 3564, Chahriaq (Joand) (405) and Gurziwan (406) Quadrangles, Afghanistan. U.S. Geological Survey Open File Report 2005-1099-B, compiled by Robert G. Bohannon.

Topographic Map of Quadrangle 3566, Sang-Charak (501) and Sayghan-O-Kamard (502) Quadrangles, Afghanistan. U.S. Geological Survey Open File Report 2005-1100-B, compiled by Robert G. Bohannon.

Topographic Map of Quadrangle 3568, Polekhomri (503) and Charikar (504) Quadrangles, Afghanistan. U.S. Geological Survey Open File Report 2005-1101-B, compiled by Robert G. Bohannon.

Topographic Map of Quadrangle 3570, Tagab-E-Munjan (505) and Asmar-Kamdesh (506) Quadrangles, Afghanistan. U.S. Geological Survey Open File Report 2005-1102-B, compiled by Robert G. Bohannon.

Topographic Map of Quadrangles 3460 and 3360, Kol-I-Namaksar (407), Ghuryan (408), Kawir-I-Naizar (413), and Kohe-Mahmudo-Esmailjan (414) Quadrangles, Afghanistan. U.S. Geological Survey Open File Report 2005-1103-B, compiled by Robert G. Bohannon.

Topographic Map of Quadrangle 3462, Herat (409) and Chesht-Sharif (410) Quadrangles, Afghanistan. U.S. Geological Survey Open File Report 2005-1104-B, compiled by Robert G. Bohannon.

Topographic Map of Quadrangle 3464, Shahrak (411) and Kasi (412) Quadrangles, Afghanistan. U.S. Geological Survey Open File Report 2005-1105-B, compiled by Robert G. Bohannon.

Topographic Map of Quadrangle 3466, Lal-Sarjangal (507) and Bamyan (508) Quadrangles, Afghanistan. U.S. Geological Survey Open File Report 2005-1106-B, compiled by Robert G. Bohannon.

Topographic Map of Quadrangle 3468, Chak Wardak-Syahgerd (509) and Kabul (510) Quadrangles, Afghanistan. U.S. Geological Survey Open File Report 2005-1107-B, compiled by Robert G. Bohannon.

Topographic Map of Quadrangle 3470 and the Northern Edge of Quadrangle 3370, Jalal-Abad (511), Chaghasaray (512), and Northernmost Jaji-Maydan (517) Quadrangles, Afghanistan. U.S. Geological Survey Open File Report 2005-1108-B, compiled by Robert G. Bohannon.

Topographic Map of Quadrangle 3362, Shin-Dand (415) and Tulak (416) Quadrangles, Afghanistan. U.S. Geological Survey Open File Report 2005-1109-B, compiled by Robert G. Bohannon.

Topographic Map of Quadrangle 3364, Pasa-Band (417) and Kejran (418) Quadrangles, Afghanistan. U.S. Geological Survey Open File Report 2005-1110-B, compiled by Robert G. Bohannon.

Topographic Map of Quadrangle 3366, Gizab (513) and Nawer (514) Quadrangles, Afghanistan. U.S. Geological Survey Open File Report 2005-1111-B, compiled by Robert G. Bohannon.

Topographic Map of Quadrangle 3368 and Part of Quadrangle 3370, Ghazni (515), Gardez (516), and Part of Jaji-Maydan (517) Quadrangles, Afghanistan. U.S. Geological Survey Open File Report 2005-1112-B, compiled by Robert G. Bohannon.

Topographic Map of Quadrangles 3260 and 3160, Dasht-E-Chahe-Mazar (419), Anardara (420), Asparan (601), and Kang (602) Quadrangles, Afghanistan. U.S. Geological Survey Open File Report 2005-1113-B, compiled by Robert G. Bohannon.

Topographic Map of Quadrangle 3262, Farah (421) and Hokumat-E-Pur-Chaman (422) Quadrangles, Afghanistan. U.S. Geological Survey Open File Report 2005-1114-B, compiled by Robert G. Bohannon.

Topographic Map of Quadrangle 3264, Nawzad-Musa-Qala (423) and Dehrawat (424) Quadrangles, Afghanistan. U.S. Geological Survey Open File Report 2005-1115-B, compiled by Robert G. Bohannon.

Topographic Map of Quadrangle 3266, Ourzgan (519) and Moqur (520) Quadrangles, Afghanistan. U.S. Geological Survey Open File Report 2005-1116-B, compiled by Robert G. Bohannon.

Topographic Map of Quadrangles 3168 and 3268, Yahya-Wona (703), Wersek (704), Khayr-Kot (521), and Urgon (522) Quadrangles, Afghanistan. U.S. Geological Survey Open File Report 2005-1117-B, compiled by Robert G. Bohannon.

Topographic Map of Quadrangle 3162, Chakhansur (603) and Kotalak (604) Quadrangles, Afghanistan. U.S. Geological Survey Open File Report 2005-1118-B, compiled by Robert G. Bohannon.

Topographic Map of Quadrangle 3164, Lashkargah (605) and Kandahar (606) Quadrangles, Afghanistan. U.S. Geological Survey Open File Report 2005-1119-B, compiled by Robert G. Bohannon.

Topographic Map of Quadrangle 3166, Jaldak (701) and Maruf-Nawa (702) Quadrangles, Afghanistan. U.S. Geological Survey Open File Report 2005-1120-B, compiled by Robert G. Bohannon.

Topographic Map of Quadrangles 3060 and 2960, Qala-I-Fath (608), Malek-Sayh-Koh (613), and Gozar-E-Sah (614) Quadrangles, Afghanistan. U.S. Geological Survey Open File Report 2005-1121-B, compiled by Robert G. Bohannon.

Topographic Map of Quadrangles 3062 and 2962, Charburjak (609), Khanneshin (610), Gawdezereh (615), and Galachah (616) Quadrangles, Afghanistan. U.S. Geological Survey Open File Report 2005-1122-B, compiled by Robert G. Bohannon.

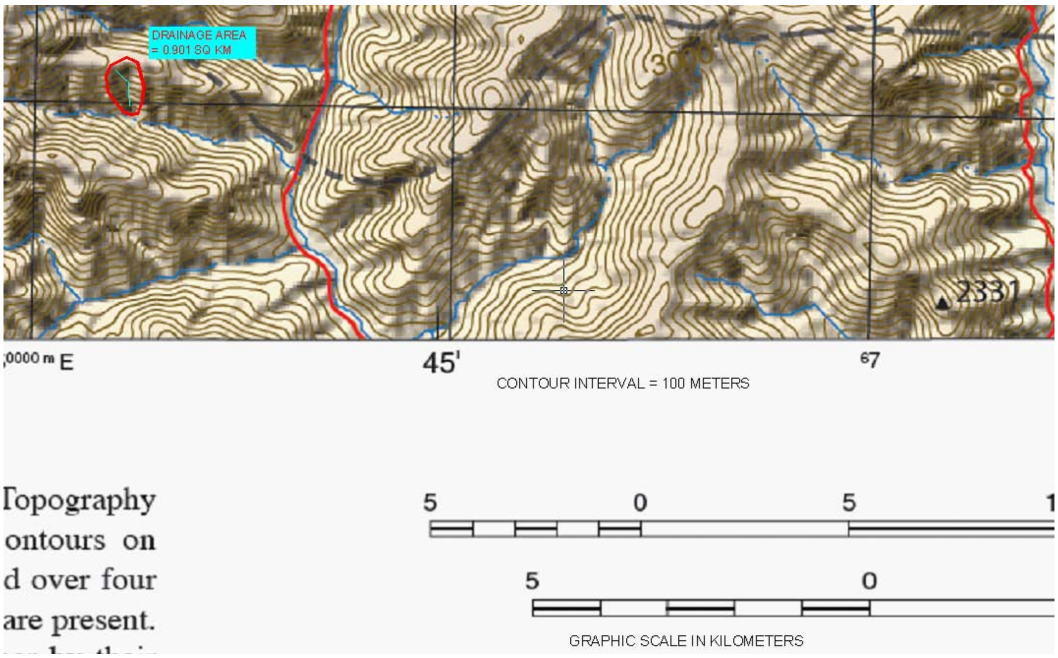
Topographic Map of Quadrangles 3064, 3066, 2964, and 2966, Laki-Bander (611), Jahangir-Naweran (612), Sreh-Chena (707), Shah-Esmail (617), Reg-Alaqadari (618), and Samandkhan-Karez (713) Quadrangles, Afghanistan. U.S. Geological Survey Open File Report 2005-1123-B, compiled by Robert G. Bohannon.

MicroImages, Inc. • 11th Floor - Sharp Tower • 206 South 13th Street • Lincoln, Nebraska • 68508-2010 • USA
Voice (402)477-9554 • FAX (402)477-9559 • email info@microimages.com • web www.microimages.com • June 2007

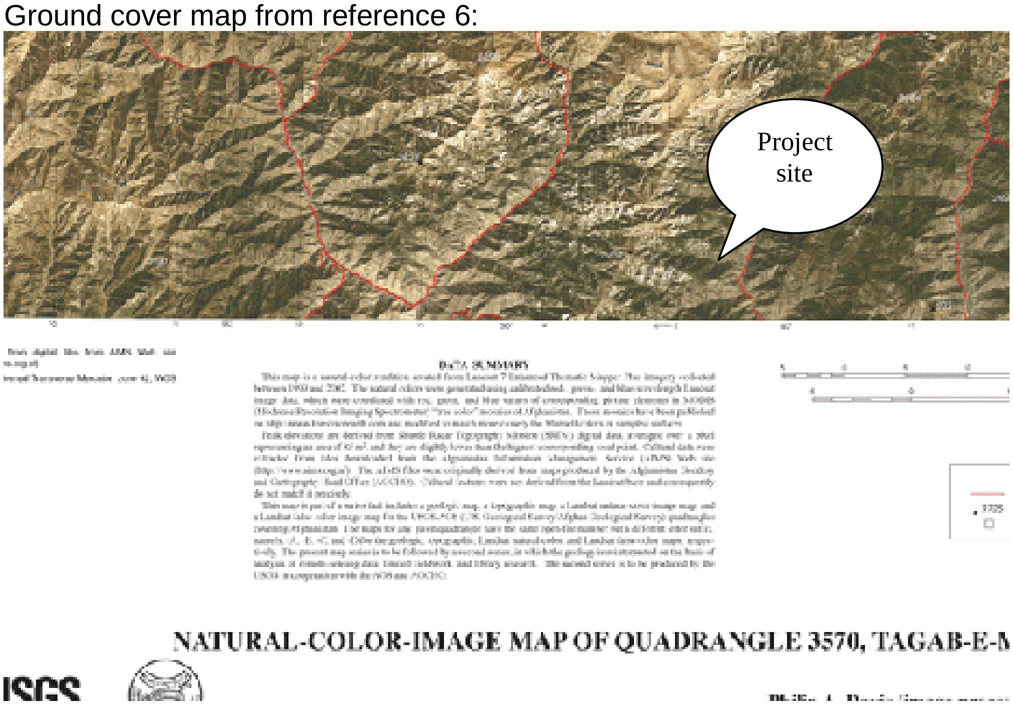
Appendix A Example of Rational Method

A site review for a road project in Nuristan Province has determined that a culvert should be designed based on the ten-year storm peak flow using the rational method. The location of the project is shown on a topographic map obtained from the internet site listed in reference 6. The web site was used to obtain a photograph that shows the ground cover conditions. These are shown below:

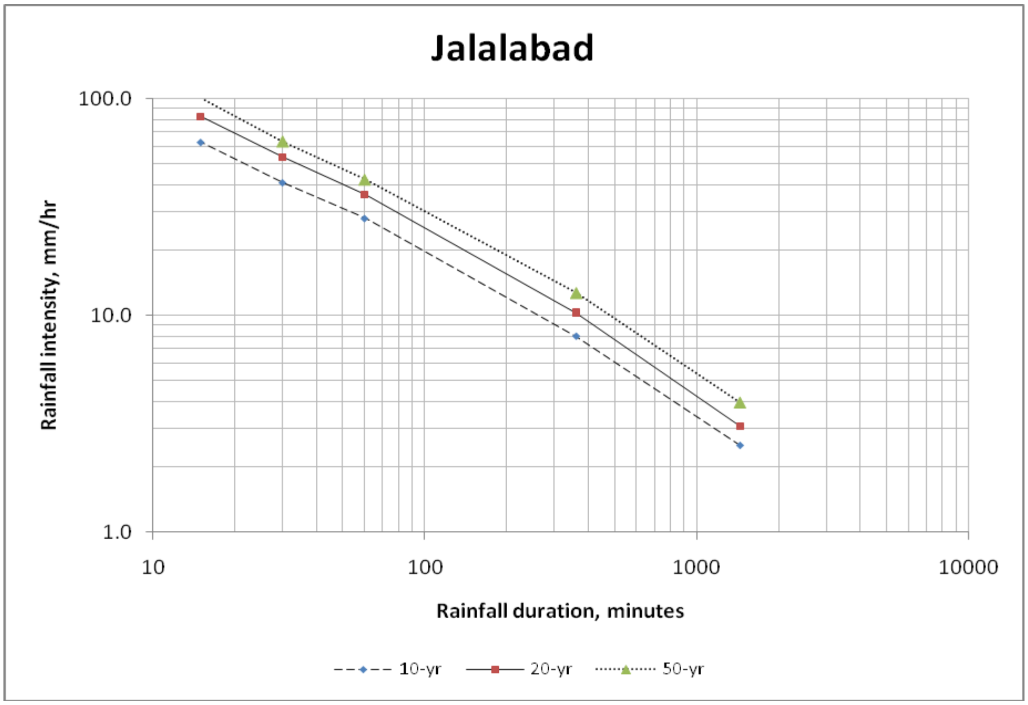
Drainage area map determined using AutoCAD



Ground cover map from reference 6:



Jalalabad IDF Chart:



Based on Topographic Map:

Step

1 **To = Overland flow time for concentration = 0.275*(Lo^0.42)*(So^-0.19)*(C^-1.0)**

So = $\frac{\Delta \text{ Elevation} =}{\text{Ls}}$ $\frac{984}{1{,}273}$ **Based on map contour over longest flow distance**
Longest length required for flow

So = 0.77

Lo (ft) = 492 **Length of shallow overland flow (Max = 150 m)**

C = 0.3 **Runoff coefficient from Table 1**
0.3 = coefficient for greater than 10% slope, semi desert

To = 0.275*(492^0.42)*(0.77^-0.19)*(0.3^-1.0) = 13 minutes

2 **Tc = Unpaved Watercourse flow time of concentration = L/Vavg = L/16.441*(Sc^0.5603)**

Sc = $\frac{\Delta \text{ Elevation} =}{\text{Lc}}$ $\frac{1{,}312}{1{,}804}$ **Based on map contour over longest flow distance**
Based on map measurement of channel length

Sc = 0.73

Tc = $\frac{1{,}804}{16.441*(0.73\text{^}0.5603)}$ = 129 seconds = 2.2 min

3 **T = Total flow time for concentration / Rainfall Duration (minutes) =**

To + Tc = 13 min + 2 min = 15 minutes

4 **Drainage Area = 0.901 km^2**

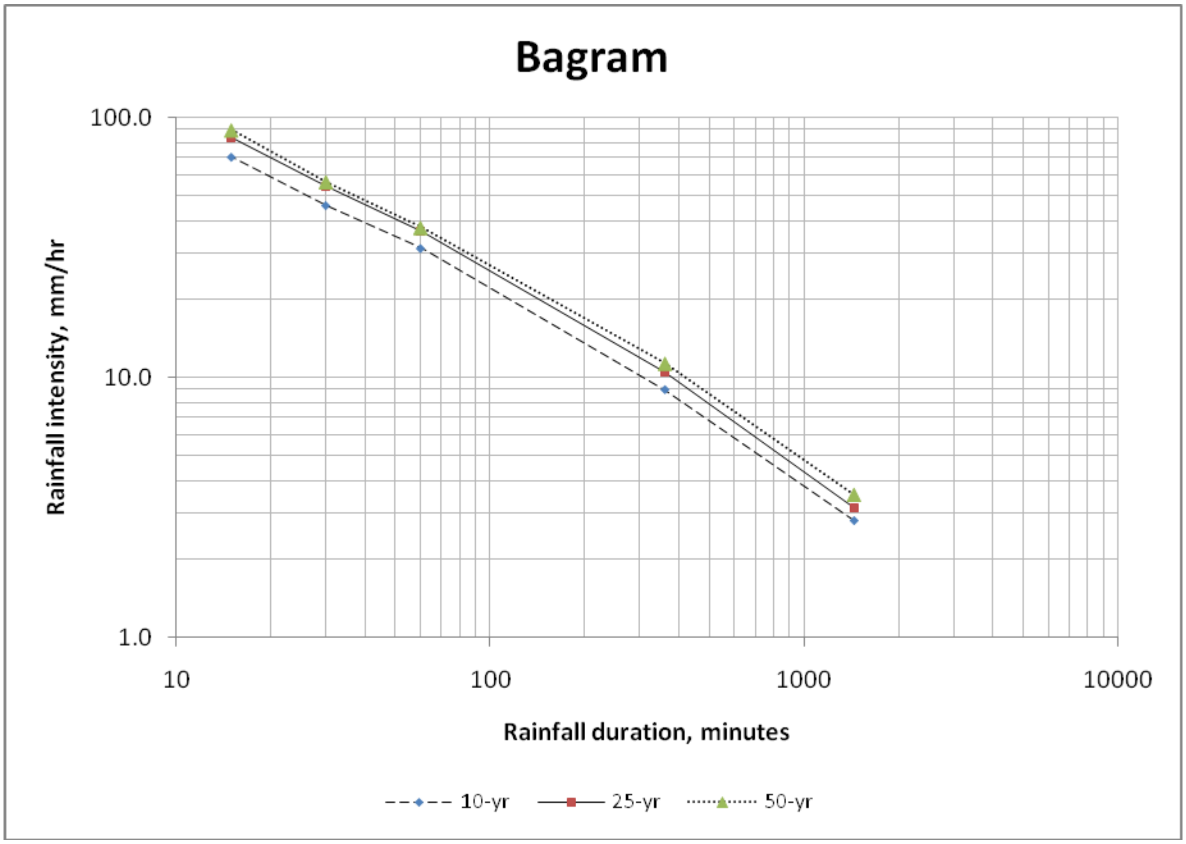
5 **Ten year storm rainfall intensity = Based on Area Rainfall Intensity Hydrograph**

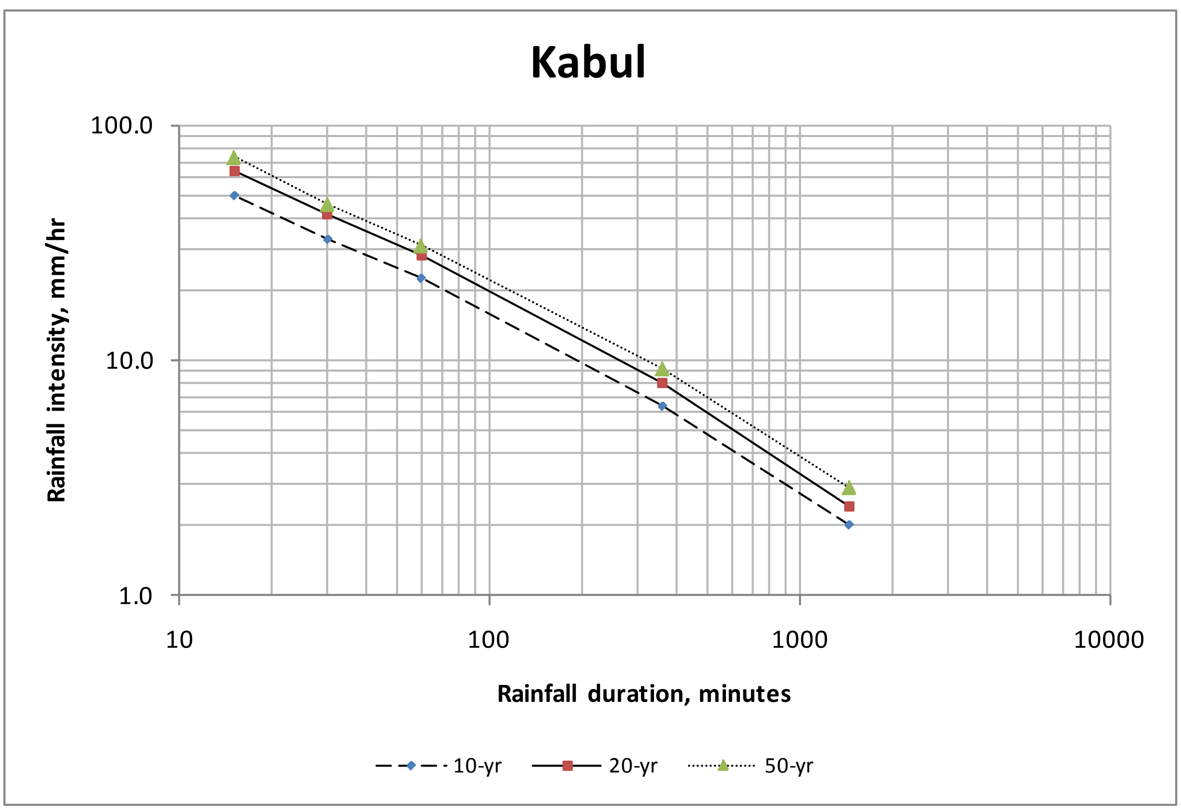
I = 62.0 mm/hr

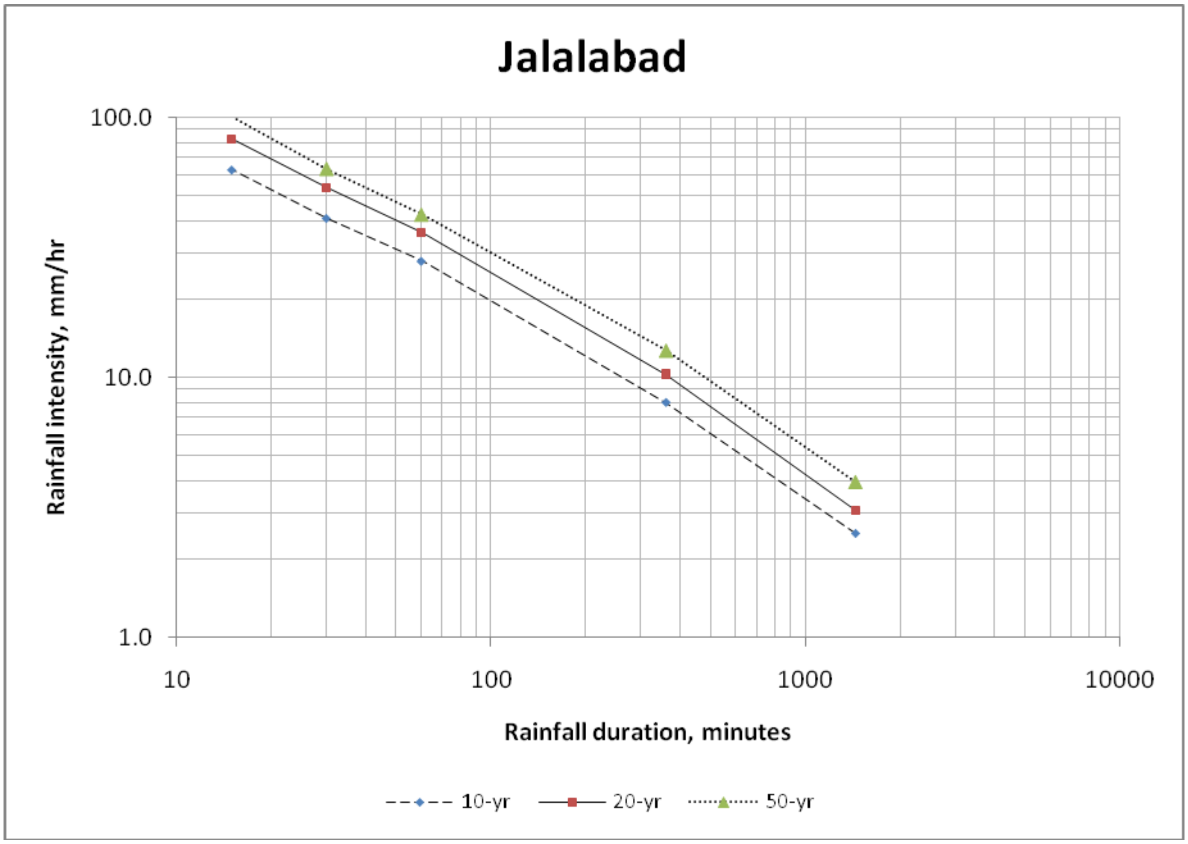
6 **Q = Flow at time of concentration (ratinoal method) = Q = 0.278*C*A*I**

Q = 0.278*.3*0.901*14.0 = 4.65 m^3 / sec

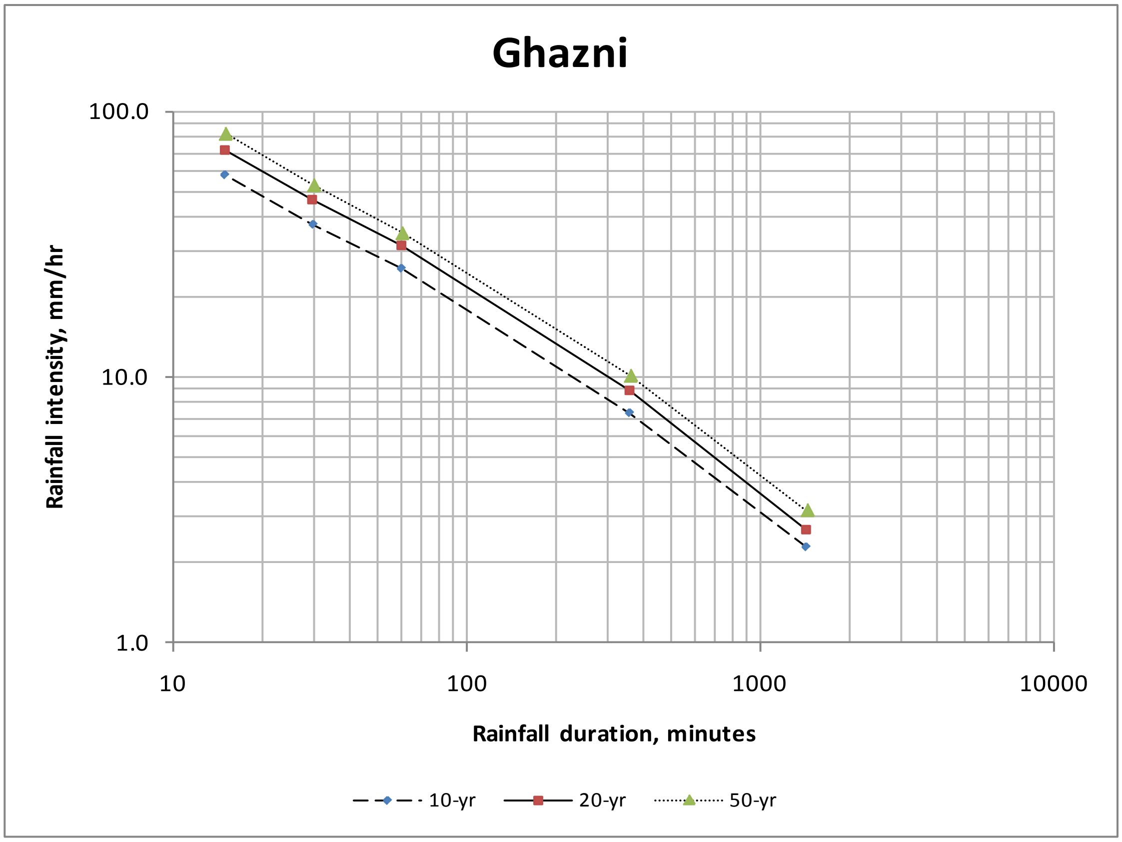
Appendix B Intensity-Duration-Frequency Curves for Selected Project Regions

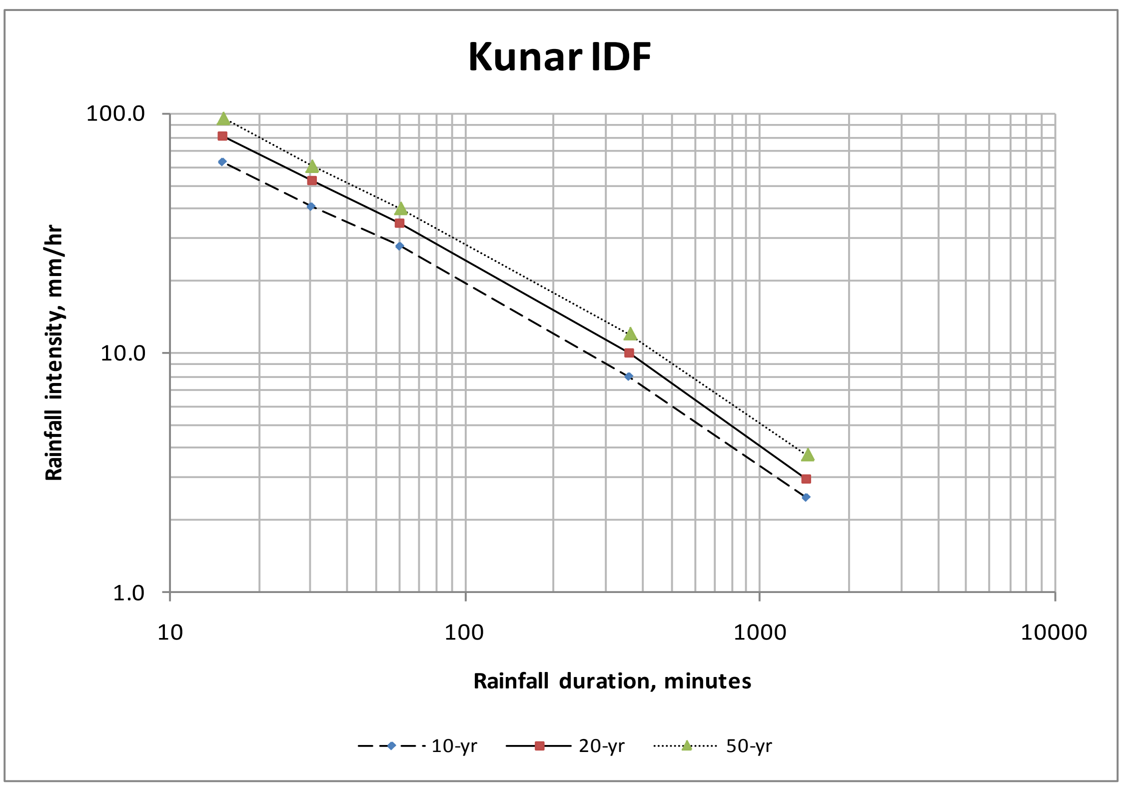


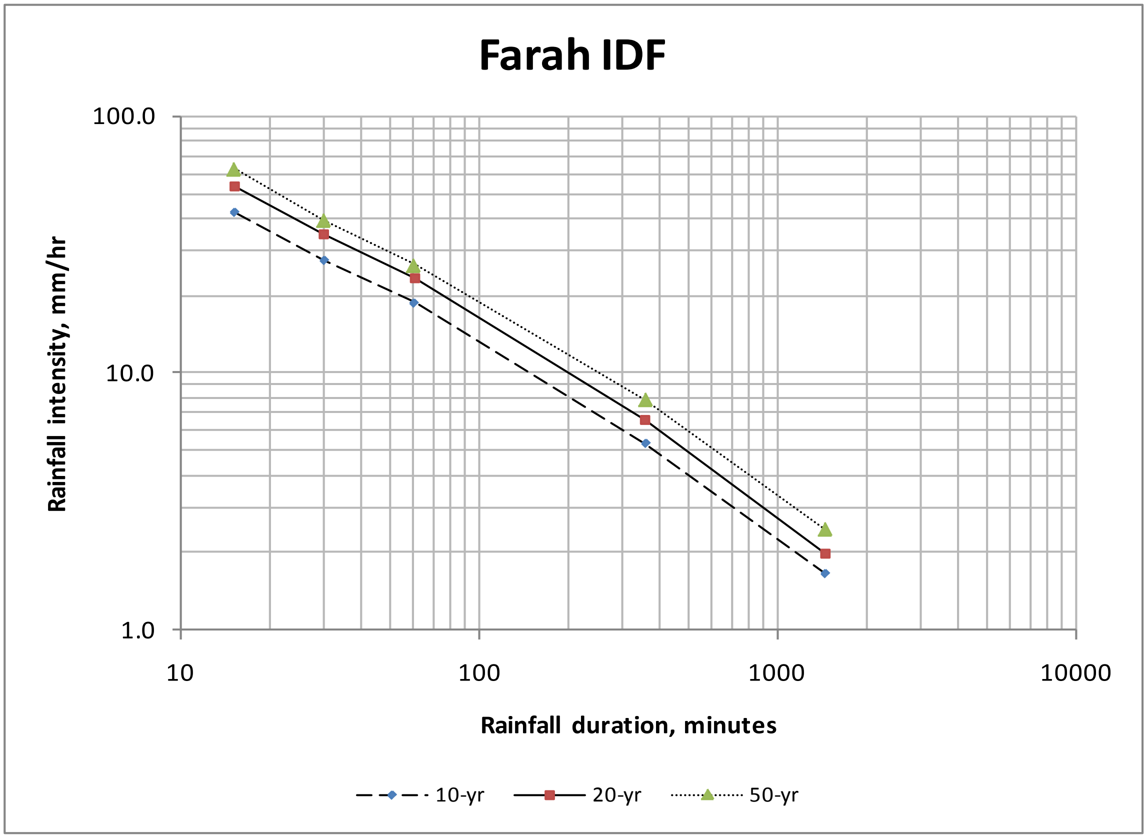


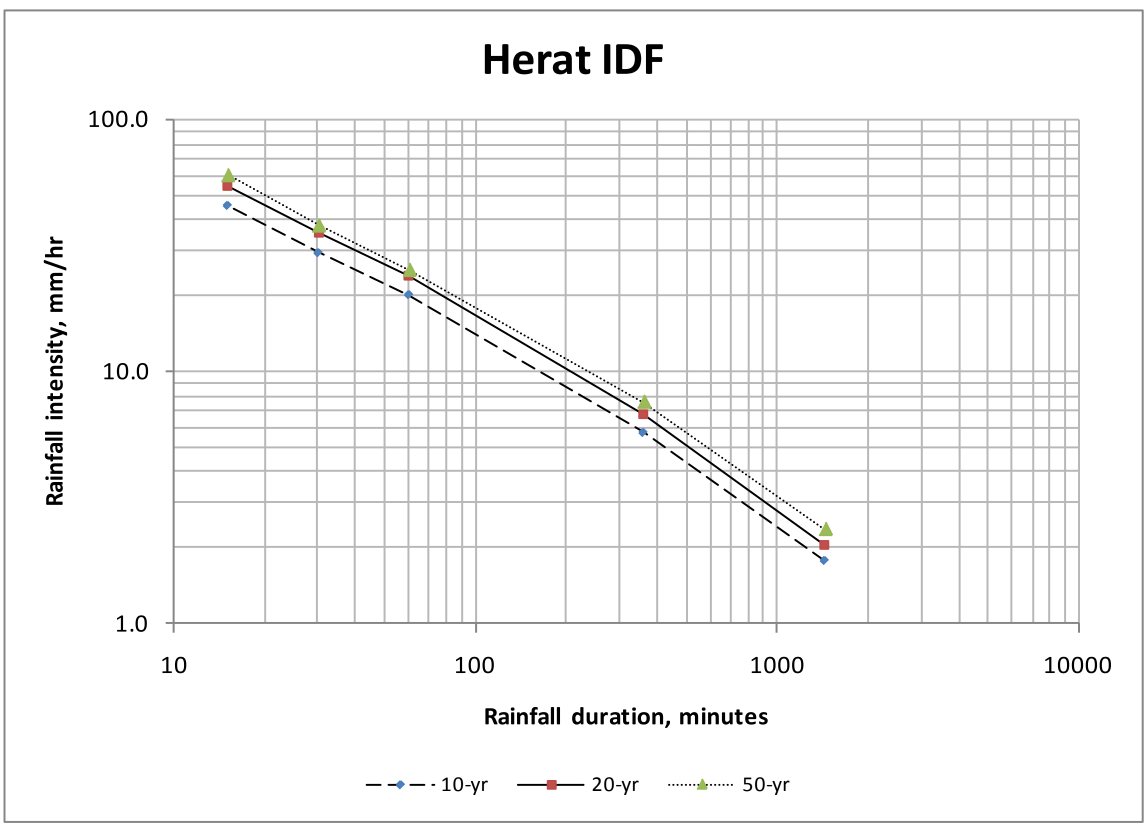

Jalalabad
100.0
10.0
1.0
10
100
1000
10000
Rainfall intensity, mm/hr
Rainfall duration, minutes
10-yr
20-yr
50-yr

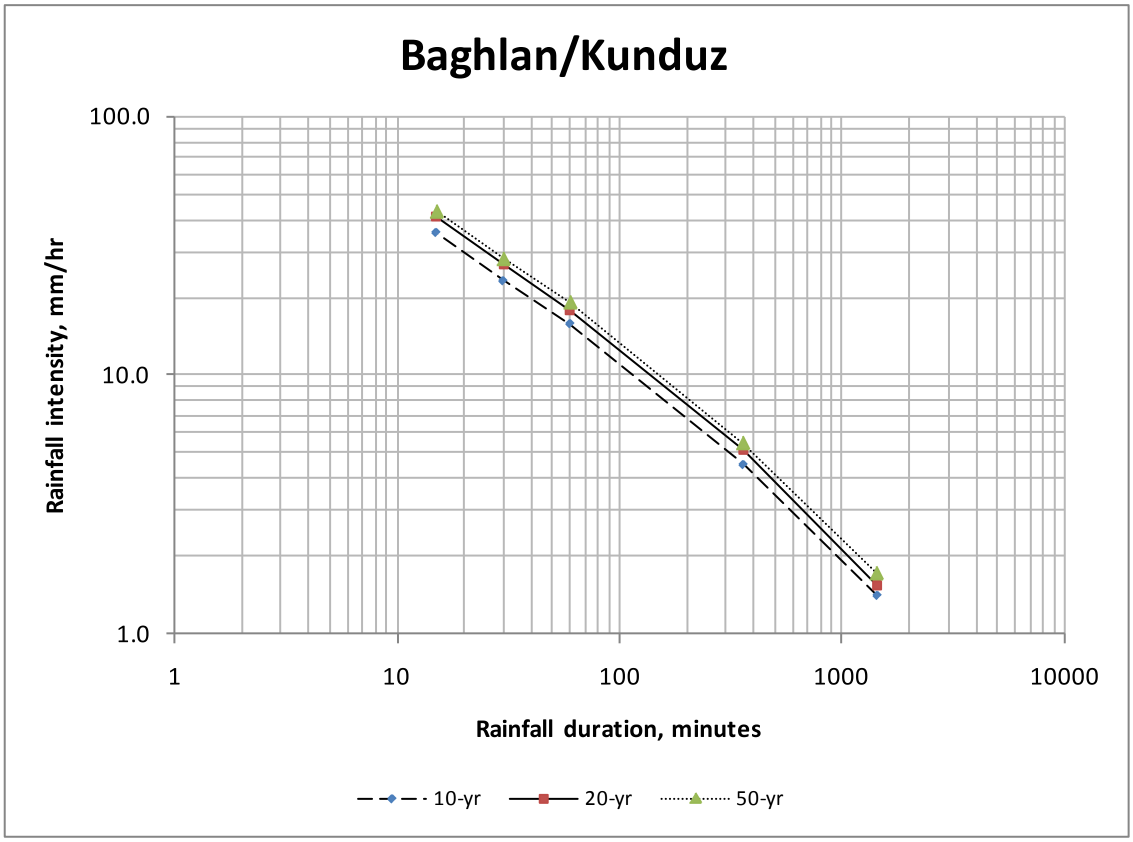

Gardez
100.0
10.0
1.0
10
100
1000
10000
Rainfall intensity, mm/hr
Rainfall duration, minutes
10-yr
20-yr
50-yr


Ghazni
100.0
10.0
1.0
10
100
1000
10000
Rainfall intensity, mm/hr
Rainfall duration, minutes
10-yr
20-yr
50-yr


Kunar IDF
100.0
10.0
1.0
10
100
1000
10000
Rainfall intensity, mm/hr
Rainfall duration, minutes
10-yr
20-yr
50-yr


Farah IDF
100.0
10.0
1.0
10
100
1000
10000
Rainfall intensity, mm/hr
Rainfall duration, minutes
10-yr
20-yr
50-yr


Herat IDF
100.0
10.0
1.0
10
100
1000
10000
Rainfall intensity, mm/hr
Rainfall duration, minutes
10-yr
20-yr
50-yr



NOTE:

If rainfall data for a 10-year storm event is not available within a particular project site area, a value of 50mm/hr shall be used in calculating design flows.

Appendix C Area Factor for Point Rainfall Reduction Curves

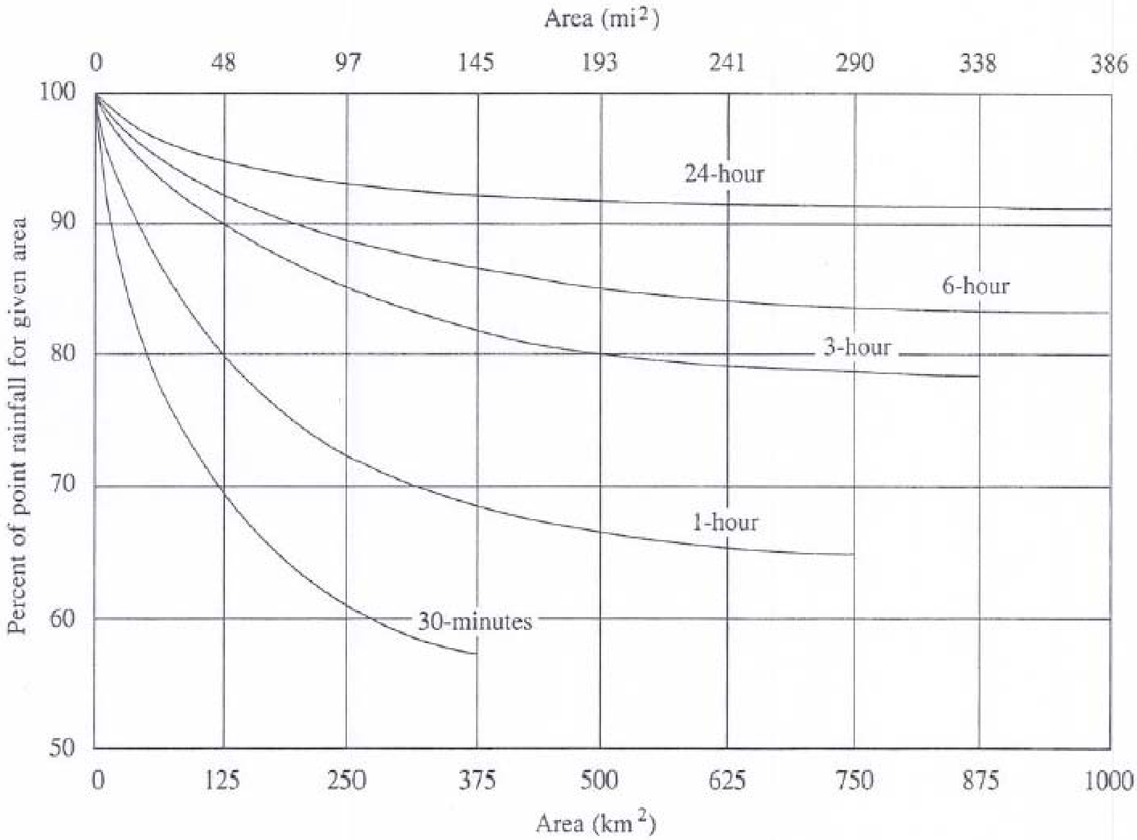


FIGURE 14.1.3
Depth-area curves for reducing point rainfall to obtain areal average values. (*Source*: World Meteorological Organization, 1983; originally published in Technical Paper 29, U. S. Weather Bureau, 1958.)

US Army Corps
of Engineers
Afghanistan Engineer District

AED Design Requirements: Sanitary Sewer & Septic System

Various Locations,
Afghanistan

June 2010
Version 1.6

TABLE OF CONTENTS

AED DESIGN REQUIREMENTS
FOR
SANITARY SEWERS & SEPTIC TANKS
VARIOUS LOCATIONS,
AFGHANISTAN

Tables **Page**

Figures **Page**

Examples **Page**

<u>Appendix</u>

1. General

The purpose of this document is to provide requirements to Contractors for any project requiring sanitary sewer and septic system design and construction. Effluent disposal is typically provided by leach fields, absorption beds, leaching chambers, or seepage pits. Septic systems are considered appropriate where the native soil conditions natural percolation properties provide biological treatment of the wastewater after receiving primary treatment in a septic tank. Because of the difficulty of distributing effluent uniformly over larger sites and the impacts of its application on the underlying groundwater, septic tanks are typically limited to projects where the total effective design population is less than 650 personnel. Depending upon the water usage rate adopted for the project, this is typically an average daily flow rate of approximately 121,000 to 148,000 liter per day (32,000 to 39,000 gallons per day) assuming 80% wastewater generated from water usage and a capacity factor of 1.5 for treatment.

Holding tanks may be authorized as an alternative to effluent disposal means in the project contract technical requirements. Because the costs of hauling wastewater and the uncertainty in its sanitary disposal once offsite are greater, holding tanks should be limited to smaller installations when no other alternatives are possible. The break even cost for package WWTP construction and annual operating cost versus the annual septage hauling cost for holding tanks indicates that installations having design populations in excess of approximately 650 personnel would be better served by a package WWTP provided sufficient land and effluent disposal means are available.

2. Field Investigations

a) Site Survey. The first step, when designing the sewer system, is determining existing site conditions. The existing site conditions shall be determined by conducting field investigations at the proposed site. As part of the field investigations, the Contractor shall conduct a topographic survey to determine existing site characteristics. Knowing this information will help determine whether a gravity system or a pressure system will be used and where to locate the septic system. In addition, the Contractor shall conduct a utility survey to determine the locations of any nearby water lines, wells, sanitary sewers, storm sewers and electrical lines. By knowing the location of the existing utilities, the Contractor can properly lay out the system.

b) Percolation Testing. The second step, once the site has been surveyed, is to perform percolation tests. While performing the tests, observe the soil characteristics and watch for groundwater within the test area. The site may be considered unsuitable if the following occurs: the soil appears to have too much sand or clay; groundwater is encountered; and/or the percolation rates are too slow. If the site is determined to be unsuitable, the septic system will need to be relocated. If another location cannot be found, then an alternative treatment system will need to be designed. If this happens, contact the COR.

Percolation testing may be carried out with a shovel, posthole digger, solid auger or other appropriate digging instruments. Percolation tests shall be accomplished uniformly throughout the area where the absorption field is to be located. Percolation tests determine the acceptability of the site and serve as the basis of design for the liquid absorption. Percolation tests will be made as follows (see Figure 1).

(1) Three or more tests will be made in separate test holes uniformly spaced over the proposed absorption field site. The average of the six tests shall be determined and will be used as the final result. ***The location of each test shall be clearly and accurately shown on the site plan submitted to AED***.

(2) Dig or bore a hole to the required depth of the proposed trenches or bed, with dimensions necessary to enable visual inspection during percolation testing.

(3) Carefully scratch the bottom and sides of the excavation with a knife blade or sharp-pointed instrument to remove any smeared soil surfaces and to provide a natural soil interface into which water may percolate. Add 50 mm of gravel (of the same size that is to be used in the absorption field) to the bottom of the hole. In some types of soils the sidewalls of the test holes tend to cave in or slough off and settle to the bottom of the hole. It is most likely to occur when the soil is dry or when overnight soaking is required. The caving can be prevented and more accurate results obtained by placing in the test hole a wire cylinder surrounded by a minimum 25 mm layer of gravel (of the same size that is to be used in the absorption field.)

(4) Carefully fill the hole with clear water to a minimum depth of 300 mm above the gravel or sand. Keep water in the hole at least 4 hours and preferably overnight. In most soils it will be necessary to augment the water as time progresses. Determine the percolation rate 12 to 24 hours after water was first added to the hole. In sandy soils containing little clay, this pre-filling procedure is not essential and the test may be made after water from one filling of the hole has completely seeped away.

(5) The percolation-rate measurement is determined by one of the following methods:
(a) If water remains in the test hole overnight, adjust the water depth to approximately 150 mm above the gravel. From a reference batter board, as shown in Figure 1, measure the drop in water level over a 30-minute period. This drop is used to calculate the percolation rate.

(b) If no water remains in the hole the next day, add clean water to bring the depth to approximately 150 mm over the gravel. From the batter board, measure the drop in water level at 30-minute intervals for 4 hours, refilling to 150 mm over the gravel as necessary. The drop in water level that occurs during the final 30-minute period is used to calculate the percolation rate.

(c) In sandy soils (or other soils in which the first 150 mm of water seeps away in less than 30 minutes after the overnight period), the time interval between measurements will be taken as 10 minutes and the test run for 1 hour. The drop in water level that occurs during the final 10 minutes is used to calculate the percolation rate.

The percolation rate is the number of minutes it takes to drop 25 mm. On page 10, Table 2 lists percolation rates and the corresponding absorption field sizing factor (liters/m^2/day). The sizing factors are used, in conjunction with average daily demand (ADD), to determine the size of the absorption field. The following is an example of how to calculate the percolation rate:

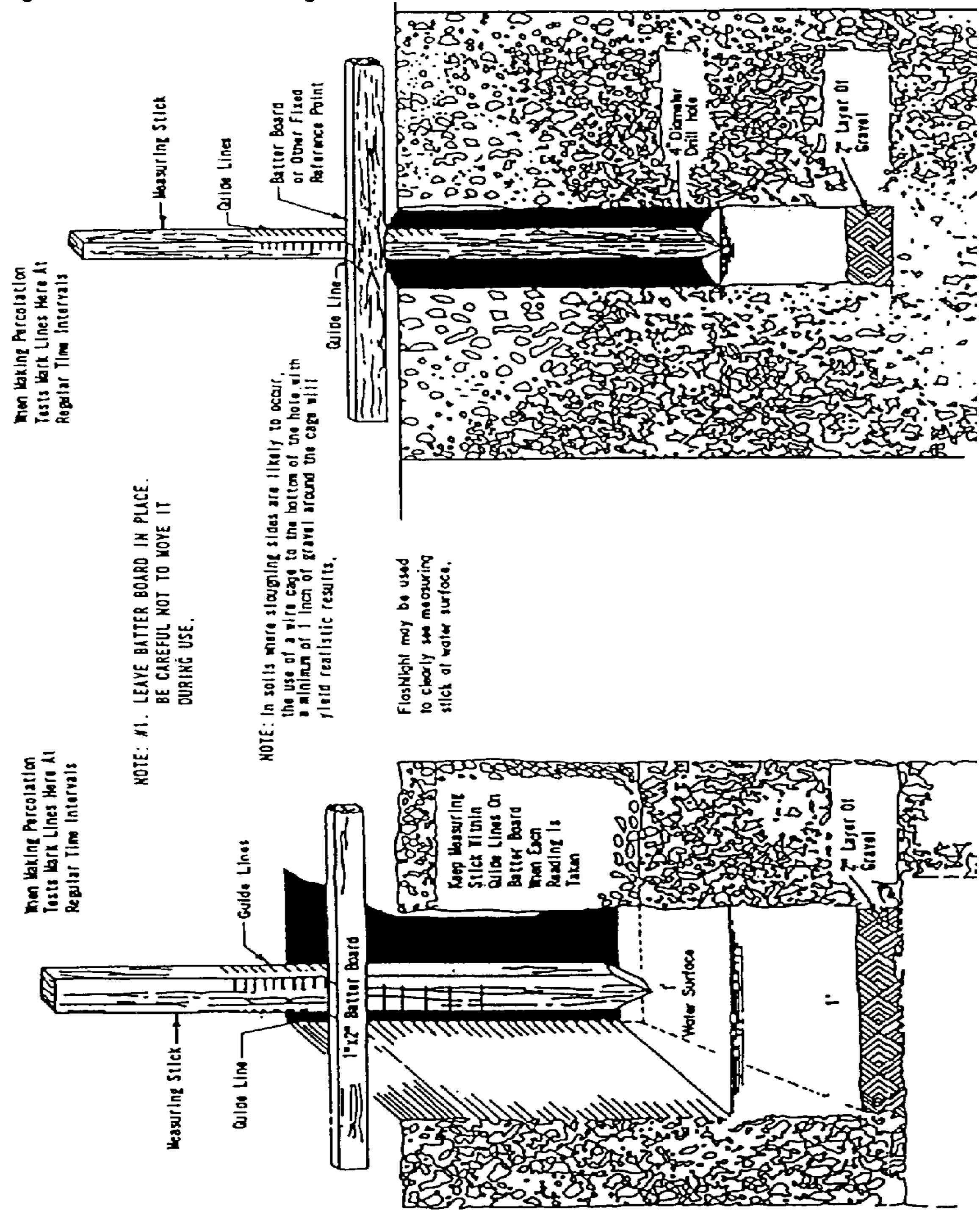
Example 1: Calculating Percolation Rates - In 30 minutes, the measured drop in the water level is 15 mm.

Minutes/25 mm = Time/(drop/25 mm) = 30 minutes/(15 mm/25 mm) = ***<u>50 Minutes/25 mm</u>***

where,

Minutes/25 mm = Minutes for water to drop 25 mm.

Figure 1. Percolation Testing.



3. Sanitary Sewer System

a) Sanitary Sewer System Layout. The development of the sewer system (a.k.a. - sanitary pipe collection network) must await the determination of the proposed compound layout, including determining locations for: buildings (including final first floor elevations and utility connections), perimeter wall, and roads, water well, septic system, power supply system, storm drainage and other features. Once the locations for these structures are determined, the Contractor can begin designing the layout of the sanitary sewers in conjunction with the water supply system. The following general criteria will be used where possible to provide a layout which is practical and economical and meets hydraulic requirements:

(1) Follow slopes of natural topography for gravity sewers.

(2) Check subsurface investigations for groundwater levels and types of subsoil encountered. If possible, avoid areas of high groundwater and the placement of sewers below the groundwater table.

(3) Avoid routing sewers through areas which require extensive restoration or underground demolition.

(4) Depending upon the topography and building location, the most practical location of sanitary sewer lines is along one side of the street. In other cases they may be located behind buildings midway between streets. The intent is to provide future access to the lines for maintenance without impacting vehicular traffic.

(5) Avoid placing manholes in low-lying areas where they could be submerged by surface water or subject to surface water inflow. In addition, all manholes shall be constructed 50 mm higher than the finished grade, with the ground sloped away from each manhole for drainage.

(6) Sewer lines shall have a minimum of 800 mm of cover for frost protection.

(7) Locate manholes at change in direction, pipe size or slope of gravity sewers.

(8) Sewer sections between manholes shall be straight. The use of a curved alignment shall not be permitted.

(9) If required by the design, locate manholes at intersections of streets where possible. This will minimize vehicular traffic disruptions if maintenance is required.

(10)Sewer lines less than 1.25 meters deep under road crossings shall have a reinforced concrete cover of at least 150 mm thickness around the pipe or shall utilize a steel or ductile iron carrier pipe. It is recommended to continue the reinforced concrete cover or carrier pipe a minimum of one (1) meter beyond the designated roadway.

(11)Sewer lines entering a manhole shall not be less than 90 degrees to the orientation of the sewer line leaving the manhole.

(12)Verify that final routing selected is the most cost effective alternative that meets service requirements.

b) Protection of water supplies. Sanitary sewer design shall meet the following criteria:

(1) Sewers shall be located no closer than 15 meters measured horizontally to water wells or earthen reservoirs that are used for potable water supplies.

(2) Sewers shall be located no closer than 3 meters measured horizontally to potable water lines; where the bottom of the water line will be at least 300 mm above the top of the sewer line, the horizontal space shall be at a minimum of 1.83 meters.
(3) Sewer lines crossing above potable water lines shall be constructed of suitable pressure pipe or fully encased in concrete for a distance of 3 meters measured horizontally on each side of the crossing. If concrete encasement is used, the sewer line shall be encased with a minimum of 150 mm of cover all the way around the pipe. Pressure pipe will be as required for force mains in TM 5-814-2/AFM 88-11, Chapter 2, and shall have no joint closer than 1 meter horizontally to the crossing, unless it is fully encased in concrete.

c) Quantity of Wastewater. The design of the wastewater system shall be based on two factors: the average daily flow and the peak diurnal flow (PDF).

(1) Average Daily Flow (ADF). The Contractor shall verify the average daily flow considering both resident (full occupancy) and non-resident (8hr per day) population. The average daily flow will represent the total waste volume generated over a 24-hour period, and is defined as 80% of the product of the total population of the facility (c), the per capita water usage rate per day (ADD) , and the applicable capacity factor (CF) (0.8* c*ADD*CF). The capacity factor for installations with populations less than 5,000 residents is 1.5. Capacity factors for larger installations shall be determined using Chapter 4 Basic Design Considerations, UFC 3-240-09FA Domestic Wastewater Treatment, Table 4-1. For example, the average daily flow at a compound with a population of 500 personnel, would be calculated by multiplying the population (500) by the water usage rate (190 lpcd) by the capacity factor (1.5) by 80% resulting in a flow of 114,000 liters/day (30,160 gallons per day).

(2) Peak diurnal rates (PDF) of flow occur on a daily basis and must be considered. The sewer shall be designed with adequate capacity to handle these peak diurnal flow rates. The peak diurnal flow rate is computed by the following equation:

$$PDF= \frac{Q\ C}{2Q^{0.167}}$$

Where PDF = Peak diurnal flowrate
Q = Average daily flow in gallons per day (including the capacity factor)
C = 38.2 for gallons per day

So for the same compound with a population of 500 personnel, the peak diurnal flow rate would be the 114,000 liters/day (calculated above multiplied by 38.2 divided by 2 times $(30{,}160)^{0.167}$ which equals 389,027.3 liters per day.

d) Gravity Sewers. The method for designing the sanitary sewers shall be determined according to the installation population.

(1) For installations with populations less than 450 personnel, all sewer pipe slopes shall be a minimum of 1.0%, regardless of pipe size. If this slope and surface topography force the laterals in the absorption field more than 1500mm below the surface of the ground, a lift station is necessary. When this occurs, the designer should contact AED immediately to discuss options. Gravity flows are always desirable, but lift stations may be necessary in certain circumstances.

(A) Minimum Pipe Diameter. The minimum pipe diameter used in the sewer system for this size installation (after the building plumbing connection) shall be 150mm. This diameter pipe may be used throughout the installation. This shall be the minimum size. Larger pipe diameters may be used, but should be used only if flows require a larger diameter.

(2) For installations with populations greater than or equal to 450 personnel the sanitary sewer shall be designed to meet the following conditions. If, based on the following conditions, it is determined that a lift station is necessary to meet the flow and pipe size requirements than the designer should contact AED immediately.

(A) Peak Diurnal Flow (PDF). Piping shall be designed to provide a minimum velocity of 0.6 meters per second (mps) or (2.0 feet per second (fps) and shall NOT flow at greater than 80% full or at a velocity greater than 3.0 mps (10 fps). It is required that all the pipes are designed to achieve a scouring velocity of 0.6 mps at the PDF.

(B) Average Daily Flow (ADF). When possible, piping shall be designed to provide a minimum scouring velocity of 0.6 mps (2.0 fps) at the ADF, and shall NOT flow at greater than 80% full or at a velocity greater than 3.0 mps (10 fps) in every segment of the sewer system. It is preferred that the scouring velocity be achieved by the ADF however it is not a requirement

(C) Flow Allocation. Flows in laterals, mains and trunk lines shall be based on allocating the proportion of the average daily and peak diurnal flow to each building or facility on the basis of the drain fixture unit flow developed for the plumbing design. [**For example**, consider a lateral receiving flow first from building A, then from building B and then from building C prior to emptying into a main. These buildings have drain fixture units of 10, 25, and 5, respectively. The entire facility has a total of 6 building and a total of 80 drain fixture units. The flows used to design the lateral receiving flow from building A would be 10/80 times the ADF and the PDF. The flows used to design the lateral after receiving flow from buildings A and B would be (10+25)/80 times the ADF and the PDF. Finally, the flows used to design the lateral after receiving flows from buildings A, B, and C would be (10+25+5)/80 times the ADF and the PDF.]

(D) Minimum Pipe Slopes. Table 1 defines the minimum pipe slopes allowed in the sewer system. These shall be the minimum provided, regardless of the calculated flow velocities to prevent settlement of solids suspended in the wastewater. Table 1 does not apply to building connections.

(E) Building connection. Sewer lines from buildings will be designed to provide a minimum velocity of 0.6 meters per second or 2.0 feet per second at the drain fixture unit flow for that building. The building connection is the pipe from the building to a manhole or pipe that has more than one pipe entering it, see Figure 2. The minimum slope of building connection shall be 1% regardless the size of the installation.

(F) Minimum Pipe Diameter. The minimum pipe diameter used in the sewer system (after the building plumbing connection) shall be 150mm. These sizes shall be provided regardless of flows being received. Larger pipe diameters shall be provided in the sewer system based on flow and velocity requirements.

Unless otherwise indicated (see Paragraph 3 (g) Building Connections and Service Lines below), gravity sewer pipe shall be installed in straight and true runs in between manholes with constant slope and direction. Pipe slopes shall be sufficient to provide the required minimum velocities and depths of cover on the pipe. Table 1 below provides the minimum allowable slopes for various diameter pipes. Table 1 does not apply to installations with populations less than 450 persons. The minimum slope for 150mm piping at these installations is to be 1%.

Table 1. Minimum Slopes for Sewers (Populations Greater than 450).

Sewer Size	Minimum Slope in Meters per 100 Meters
100 mm	1.00
150 mm	0.62
200 mm	0.40
250 mm	0.28
300 mm	0.22
350 mm	0.17
375 mm	0.15
400 mm	0.14
450 mm	0.12
525 mm	0.10
600 mm	0.08

This table does not state that pipes are designed at this slope regardless of flow depth and velocity. Other criteria listed above shall be used to determine the slopes necessary to meet the conditions previously listed above. The word "minimum" is defined as "the least quantity or amount possible, assignable, allowable, or the like". Greater slopes shall be used as needed to achieve the design requirements previously listed.

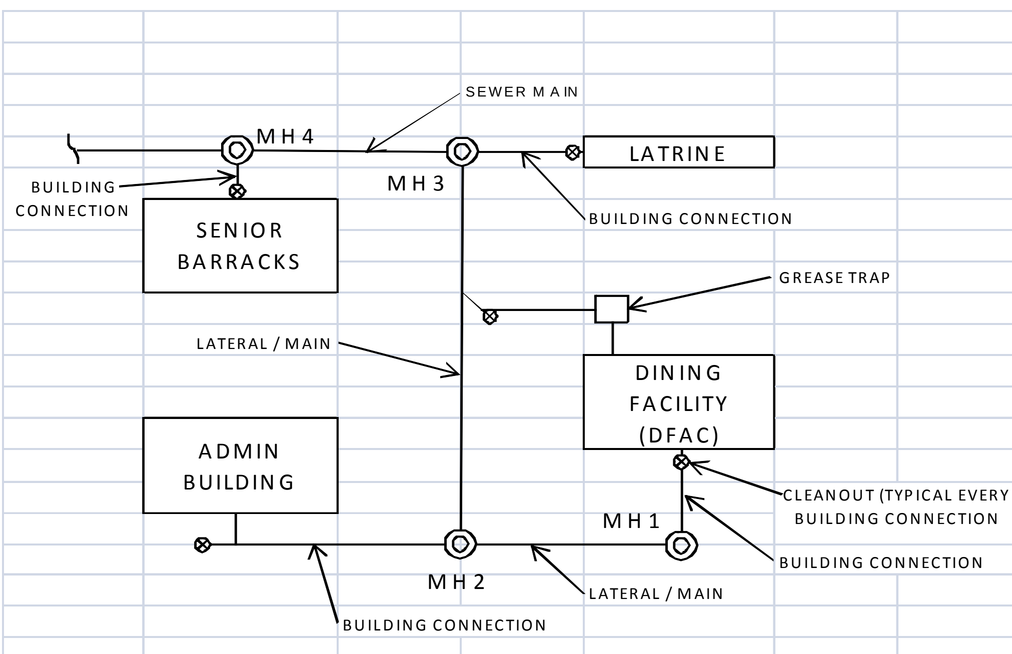


Figure 2. Schematic Pipe Definitions

e) Pipe, Fittings and Connections. Pipe, fittings and connections shall conform to the respective specifications and other requirements as listed in Contract Section 01015 and all of its referenced codes.

f) Manholes.

(1) The distance between manholes must not exceed 120 meters in sewers of less than 450 mm in diameter. For sewers 450 mm in diameter and larger, a spacing of up to 180 meters is allowed provided the velocity is sufficient to prevent settlement of solids.

(2) For pipe connections, the crown of the outlet pipe from a manhole will be on line with or below the crown of the inlet pipe. Where conditions are such as to produce unusual turbulence in the manhole, it may be necessary to provide an invert drop to allow for entry head, or increased velocity head, or both. Where the invert of the inlet pipe would be more than 450 millimeters above the manhole floor, a drop connection will be provided.

(3) Manhole frames and covers must be sufficient to withstand impact from wheel loads where subject to vehicular traffic. Covers with nominal sides measuring 762 mm or larger shall be installed where personnel entry may occur. Cover frames and/or heavy duty hinges shall prevent covers from dropping into the manholes, or circular covers shall be provided.

(4) The following construction practices will be required: 1) Smooth flow channels will be formed in the manhole bottom. Laying half tile through the manhole, or full pipe with the top of the pipe being broken out later, are acceptable alternatives; 2) for manholes over 1 meter in depth, one vertical wall with a fixed side-rail ladder will be provided; 3) drop connections will be designed as an integral part of the manhole wall and base; 4) in areas subject to high groundwater tables, manholes will be constructed of materials resistant to groundwater infiltration.

(5) The primary construction materials to be used for manhole structures are precast concrete rings and cast-in-place, reinforced concrete. Cast-in-place construction permits greater flexibility in the configuration of elements, and by varying reinforcing the strength of similar sized structures can be adjusted to meet requirements. In general, materials used should be compatible with local construction resources, labor experience, and should be cost competitive. Concrete shall have a 21 MPa minimum compressive strength at 28 days.

g) Building Connections and Service Lines. Building connections will be planned to eliminate as many bends as practical and provide convenience in rodding. Bends greater than 45 degrees made with one fitting shall be avoided and shall be made with combinations of elbows such as 45-45 or 30-60 degrees. Provide a cleanout at every combination of elbows.

h) Cleanouts. Cleanouts provide a means for inserting cleaning rods into the underground piping system. Install a cleanout within 2 meters of a building on all sewer building connections. A manhole may be used in lieu of a cleanout. An acceptable cleanout will consist of an upturned pipe terminating at, or slightly above, final grade with a plug or cap. Preferably the cleanout pipe will be of the same diameter as the sewer pipe, but never smaller than 150 mm.

i) Grease Interceptors. Grease interceptors are used to remove grease from wastewater to prevent it from entering the sanitary sewer and septic systems. All Dining Facilities (DFACs) shall drain DFAC cooking and sink and floor drain waste to a grease interceptor prior to the sanitary sewer system. The grease interceptor shall connect to the sanitary sewer system. Sanitary wastes from the DFAC shall flow in a separate pipe to the sanitary sewer system and shall not flow into the grease interceptor.

The grease interceptor shall either be a gravity type or hydro-mechanical type. If the designer selects a gravity type, the grease interceptor shall be of reinforced cast-in-place concrete, reinforced precast concrete or equivalent capacity commercially available steel, with removable three-section, 9.5 mm checker-plate cover, and shall be installed outside the building. Concrete shall have 21MPa minimum compressive strength at 28 days. Steel grease interceptors shall be installed in a concrete pit and shall be epoxy-coated to resist corrosion as recommended by the manufacturer. For sizing of the grease interceptor, follow the guidance provided in the AED Design Guide which is based on the EPA document 625/1-80-012 Onsite Wastewater Treatment and Disposal Systems.

If the designer selects a hydro-mechanical type, the grease interceptor shall be sized and tested in accordance with Standard PDI- G101, Testing and Rating Procedure for Type I Hydro-Mechanical Grease interceptors with Appendix of Installation and Maintenance.

Drainage to grease interceptors shall be separate and distinct from other sanitary sewer lines. Wastes that do not required treatment or separation shall not be discharged into any interceptor or separator, per ICC IPC 2007 Section 1003.2 Approval.

j) Oil Water Separators. Design and install oil water separators per the AED Design Guide which is based on the ICC IPC 2007 Section 1003.4.2 Oil Separator Design.

k) Field Tests and Inspections. Prior to burying the sewer lines, field inspections and testing shall be done to ensure the lines were properly installed and free of leaks. When conducting tests and inspections the following steps shall be conducted:

(1) Check each straight run of pipeline for gross deficiencies by holding a light in a manhole; it shall show practically a full circle of light through the pipeline when viewed from the adjoining end of the line. When pressure piping is used in a non-pressure line for non-pressure use, test this piping as specified for non-pressure pipe.

(2) Test lines for leakage by either infiltration tests or exfiltration tests.

(3) Deflection testing will not be required however; field quality control shall ensure that all piping is installed in accordance with deflection requirements established by the manufacturer.

4. Septic System

a) General. When determining an appropriate septic tank location, the Contractor shall provide protection for the septic system by ensuring that vehicles, material storage and future expansion shall be kept away from the area. Signage or other prevention methods (i.e., pipe bollards) shall be used to provide this protection. The finished grade for the site shall ensure that storm water runoff shall drain away from the site to prevent ponding, inflow and infiltration. Once an appropriate site is located, the Contractor shall conduct soil investigations for the site to determine ground water levels, soil conditions and the percolation rate.

b) Septic Tank. Septic tanks are buried, watertight receptacles designed and constructed to receive and partially treat wastewater. The tank separates solids from the liquid, provides limited digestion of organic matter, stores solids, and allows the clarified liquid to discharge for further treatment and disposal. Settleable solids and partially decomposed sludge accumulate at the bottom of the tank, while scum rises to the top of the tank's liquid level. The partially clarified liquid is allowed to flow through an outlet opening position below the floating scum layer. The clarified liquid will be disposed of to the absorption field for further treatment and disposal.

Factors to be considered in the design of a septic tank include tank geometry, hydraulic loading, inlet and outlet configurations, number of compartments and temperature. If a septic tank is hydraulically overloaded, retention time may become too short and solids may not settle properly.

For Afghanistan, a baffled multi-compartment or dual chamber design shall be utilized. Refer to Attachment A for further details. The septic tank shall be designed with a length-to-width ratio of 2:1 to 3:1 and the liquid depth should be between 1.2 meters and 1.8 meters. This depth is determined by the outlet pipe invert elevation. If not specified in the contract, the septic tank shall be sized based on the ADF, an additional 100% for sludge storage capacity and peak flows (0.8*c*ADD*CF*2). The tank shall be constructed of reinforced, cast-in-place concrete, with a minimum compressive strength of 21MPa at 28 days. Wastewater influent and effluent shall enter and exit on the short sides of the tank, which will allow the wastewater longer detention and settling time. The baffled tank shall have two compartments, with the first compartment (influent entry point) having 2/3 thirds the volume capacity of the tank. The tank shall have a minimum earth backfill cover of 300 mm. Access shall be provided at the entry (influent) and exit (effluent) points of the tank by installing reinforced concrete risers, with steel access hatches, that will rise 50 mm above the finished grade. The following is an example of how to determine the volume and dimensions of the septic tank:

Example 2: Size a Septic Tank - Size a septic tank for a design population of 120 individuals.

-Assume that tank volume and dimensions are not specified in the contract documents.

*V = ADD*0.8*c*2*CF*
*=190 (liters/capita/day)*0.8*120(capita)*2(sludge retention)*1.5(capacity factor)*
= ***54,720 liters (54.72 m^3)***

Where,

ADD = Average Daily Demand (Water Flow) per Person (liters/capita/day)
0.8 = conversion of water use to sewage flow
c = design population (capita)
2 = represents an additional 100% storage for sludge and peak surges
CF= Capacity Factor from UFC 3-240-09FA Domestic Wastewater Treatment
V = Volume (cubic meters)

-Assume 1.8 meter liquid depth and a length-to-width ratio of 2:1.

A = V/1.8 meters (liquid depth) = 54.72 (m^3)/1.8 (meters) = 30.4 m^2
LW = A
2W(W) = 30.4 (m^2)*
W^2 = 30.4 (m^2)/2
W = (15.2 m^2)$^{1/2}$ = 3.90 meters (3900 mm)*
*L = 2*W = 2*3.90 meters = 7.80 meters (7800 mm*)*

Inside dimensions of tank = ***7800 mm X 3900 mm X 1800 mm (liquid depth)***

where,

A = Area
*L = Length (meters) = 2*W*
W = Width (meters)

**Always round up to the nearest 100 mm for final septic tank dimensions.*

c) Absorption Field. Absorption fields (also termed “leach fields”) are used, in conjunction with septic tank treatment, as the final treatment and disposal process for the septic system. Absorption fields normally consist of perforated distribution pipe laid in trenches or beds that are filled with rock. Refer to Attachments B or C for minimum perforation requirements. The septic tank effluent is distributed by the perforated pipe and allowed to percolate through the ground, where it is filtered and treated by naturally occurring bacteria and oxygen. <u>Maximum depth for leach field percolation pipe lines shall be one (1) meter to allow for air exchange with the surface</u>.

Once effluent is released from the septic tank, it travels by gravity through a solid PVC pipe, at a minimum 1.0% slope, to the distribution box. The distribution box is a reinforced concrete structure that distributes the septic tank effluent evenly throughout the absorption field through several 100 mm diameter perforated pipes. Distribution piping and laterals shall be placed at a depth between 650 mm to 1500 mm. Because of the desire for the effluent to be distributed evenly over the absorption trenches or beds, the perforated pipe shall have a maximum slope of 0.5% and shall be capped at the end of each pipe. Generally, distribution piping is spaced from one meter to 1.8 meters apart and is no longer than 30 meters.

Absorption trenches are a minimum 610 mm wide but can be widened to shorten the length of the trench. A bed can be as wide as needed based on the total area needed for absorption, but maybe limited in size due to available real estate, or by construction constraints. Large absorption beds are susceptible to the bed bottom being compacted during excavation and pipe installation. Compaction of the bed bottom will degrade percolation and may lead to failure of the absorption field. The absorption field has three (3) zones:

(1) The first zone is the absorption zone, which is the layer of in-situ material that filters and treats the effluent. This zone is determined to be suitable material for wastewater treatment based on the percolation test results, with a minimum thickness of 600 mm. Below the absorption zone, the material is considered unsuitable soil or bed rock or the seasonal water table is too high. If percolation tests determine that there isn’t a minimum 600 mm of suitable soil, the Contractor can remove the unsuitable soil to the desired depth and replace it with material determined to be suitable; however, the Contractor must get approval from the COR before attempting this.

(2) The second zone is the drainage zone, which is a 300 mm thick layer of rock fill, where the distribution pipe network lies. The bottom of this zone is filled with a minimum 150 mm of 19 mm to 38 mm diameter rock. The perforated distribution pipe is laid on top of the rock. A minimum of 50 mm of rock is placed carefully over the pipe network, and then a semipermeable membrane (geotextile fabric) is placed over the rock to prevent fine-grained backfill from clogging it.

(3) The final zone is the backfill zone. This is the upper most part of the absorption field, where backfill material is placed and is a minimum 500 mm thick. The backfill material protects the lower lying zones from storm water infiltration and freezing. The Contractor shall leave a mound of backfill material above the desired finished grade to allow for settlement.

Table 2 lists percolation rates and the corresponding sizing factor (m^2/liters/day). The sizing factors are used, in conjunction with average daily flow (ADF), to determine the size of the absorption field. The following is an example of how to calculate the absorption field size for trenches and beds:

Example 3: Size of Absorption Field - Size an absorption field (trench type) for a facility with an average daily flow of 27,360 liters/day and a percolation rate of 50 minutes.

*A=Average Daily Flow*Water Absorption of Soil*
*=27,360 liters/day*0.054 m^2/liters/day=* ***1,477.44 m^2***

where,

A = Area footprint needed for the absorption field (m^2)
Average Daily Flow (liters/day)
Water Absorption of Soil = By looking below, at Table 2, a percolation rate of 50 minutes falls in the 46 to 60 row and the correlating sizing factor is determined to be 0.054 m^2/liters/day.

Dimensions for trenches:

-Assume a 0.9144 meter wide trench bottom.
-Assume maximum trench length to be 30 meters.

$N_T = A/(T_w*T_L)$ = 1,477.44 m^2/(0.9144 m*30 m) = 53.86 say:* ***54 Trenches (0.9144 meters X 30 meters)*

where,

N_T = Number of Trenches
T_w = Trench width (meters)
T_L = Trench Length (meters)

**Note: Trench bottom area can be reduced by 20 percent, if 305 mm of rock is placed below the distribution pipe. The area can be reduced by 34 percent for 457 mm of rock being placed below the pipe and by 40% for the maximum rock depth of 610 mm. Keep in mind that the additional rock added below the distribution pipe adds additional thickness required for the drainage zone. For example, where normally 150 mm of rock is placed below the pipe for a total 300 mm thickness for the drainage zone. Placing 305 mm of rock is placed below the pipe increase total thickness for the drainage zone to 455 mm of rock, (305 mm below the pipe; 100 mm around the pipe; and 50 mm above the pipe).*

Dimensions for bed: ***(Absorption Beds are not recommended for large systems due to the difficulty of constructing the bed bottom without compacting or disturbing it, and relative inability to function over terrain at various elevations.)***

Absorption Bed Design Population of Twelve:

*Average Daily Flow = 190 lpdc*0.8*12*1.5 = 2,736 lpd*
*Area Required = 2,736 lpd * 0.054 m^2/liter/day = 147.74 m^2*
Absorption Bed Dimensions = $A^{1/2} = (147.74\ m^2)^{1/2}$ = 12.15 meters, say: 13 meters per side
Absorption Bed Dimensions = ***13 meters X 13 meters***
Refer to Attachments B and C for further design details of absorption fields.

Table 2. Soil Treatment Areas in Square Meters.

Percolation Rate, Minutes for Water to Drop 25 mm	Water Absorption of Soil (m^2/liters/day)
Faster than 0.1	Soil too coarse for sewage treatment
0.1 to 5	0.020
6 to 15	0.031
16 to 30	0.041
31 to 45	0.049
46 to 60	0.054
Slower than 60	Soil too fine for sewage treatment

d) Pressure Dosing of Leach Fields. Pressure dosing tanks are required per Unified Facilities Criteria (UFC) 3-240-09a, Domestic Wastewater Treatment, dated January 2004, Chapter 6, Section 6-2. Dosing tank (also known as a dosed system or Pressure Distribution System, PDS) is recommended for treatment systems with over 10 or more population equivalents and is preferred where backup power is available in larger systems in lieu of using more than one effluent gravity distribution box. No more than seven leach field laterals shall be connected to one effluent gravity distribution box.

PDS provides simultaneous distribution of sewage effluent over the entire infiltration surface. This enhances soil treatment by maintaining unsaturated soil conditions and inhibits failure by clogging. Dosing helps to maintain aerobic conditions in the soil, but this benefit would be negated if the infiltration surface is installed greater than 3 feet (1meter) below the ground surface. It improves the performance and increases the lifespan of the tile field. Pumping which is usually associated with dosing tanks provides flexibility in locating disposal fields where soil conditions are most suitable. Components of a PDS system are shown in Figure 2.

<u>PDS SYSTEM COMPONENTS</u>

1) Septic Tank or Holding Tank -Domestic sewage is transported to a septic tank. Septic tanks are to be comprised of two chambers and should be as per Paragraph 4(b) above. Septic tanks should be secured against hydraulic uplift in areas that have the potential for high groundwater levels. They should be adequately sealed to prevent the infiltration of groundwater to the sewage disposal systems or the exfiltration of sewage to the surrounding environment.

2) Pump - Pump should not be able to pump solids as large as the lateral orifice diameter with the impellers being of cast iron or other corrosion resistant material. Pumps should be serviceable from ground level without the need to enter the pump chamber or cut the pipe. The pump should have a quick release fitting to allow for easy removal and installation of the pump for maintenance purposes.

3) Pump Screen - All pressure distribution pumps should be surrounded by a plastic mesh screen with 3.175mm (0.125 in) diameter holes. The screen should have a sufficient surface area so that the velocity of the sewage effluent passing through the screen does not allow the screen to become plugged with solids.

4) Pump Chamber - A pump chamber separate from a septic tank is required for PDS. The pump chamber shall be sized to allow a set volume of sewage.

5) Control Panel - All pumps shall be connected to a control panel. The panel should have an alarm in case the pump fails to operate properly.

6) Transport Pipe - The transport pipe is the pipe that connects the sewage effluent pump to the manifold pipe. The diameter of this pipe is determined by friction losses caused by the flow of sewage effluent through the pipe and by the desire to have a cleansing velocity where possible of 0.6 m/s (2 ft/s) passing through the pipe during operation. There should be a flexible connection between the pump and the transport pipe to allow for the possible settlement of the pump chamber or the septic tank after installation. The PVC pipe shall conform to ASTM D2241 PVC Pressure-Rated Pipe (SDR series) and have a maximum SDR of 35.

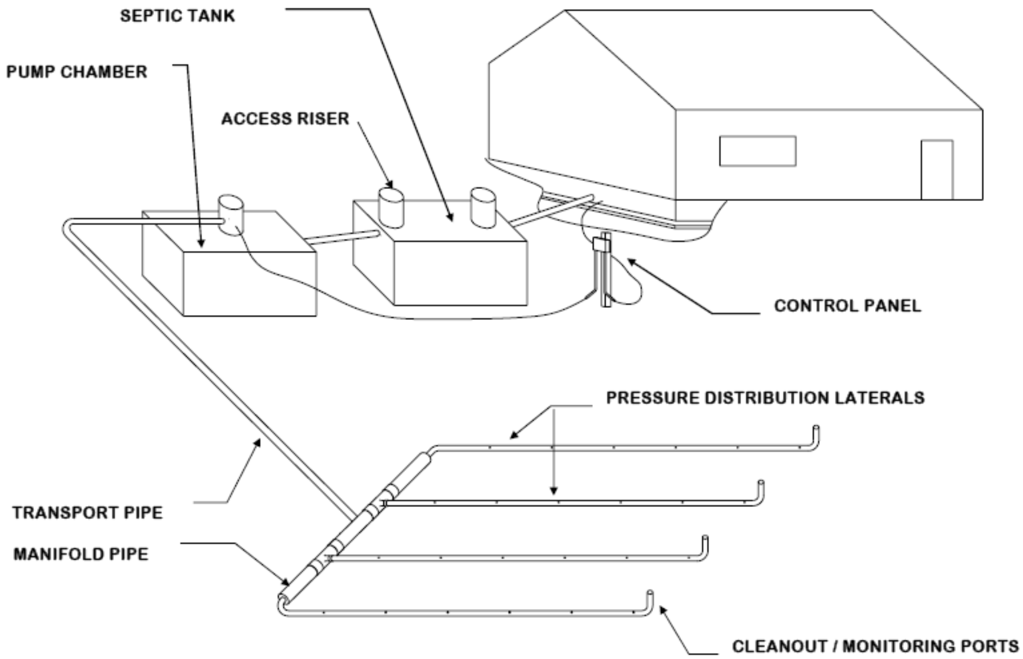
7) Manifold Pipe - The manifold pipe is located between the transport pipe and the laterals in the leach field. This pipe is sized so that there is no more than a 15% variation in the rates of discharge between the first and last orifices in the network. The PVC pipe shall conform to ASTM D2241 PVC Pressure-Rated Pipe (SDR series) and have a maximum SDR of 35.

8) Lateral Pipe - Laterals in the PDS are used to distribute the sewage effluent to the soil. Their length configuration and number are determined by soil condition, percolation rate and leach field geometry. The diameter of a lateral should be the smallest diameter that achieves nearly uniform pressure along the entire length of the lateral. The PVC pipe shall conform to ASTM D2241 PVC Pressure-Rated Pipe (SDR series) and have a maximum SDR of 35.

9) Orifice - Orifice shields are placed over the orifices (holes) in the laterals to prevent sewage effluent from being forced under pressure to the surface of the drain field when the orifices are in the 12 o'clock position (the crown of the lateral). They also prevent the orifices from becoming blocked by drain rock in the leach field. The shields will be asphalt building paper; minimum of 0.73 kg/m^2 or geotextile material.

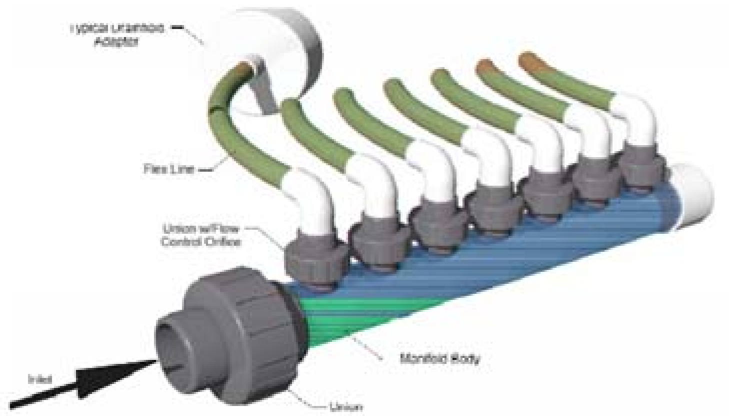
   Clean outs - Clean outs shall be placed at the ends of the laterals, a minimum of 0.5 meter above ground. They should have treaded caps at their ends to allow for inspection and cleaning of the laterals. In cold climates they should be insulated to prevent the laterals from freezing.

Figure 3. Pressure Distribution System (Source: Washington State Department of Health.)



Many project septic designs rely on gravity flow from the septic tank outlet to a distribution box (not shown in Figure 2) which serves the function of the manifold pipe. The gravity dosing concept requires a very level leach field and favorable soil characteristics for absorption (sandy gravelly soil) in order to provide long term treatment.

Pressure dosing applies the effluent over the entire absorption area in such a way that the hydraulic loading is more evenly distributed thus promoting better soil treatment by maintaining vertical unsaturated flow and reducing the degree of clogging in finer textured soils. It can be applied to sloping sites but will require an additional features (in addition to other PDS components) shown in Figures 2 and 3. These figures show manifold and automatic dosing valve that operates using hydraulic pressure to change the distribution line that receives the effluent each time the effluent pump cycles in the dosing tank.



3.4.4. Distributing valves can be used as a means for distributing effluent to multiple drainfield laterals or zones. The water pressures in the transport line activate these valves. Each time the pump is turned on, the valve rotates to dose the next drainfield. Figure 3 shows a distributing valve assembly. Distributing valves must be designed with the following features:

3.4.4.1. Unions to allow easy removal of the valve.

Figure 4. Typical Pressurized Flow Distribution Device

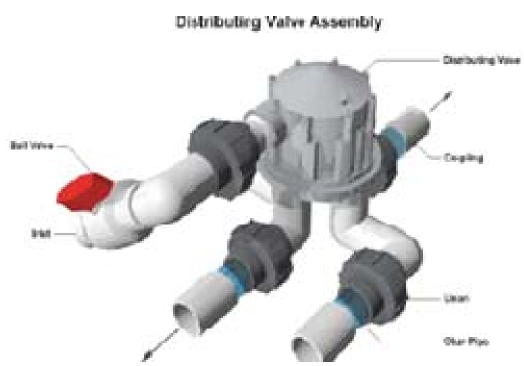
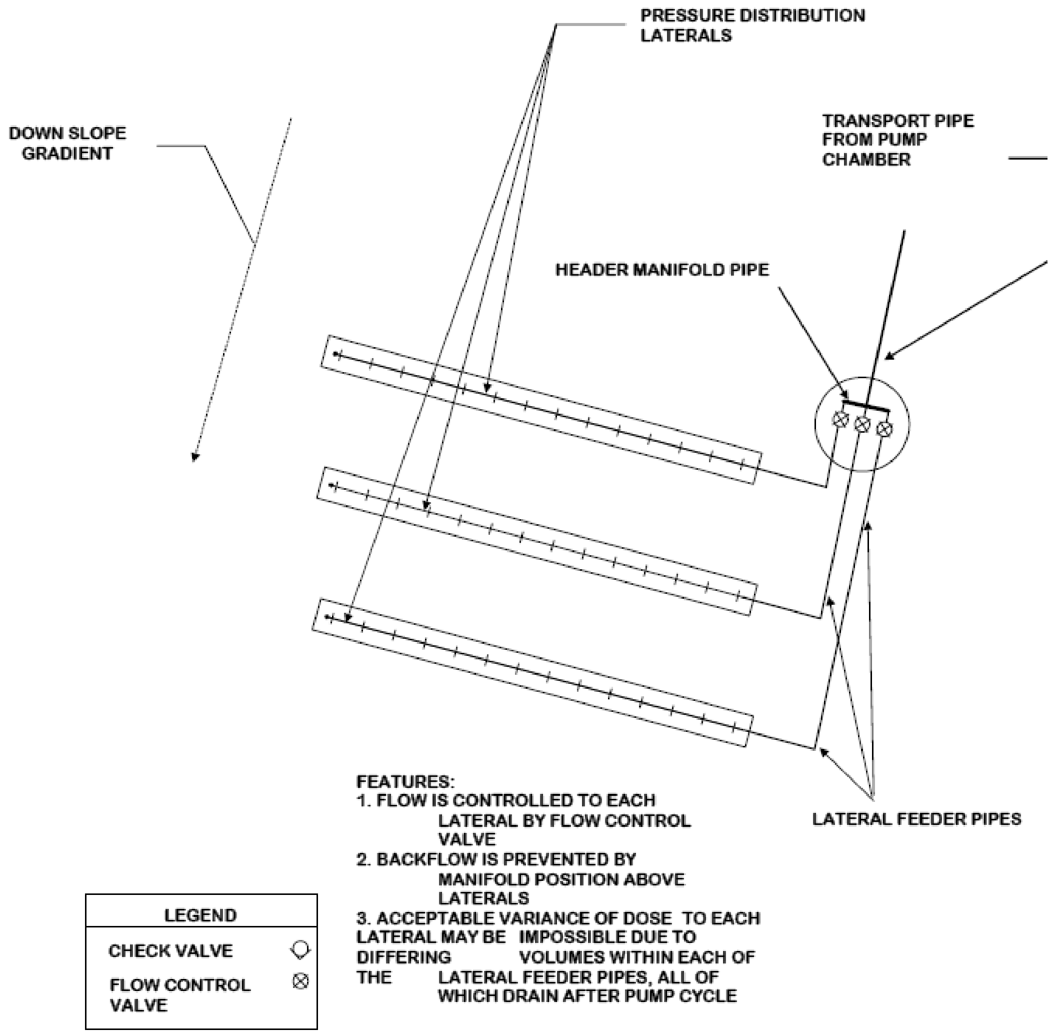


Figure 5. Typical Flow Distribution Valve

Pressure distribution would permit the use of sloping sites as seen in the following schematic:



While there are additional operational costs associated (valve cleaning and occasional replacement), the advantages for sloping sites include:

- Lower cost of leveling the site
- Avoiding the destruction of the natural soil horizon and the microbes that promote the biological treatment of the wastewater effluent
- Ground water recharge of the aquifer where water is being drawn
- Use previously unusable sites where the alternative (holding tanks or package WWTP) are more costly
- More frequent smaller doses assure unsaturated flow through the soil and reduce the potential for clogging and destroying the treatment capacity of the site.

b) PDS SYSTEM DESIGN EXAMPLE STEPS

The following design example is based on information prepared by Professor James Converse, dated 2000 (reference 3). The designer is recommended to review all reference material listed at the end of 4 (d) as well as apply his/her engineering judgment. Units are shown as imperial, however can be easily converted to “soft” metric.

Design is a two part process:

PART 1 consists of sizing the distribution network which distributes the effluent in the aggregate and consists of the laterals, perforations (orifice) and manifold.

PART 2 Consists of sizing the force main, pressurization unit and the doze chamber and selecting controls.

Within each part, there are several steps associated with design.

Example 4: Size a pressure distribution network with the following given information: (units are in imperial units)

Absorption area – 113ft long by 4 ft wide
Force Main – 125 ft long
Elevation Difference – 9 ft
Number of elbows - 3

Part 1. Design of the distribution network.

Step 1. **Configuration of the network.**
This is a narrow absorption unit on a sloping site.

Step 2. **Determine the lateral length.**
Use a center feed (meaning the manifold for the line from the dosing tank is in the center of the absorption field), the lateral length is:

Lateral length = (B/2) – 0.5 ft

Where B = absorption length.
(113/2) – 0.5 = 56 ft

Note: Recommend to have the manifold down the center of the absorption field and then have laterals going out from the center.

Step 3. **Determine the perforation spacing and size**.
Each perforation will cover 6ft^2/orifice (note: this value is a based on rule of thumb for 30-36” orifice spacing). This value is the area/orifice parameter. This means, each orifice will discharge an equivalent effluent on a 2ft x 3 ft area.

Will use two laterals on each side of the center feed.

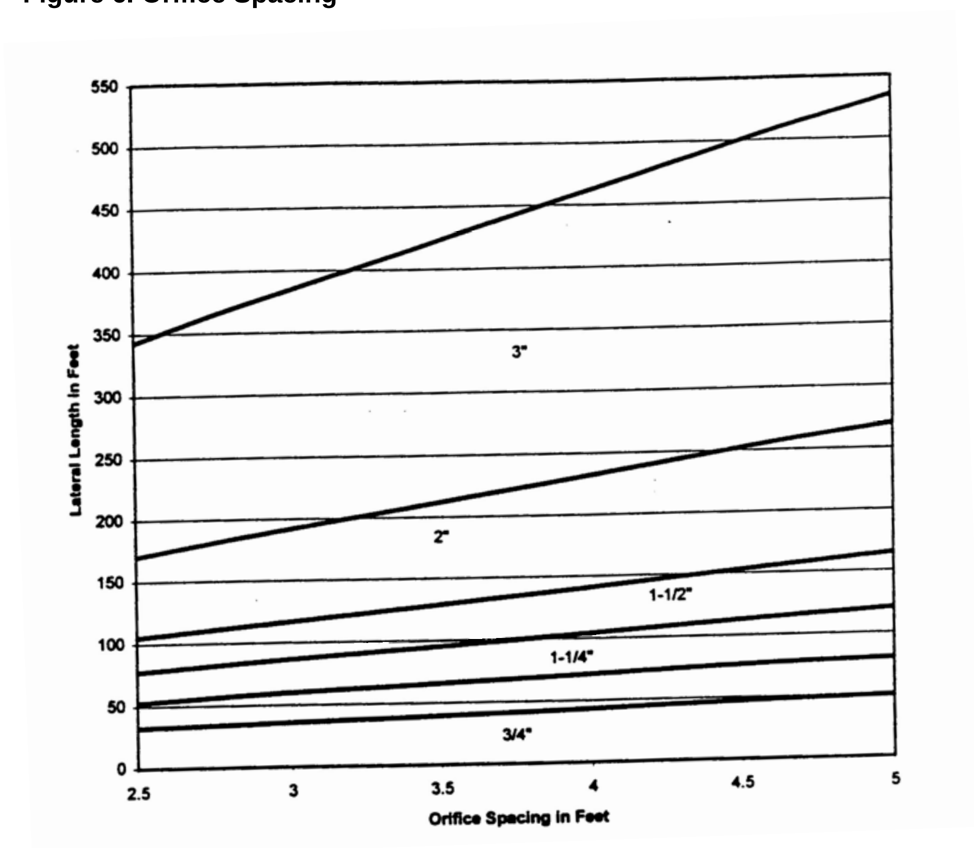
Spacing = (area/orifice x number of laterals / (absorption area width).

Spacing = (6ft^2 x 2/(4ft)) = 12/4 = 3 ft

Size – use either a 3/16" or ¼" diameter. Recommend using a 3/16" diameter orifice. Whichever diameter is used, it requires placement of an effluent filter (screen, such as wire mesh at the outlet pipe) in the septic tank to eliminate carryover of large particles. Designer can choose to use larger or smaller diameter, (refer to reference 3 for other sizes).

Step 4. **Determine the lateral diameter.**
Refer to the following figure, "Orifice Spacing in Feet" for a 3/16" diameter orifice.

Figure 6. Orifice Spacing



Using lateral length of 56 feet and orifice spacing of 3, the point of intersect is between the 1-1/4" and 1-1/2" lines. Round up to the next value, therefore it will be 1-1/2".

In this design example, the lateral diameter is 1.5"

Step 5. **Determine number of perforations per lateral and number of perforations.**
Using 3 feet spacing in 56 feet length yields:

N = (p/x) + 0.5 = (56/3) + 0.5 = 19 perforations/lateral

Number of perforations = 4 laterals x 19 perforations/lateral = 76.

Check: Maximum of 6 ft^2/perforation
Number of perforations = (113 ft x 4 ft)/6ft^2 = 75 , so OK.

Note: Perforation is the same as orifice. The 4 laterals correspond to two laterals on each side of the center manifold.

Step 6. **Determine lateral discharge rate (LDR).**
Using network pressure (distal) pressure of 3.5 ft and 3/16" diameter perforations, using the table below, gives a discharge rate of 0.78 gallons per minute (gpm) regardless of the number of laterals.

Table A -1 Orifice Diameter

	Orifice diameter (in.)				
Pressure	1/8	3/16	1/4	5/16	3/8
(ft)	(gpm)				
2.5	0.29	0.66	1.17	1.82	2.62
3.0	0.32	0.72	1.28	1.00	2.87
3.5	0.34	0.78	1.38	2.15	3.10
4.0	0.37	0.83	1.47	2.30	3.32
4.5	0.39	0.88	1.56	2.44	3.52
5.0	0.41	0.93	1.65	2.57	3.71
5.5	0.43	0.97	1.73	2.70	3.89
6.0	0.45	1.02	1.80	2.82	4.06
6.5	0.47	1.06	1.88	2.94	4.23
7.0	0.49	1.10	1.95	3.05	4.39
7.5	0.50	1.14	2.02	3.15	4.54
8.0	0.52	1.17	2.08	3.26	4.69
8.5	0.54	1.21	2.15	3.36	4.83
9.0	0.55	1.24	2.21	3.45	4.97
9.5	0.57	1.28	2.27	3.55	5.11
10.0	0.58	1.31	2.33	3.64	5.24

Values were calculated as: gpm = $11.79 \times d^2 \times h^{1/2}$ where d = orifice dia. in inches, h = head feet.

Note: The value 3.5 ft for distal pressure corresponds with 3/16" diameter orifice.

LDR = 0.78 gpm/perforation x 19 perforations = 14.8 gpm

Step 7. **Determine the number of laterals.**
This is already completed in steps 3 and 4.

Total of four laterals, two on each side of the manifold. The spacing between laterals to be at 2 ft apart.

Step 8. **Calculate the manifold size.**
This is the same size as the force main, the line from the dosing tanking out to the absorption field. Since the diameter of the laterals are 1.5 "(see step 4), the force main and the manifold should be at least half size larger, say 2" diameter.

Step 9. **Determine network discharge rate (NDR).**
NDR = 4 laterals x 14.8 gpm/lateral = 59.2 gpm. Round up to 60 gpm.

This is the value that will be utilized in Part 2 when sizing for a pump.

Step 10. **Provide for flushing of laterals.**
Provide cleanouts at the end of each lateral line as well as at the end of the manifold. This will allow for ease of maintenance.

Part 2. Design of Force Main, Pressurization Unit, Dose Chamber and Controls.

Step 1. Total Dynamic Head (TDH)

The total dynamic head is the sum of the following:

System head + Elevation head + Head Loss = TDH

System Head = 1.3 x distal head (ft)

SH = 1.3 (3.5) = 4.55 ft. (Note: distal head of 3.5 comes from Step 6 Part 1. This value is a rule of thumb).

Elevation Head = (pump shut off to network elevation) this is also the static head. Will depend on the design invert elevations.
Head Loss due to friction = This is the sum of losses due to fittings and pipe run.

For fittings and friction loss, use the two tables below.
For 2" diameter fittings the friction loss for the 90 degree elbow is 9. Since we have three elbows (in the problem statement) the total head loss due friction from fittings is 27ft.

Below is the friction loss table to use for our problem. In our example, the flow rate is 60 gpm and the nominal pipe size of the manifold and the force main is 2-inches in diameter. Therefore, the friction loss per 100 feet of pipe is 7. This value will depend on the size and type of pipe used in design. Refer to a standard fluids/hydraulics references for values other than given below.

Table A-2. Friction loss in plastic pipe.

Flow	Nominal Pipe Size						
(gpm)	3/4	1	1-1/4	1-1/2	2	3	4
			(Feet/100 ft of pipe)				
2							
3	3.24						
4	5.52						
5	8.34						
6	11.68	2.88		Velocities in this area are below 2 fps.			
7	15.53	3.83					
8	19.89	4.91					
9	24.73	6.10					
10	30.05	7.41	2.50				
11	35.84	8.84	2.99				
12	42.10	10.39	3.51				
13	48.82	12.04	4.07				
14	56.00	13.81	4.66	1.92			
15	63.63	15.69	5.30	2.18			
16	71.69	17.68	5.97	2.46			
17	80.20	19.78	6.68	2.75			
18		21.99	7.42	3.06			
19		24.30	8.21	3.38			
20		26.72	9.02	3.72			
25		40.38	13.63	5.62	1.39		
30		56.57	19.10	7.87	1.94		
35			25.41	10.46	2.58		
40			32.53	13.40	3.30		
45			40.45	16.66	4.11		
50			49.15	20.24	4.99		
60				28.36	7.00	0.97	
70				37.72	9.31	1.29	
80	Velocities in theses areas				11.91	1.66	
90	exceed 10 fps, which is too great				14.81	2.06	
100	for various flows and pipe diameters				18.00	2.50	0.62
125					27.20	3.78	0.93
150						5.30	1.31
175						7.05	1.74

Note: Table is based on - Hazen-Williams formula: $h = 0.002082L \times (100/C)^{1.85} \times (gpm^{1.85}/d^{4.8655})$ where: h = feet of head, L = length in feet, C= Friction factor from Hazen-Williams (145 for plastic pipe), gpm = gallons per minute, d= nominal pipe size.

Table A-3. Friction losses through plastic fittings in terms of equivalent lengths of pipe. (Sump and Sewage Pump Manufacturers, 1998)

	---------------Nominal size fitting and pipe ------------------------					
Type of Fitting	1-1/4	1-1/2	2	2-1/2	3	4
90° STD Elbow	7.0	8.0	9.0	10.0	12.0	14.0
45° Elbow	3.0	3.0	4.0	4.0	6.0	8.0
STD. Tee (Diversion)	7.0	9.0	11.0	14.0	17.0	22.0
Check Valve	11.0	13.0	17.0	21.0	26.0	33.0
Coupling/ Quick Disconnect	1.0	1.0	2.0	3.0	4.0	5.0
Gate Valve	0.9	1.1	1.4	1.7	2.0	2.3

The total head loss due to friction is as follows:
7 (125 ft + 27 ft)/100 ft = 10.6 ft
Total Dynamic Head (TDH) = 4.5ft + 9 ft + 10.6 ft = 24.1 ft

Step 2. **Pump Summary.**

Pump must discharge 60 gpm against a head of 24.1 feet with 2" force main.

These are the calculated flow and head values. The actual flow and head will be determined by the pump selection. A system performance curve plotted against the pump performance curve will give a better estimate of the flow rate and total dynamic head the system will operate under.

Step 3. **Determine the dose volume.**

For dose volume, use 5 times the lateral void volume. Void volume is selected from the table listed below.

Table A-4. Void volume for various diameter pipes.

Nominal Pipe Size (In.)	Void Volume (gal./ft)
3/4	0.023
1	0.041
1-1/4	0.064
1-1/2	0.092
2	0.163
3	0.367
4	0.650
6	1.469

For 1.5" lateral line, the void volume is 0.092 gal/ft.

For 2" force main line, the void volume is 0.163 gal/ft

Dose rate in laterals = 5 x 56ft x 4 laterals x 0.092 gal/ft
= 103 gal/dose

Dose rate in main line = 125 feet x 0.163 gal/ft
= 20.4 gal/dose

Dose rate total = 103 gal + 20 gal = 123 gallons/dose
There will be 5 doses at 123 gallons per dose throughout a 24 hour period.

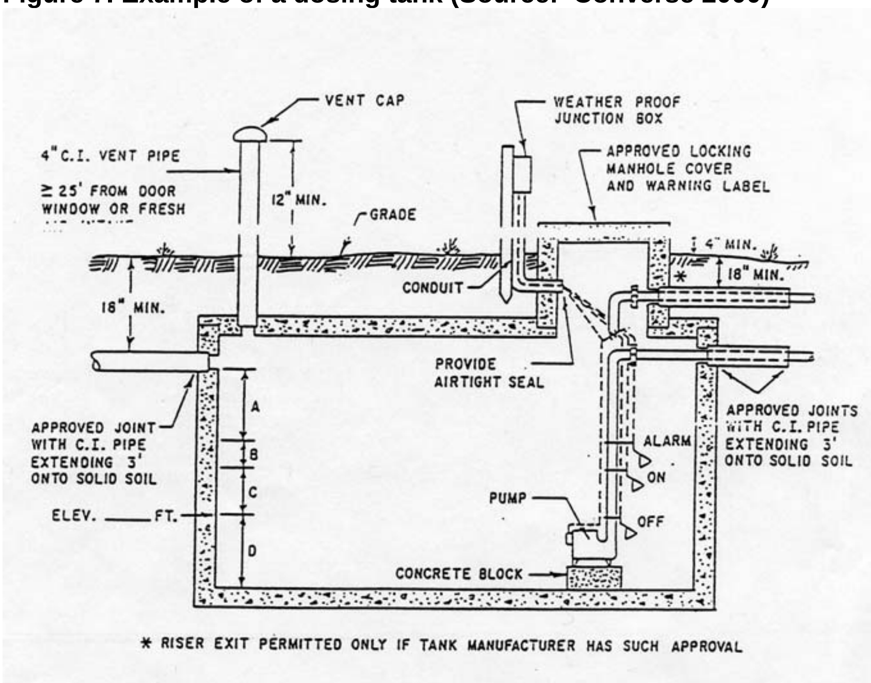
Step 4. **Size the dose chamber.**

Based on the dose volume, storage volume and room for a block beneath the pump and control space, 500- 700 gallon chamber will suffice.

Step 5. **Select controls and alarms.**

This example is for demand dosing. The controls will include on-off float and alarm float. The designer will set the on float based on the volume of the dose (123 gallons). The pump will dose five times through out the day.

Figure 7. Example of a dosing tank (Source: Converse 2000)



e) Leaching Chambers. Leaching chambers are used where absorption fields or pressure dosing of leach fields are specified. They serve as an alternative to using perforated pipe and gravel.

Leaching chambers are constructed of molded plastic (polypropylene or high density polyethylene); and are dome shaped with a solid top, an open bottom and louvered sidewalls. The chambers are furnished in 5 to 6-foot plus sections in widths of up to 34 inches that are field connected to one another to make each row. A row is finished by installing an end plate/cap on each end of the row. Each row is then connected to one another through interconnecting piping.

Leaching chambers may also be constructed with mortared stone masonry or reinforced concrete walls, and a reinforced concrete cover.

These systems are particularly ideal where space is limited. Absorption field size may be reduced by 40 percent using leaching chambers. In addition, these systems can take traffic loading (H-10, H-20, etc.) when the correct model or design is used, and properly installed.

Chamber rows may either be installed individually in trenches or in a bed. Row lengths shall not exceed 30 meters.

Trenches shall be excavated to the width and depth required. The bottom of each row shall be level and flat. Scarify the bottom and sidewall surfaces to remove any smearing than may have occurred during excavation. See Figure 8 and Appendix B for an example of a trench installation.

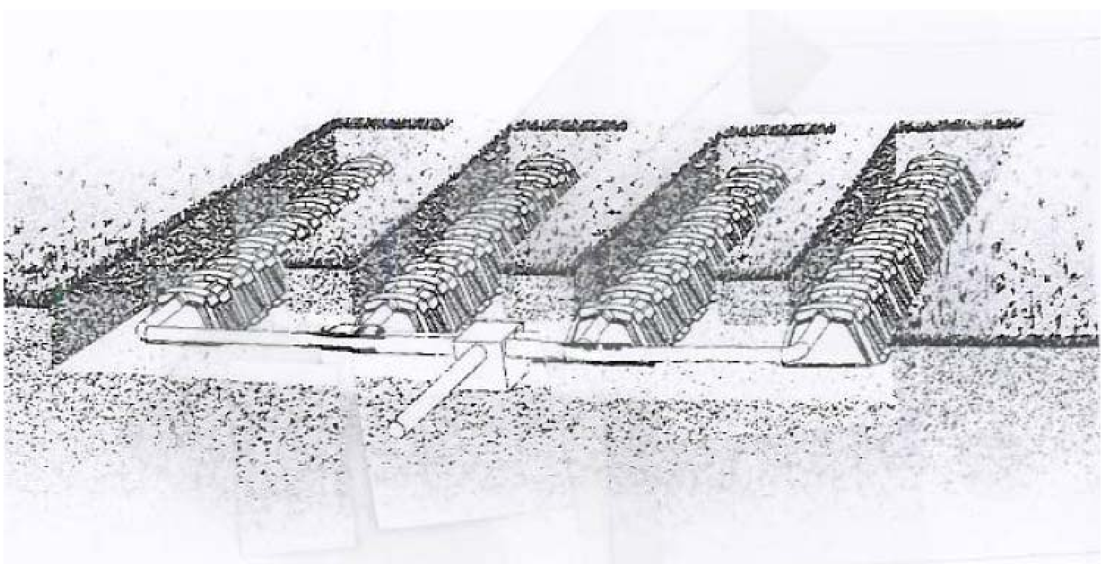


Figure 8. Example of Leaching Chamber Trench Installation

For bed installation, excavate area and level installation area. Scarify surface to remove any smearing that may have occurred during excavation. Place chamber rows 6 inches apart. See Figure 9 and Appendix B for an example of bed installation.

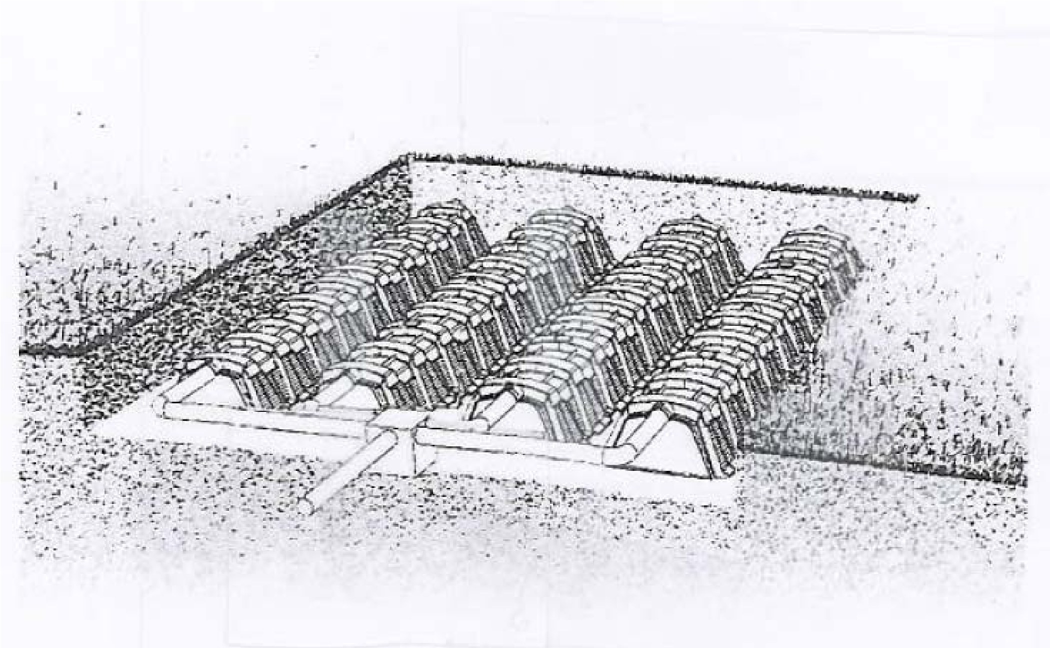


Figure 9. Example of Leaching Chamber Bed Installation

Install an inspection port at the end of each row. These ports allow inspection of the leaching chamber and provide a visual marker to lineate the extent of the absorption field. Protect end cap of inspection ports using a valve box cover or other means to protect them from traffic loads.

f) Seepage Pits. Seepage pits or dry wells are deep excavations used for subsurface disposal of pretreated wastewater (influent processed through a septic tank). Wastewater enters the chamber where it is stored until it seeps out through the chamber wall and infiltrates the sidewall of the excavation. Seepage pits are generally discouraged, in favor of trench or bed systems. Seepage pits have been shown to be an acceptable method of disposal for very small wastewater flows. Seepage pits are used where land area is too limited for trench or bed systems ; and either the groundwater level is deep at all times, or the upper .9 to 1.2 m of the soil profile is underlain by more permeable unsaturated soil material of great depth.

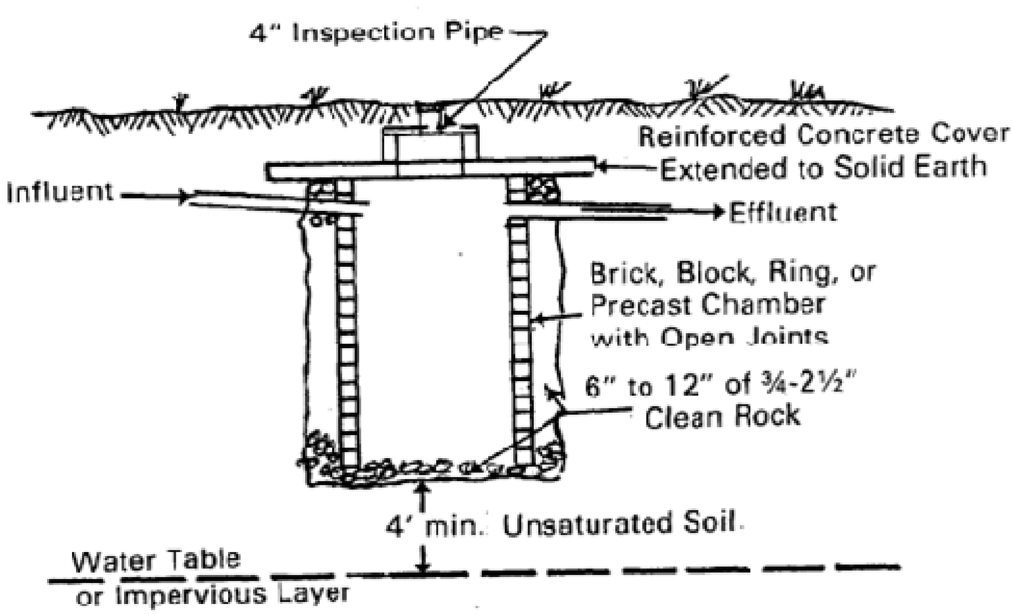


Figure 10. Seepage Pit Cross Section

The suggested site criteria are similar to those for trench and bed systems except that percolation rates slower than 12 min/cm are generally excluded. Maintaining sufficient separation between the bottom of the seepage pit and the high water table is a particularly important consideration for protection of groundwater quality. Seepage pits bottoms shall be a minimum of 1.22 m (4 ft.) above the seasonally high water table as shown in Figure 10. In Afghanistan, marked and rapid swings in water table levels due to seasonal rains and flooding make prediction of appropriate seepage pit depth extremely difficult

Seepage pit sizing and the infiltrative surface of the pit shall be in accordance with Chapter 7 Disposal Methods, paragraph 7.2.3.3 Design, of EPA 625/1-80-012. Since the dominant infiltration surface of a seepage pit is the sidewall, the depth and diameter of the pit is determined from the percolation rate and thickness of each soil layer exposed by the excavation. A weighted average of the percolation test results is used for design. Seepage pits will require a labor intensive and more detailed soil percolation rate study for each of the distinct soil layers encountered in the excavation to its ultimate depth. Infiltration rates presented earlier in Table 2 are used to compute the necessary sidewall area. Table 7-6 can be used to determine the necessary seepage pit sidewall area for various effective depths <u>below the seepage pit inlet</u>.

TABLE 7-6

SIDEWALL AREAS OF CIRCULAR SEEPAGE PITS (ft^2)[a]

Seepage[b] Pit Diameter ft	Thickness of Effective Layers Below Inlet (ft)									
	1	2	3	4	5	6	7	8	9	10
1	3.1	6	9	13	16	19	22	25	28	31
2	6.3	13	19	25	31	38	44	50	57	63
3	9.4	19	28	38	47	57	66	75	85	94
4	12.6	25	38	50	63	75	88	101	113	126
5	15.7	31	47	63	79	94	110	126	141	157
6	18.8	38	57	75	94	113	132	151	170	188
7	22.0	44	66	88	110	132	154	176	198	220
8	25.1	50	75	101	126	151	176	201	226	251
9	28.3	57	85	113	141	170	198	226	254	283
10	31.4	63	94	126	157	188	220	251	283	314
11	34.6	69	104	138	173	207	242	276	311	346
12	37.7	75	113	151	188	226	264	302	339	377

[a] Areas for greater depths can be found by adding columns. For example, the area of a 5 ft diameter pit, 15 ft deep is equal to 157 + 79, or 236 ft.

[b] Diameter of excavation.

Example 5: Size of a Seep Pit – Based on a population of ten (10) and a percolation rate of 6 min/cm.

*ADF = 10 capita*190 liters/capita/day*0.8*1.5 capacity factor = 2,280 liters/day*
*Area Required = 2,280 lpd * 0.031 m^2/liter/day = 70.68 m^2 = 761 ft^2*
Seep Pit Size equals approximately 2 pits (A= 754 ft^2) 3.66 m in diameter and 3 m deep below the inlet pipe.

The seepage pits should be separated by at least 2 diameters spacing between the sides of the pits.

This example shows that seepage pits when properly sized are not viable for larger wastewater flows.

5. Design Submittal Information

a) Gravity Sewer. A preliminary sewer flow depths can be calculated assuming normal flow regime. This is a simplification of actual flow regime because there will be energy losses at the entrance of other side laterals and manhole energy losses that will make the water profile will not be uniform. If sewers are at flat grades for long pipe runs, these losses shall be considered because they will over the length of the project become a significant design factor. For short pipe runs and normal slopes (greater than minimum slope values) the uniform flow assumption can be used. Design documentation for gravity sewer design using the uniform flow assumption shall include the following:

- Sewer pipe line number
- Pipe diameter
- Pipe length
- Pipe slope
- Incremental wastewater inflow
- Total cumulative wastewater flow
- Pipe roughness (n value)
- Full flow area of pipe
- Hydraulic radius and full flow velocity
- Calculation of ratio of actual design flow to full flow in the pipe
- Calculation of the actual design flow velocity for sewer pipe length

Flow velocities shall be compared to design standards and profiles or pipe size adjusted accordingly. An example analysis is shown in Appendix A.

c) **Pressure Distribution.** A series of design examples follows this section that illustrates the design submittal information required for pressure distribution systems (PDS) for effluent disposal in leach fields.

d) **Septic Tanks and Leach Fields.** Required design calculations to be submitted fro projects are shown in Examples 1 through 3 previously described.

6. As-Builts

Upon completion of installing the sanitary sewer and septic systems, the Contractor shall submit editable CAD format As-Built drawings. The drawings shall show the final product as it was constructed in the field, with the exact dimensions, locations, materials used and any changes made to the original design. Refer to Contract Sections 01335 and 01780A of the specific project for additional details.

Reference – Dosing tank:

1) Unified Facility Criteria 3-240-9a Domestic Water Treatment, January 2004
http://www.wbdg.org/ccb/DOD/UFC/ufc_3_240_09fa.pdf
2) Environmental Protection Agency (EPA) 625 R-00 008, February 2002
http://www.epa.gov/safewater/uic/class5/pdf/techguide_uic-class5_2002_onsite_wwt_sys_man.pdf
3) Pressure Distribution Network Design by James C. Converse January 2000
http://www.iowadnr.com/water/wastewater/files/dg_app_b_pdd.pdf
4) Washington State Department of Health, publication # 337-022, July 2007
http://www.doh.wa.gov/ehp/ts/WW/pres-dist-rsg-7-1-2007.pdf

Appendix A- Example Gravity Sewer Calculation

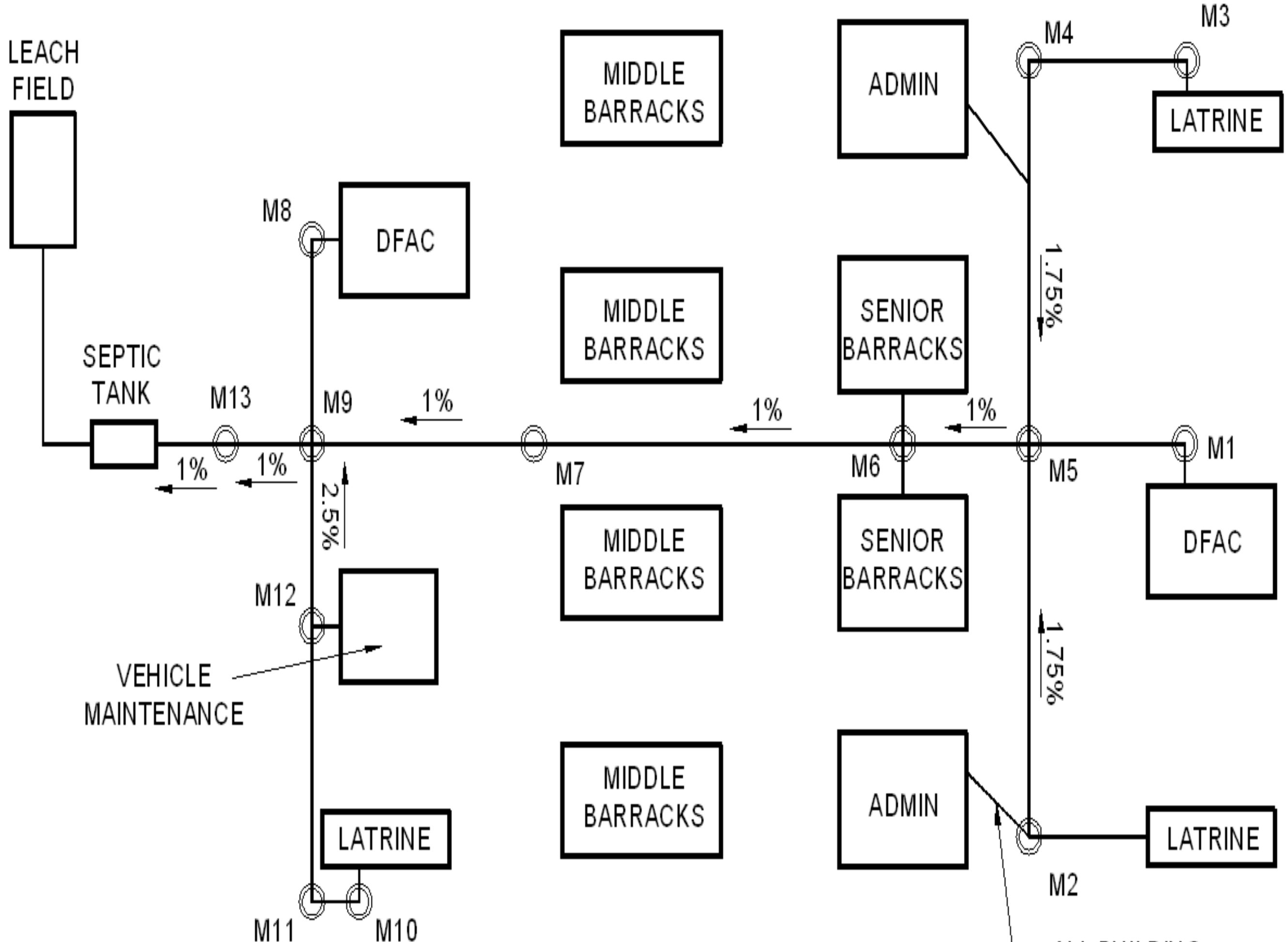
Example Gravity Sewer Calculation

The objectives of the analysis for the system include:

1. Verifying the minimum diameter (without flowing more than 80% full) for the collection piping is large enough to convey flow throughout the system such that the technical criterion for minimum velocity is achieved. This is to be done with the necessary slope greater than the minimum grade that reduces settling of suspended solids and provides economical construction depth, without the need for excessive number of lift stations.
2. Verifying whether sewer drop structures are needed to achieve minimum cover at acceptable sewer grade on long pipe runs.
3. Verifying if there is a need for lift stations over long pipe runs.
4. Verifying the heights assumed for manholes are acceptable; generally less than 5 meters depth.

The project site plan is used to obtain pipe information for the calculations. See the attached site plan example. The analysis can be set up in an electronic spreadsheet format.

1. Organize a numbering system from upstream to downstream
   a. Enter hydraulic information for each pipe run
      i. Pipe diameter
      ii. Pipe length
      iii. Pipe slope
      iv. Pipe roughness properties – Manning's n value
   b. Determine the flow added at each intersection that represents the downstream pipe discharge.
2. Add flows in the downstream direction and enter into a table containing the pipe information
3. Calculate the full flow capacity and velocity of the pipe runs
   a. Use Manning's equation to calculate flow velocity
   b. Calculate flow base on total flow area and velocity
4. Use a nomograph or flow properties chart (see attached) to calculate the proportional flow and velocity for the actual flow rate in each pipe based on the design flow
5. Check if the flow depth is less than 80% and the flow velocity exceeds the minimum required.
6. If not, try again
7. Tabulate the design information on the construction drawing sewer site plan in a pipe schedule



THIS EXAMPLE IS FOR AN INSTALLATION OF 650 PERSONNEL(PN), FOR INSTALLATIONS WITH LESS THAN 450 PN USE THE GUIDANCE FOUND WITHIN PARAGRAPH 3.d.

EXAMPLE SEWER LAYOUT FOR POPULATION >450

Population	650					
Average Daily Demand (ADD)	190	liters/day				
Capacity Factor	1.5					
Average Daily Flow (ADF)	148200	liters/day	39154.44	gal/day	1.715	Lps
Peak Diurnal Flow (Pd)	484046.56	liters/day	5.60	Lps		

(light blue)	Indicates designer entered value
(light orange)	Calculated from the graph

Building	Building Fixture Units	Percent of Total Fixture Flow	GPM	Lps
Administration	78	8.3%	60.56	3.82
Administration	78	8.3%	60.56	3.82
Dining Facility	55.9	6.0%	52.36	3.30
Dining Facility	55.9	6.0%	52.36	3.30
Senoir Barrack	28	3.0%	40.4	2.55
Senoir Barrack	28	3.0%	40.4	2.55
Latrine Facility	202	21.5%	90.44	5.71
Latrine Facility	202	21.5%	90.44	5.71
Latrine Facility	202	21.5%	90.44	5.71
Vehicle Mainten.	8	0.9%	22.2	1.40
Total Fixture Units	937.8			

VALUES BASED ON THE IPC 1007 TABLE E103.3(3)

THIS COLUMN INDICATES THAT THE FIXTURE UNIT FLOW IS GREATER THEN 0.6 M/SEC AND FLOWING LESS THEN 80% FULL (d/D).

BUILDING CONNECTIONS

Pipe Description	Drain Fixture	Slope	n	diameter		Area (Full)		R		Vel full,Vf	flow Full, Qf		Qs/Qf	Vs/Vf		d/D	Vs, m/s	Sized Correctly
	Lps			mm	m	mm^2	m^2	mm	m	m/s	m^3/s	Lps	Fixture	Fixture		Fixture	Fixture	Fixture
Administration	3.82	1.00%	0.013	150	0.15	17671.459	0.018	37.5	0.0375	0.86	0.0152	15.23	0.251	0.800		35.0%	0.69	OK
Dining Facilty	3.30	1.00%	0.013	150	0.15	17671.459	0.018	37.5	0.0375	0.86	0.0152	15.23	0.217	0.766		32.6%	0.66	OK
Senoir Barracks	2.55	1.00%	0.013	150	0.15	17671.459	0.018	37.5	0.0375	0.86	0.0152	15.23	0.167	0.701		29.2%	0.60	OK
Laterine Facility	5.71	1.00%	0.013	150	0.15	17671.459	0.018	37.5	0.0375	0.86	0.0152	15.23	0.375	0.919		43.4%	0.79	OK
Vehicle Mainten.	1.40	2.00%	0.013	150	0.15	17671.459	0.018	37.5	0.0375	1.22	0.0215	21.54	0.065	0.489		17.1%	0.60	OK

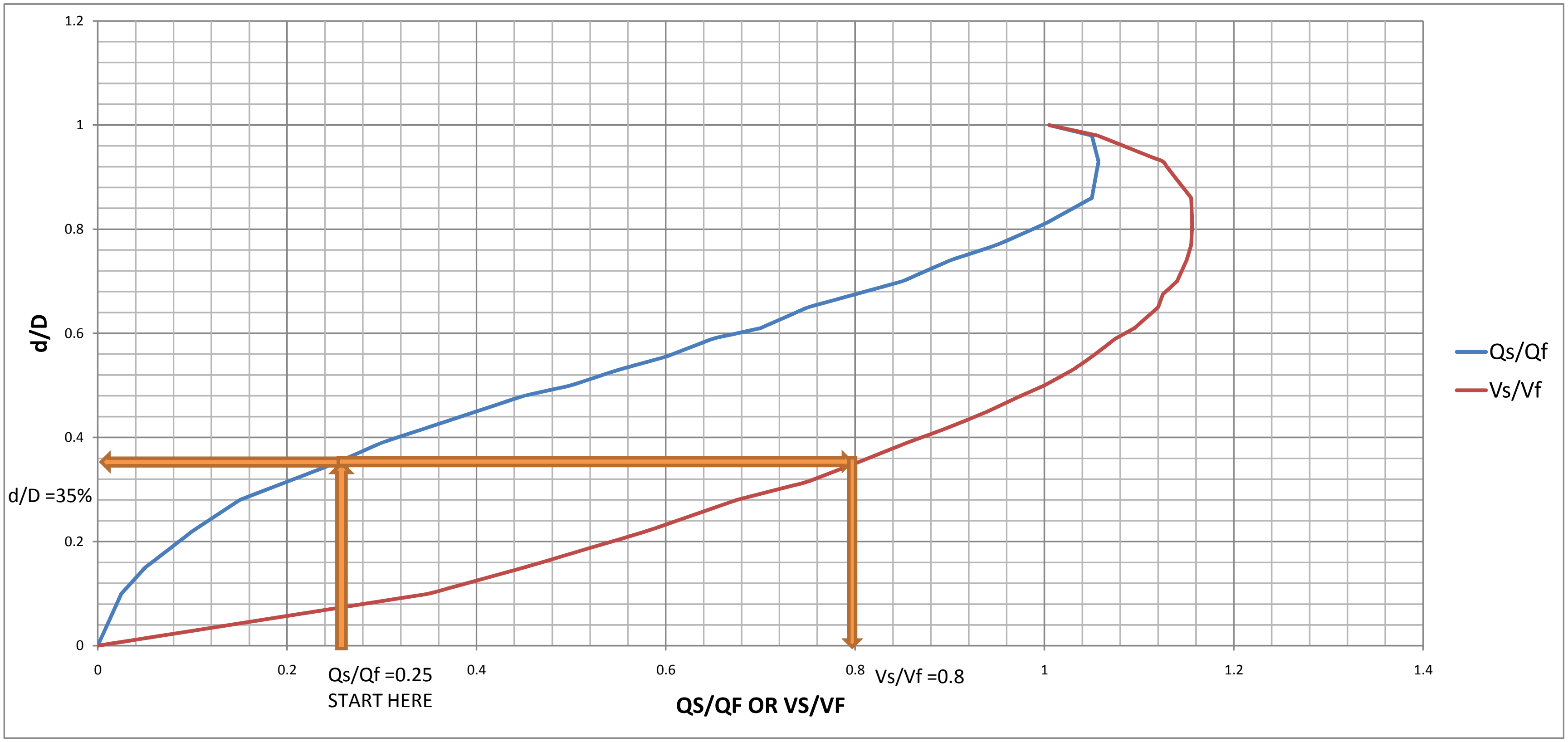
M5-M6 PEAK DIURNAL FLOW IS CALCULATED:
(8.3%*2+21.5%*2+6.0%)*5.6 Lps = 3.68 Lps
USE THE SAME METHOD TO CALCULATE THE ADF:
(8.3%*2+21.5%*2+6.0%)*1.72 Lps = 1.13 Lps

ENSURE THAT EITHER THE DIURNAL OR ADD FLOW IS GREATER THEN 0.6 M/SEC AND FLOWING LESS THEN 80% FULL (d/D)

LATERALS AND MAINS

Pipe Description	Diunral	Adf	Slope	n	diameter		Area (Full)		R		Vel full,Vf	flow Full, Qf		Qs/Qf		Vs/Vf			Vs, m/s		d/D		Sized Correctly	
	Lps	Lps			mm	m	mm^2	m^2	mm	m	m/s	m^3/s	Lps	Diurnal	ADF	Diurnal	ADF		Diurnal	ADF	Diurnal	ADD	Diurnal	ADF
M1-M5	Building Connection																							
M2-M5	1.67	0.51	1.75%	0.013	150	0.15	17671.46	0.018	37.5	0.038	1.14	0.0201	20.15	0.083	0.025	0.536	0.350		0.61	0.40	19.6%	10.0%	OK	Try Again
M3-M4	Building Connection																							
M4-M5	1.67	0.51	1.75%	0.013	150	0.15	17671.46	0.018	37.5	0.038	1.14	0.0201	20.15	0.083	0.025	0.536	0.350		0.61	0.40	19.6%	10.0%	OK	Try Again
M5-M6	3.68	1.13	1.00%	0.013	150	0.15	17671.46	0.018	37.5	0.038	0.86	0.0152	15.23	0.242	0.074	0.792	0.512		0.68	0.44	34.4%	18.4%	OK	Try Again
M6-M7	4.01	1.23	1.00%	0.013	150	0.15	17671.46	0.018	37.5	0.038	0.86	0.0152	15.23	0.264	0.081	0.815	0.531		0.70	0.46	36.1%	19.3%	OK	Try Again
M7-M9	4.01	1.23	1.00%	0.013	150	0.15	17671.46	0.018	37.5	0.038	0.86	0.0152	15.23	0.264	0.081	0.815	0.531		0.70	0.46	36.1%	19.3%	OK	Try Again
M8-M9	Building Connection																							
M10-M11	Building Connection																							
M11-M12	Building Connection																							
M12-M9	1.25	0.38	2.50%	0.013	150	0.15	17671.46	0.018	37.5	0.038	1.36	0.0241	24.08	0.052	0.016	0.455	0.224		0.62	0.31	15.3%	6.4%	OK	Try Again
M9-M13	5.60	1.72	1.00%	0.013	150	0.15	17671.46	0.018	37.5	0.038	0.86	0.0152	15.23	0.368	0.113	0.914	0.605		0.79	0.52	43.1%	23.6%	OK	Try Again
M13- Septic Tank	5.60	1.715	1.00%	0.013	150	0.15	17671.46	0.018	37.5	0.038	0.86	0.0152	15.23	0.368	0.113	0.914	0.605		0.79	0.52	43.1%	23.6%	OK	Try Again

AED Design Requirements
Sanitary Sewer & Septic Systems



Appendix B – Drawing Detail

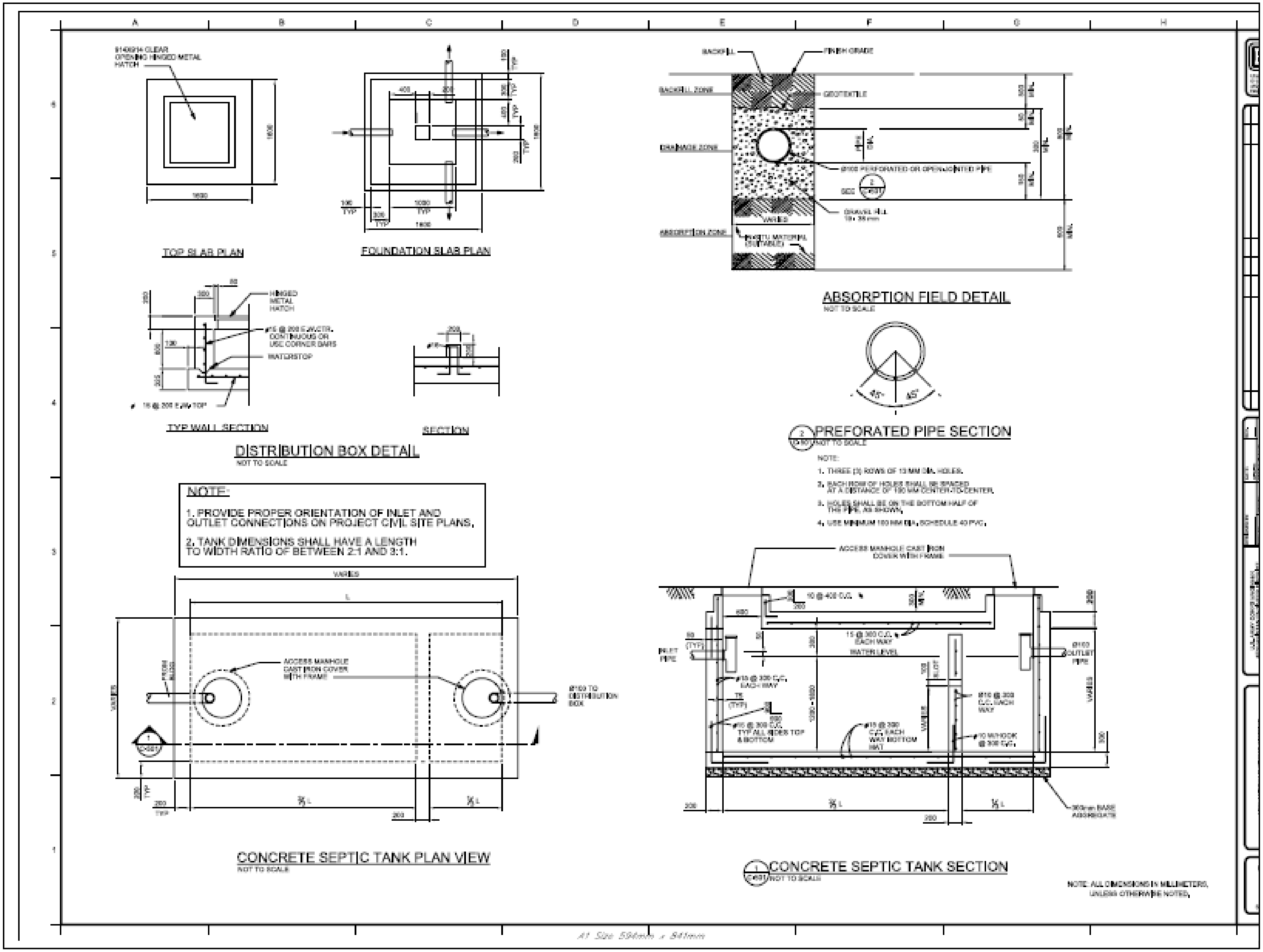
Septic Tank Details

Absorption Bed and Trench Details

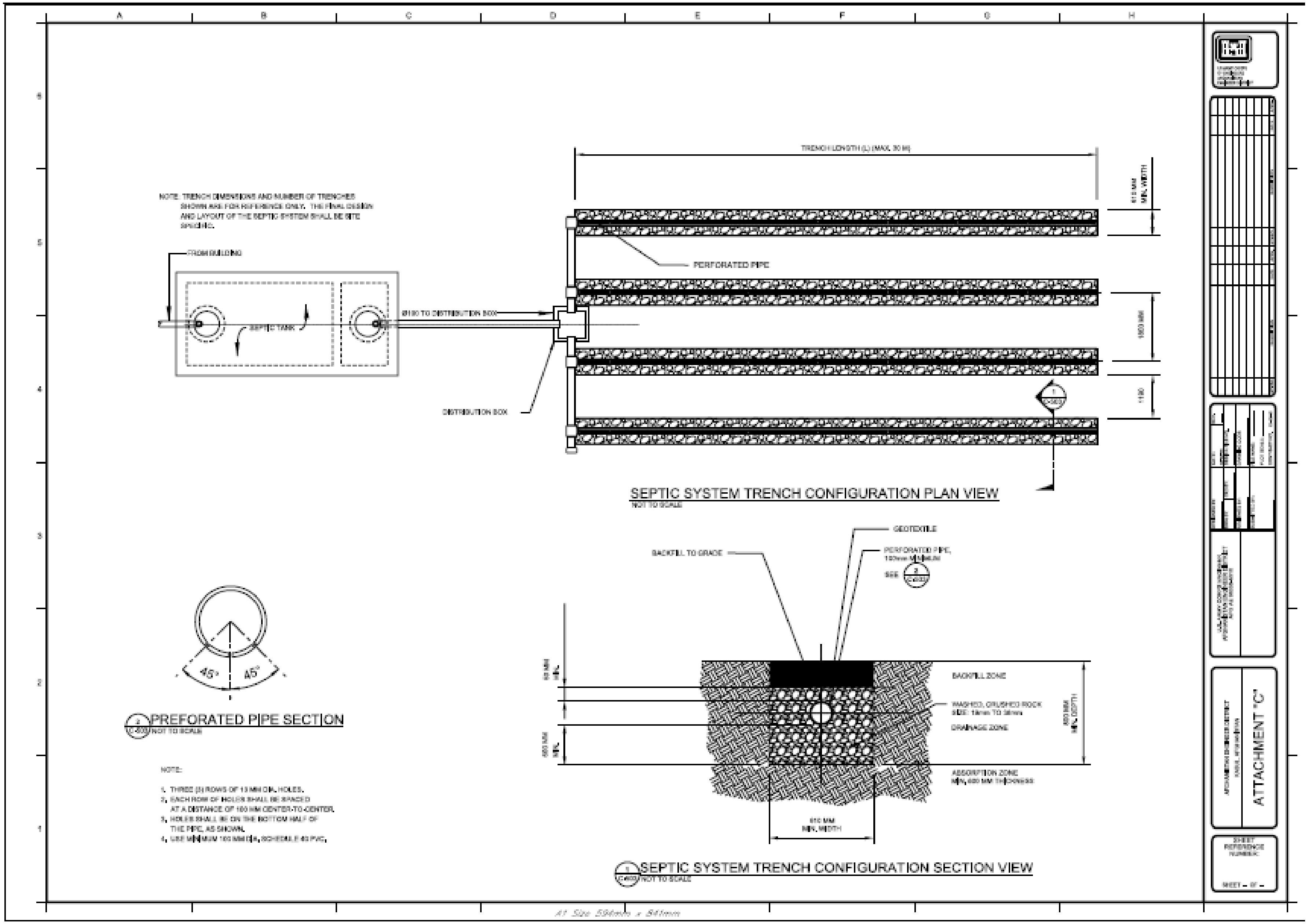
Dosing system Layout

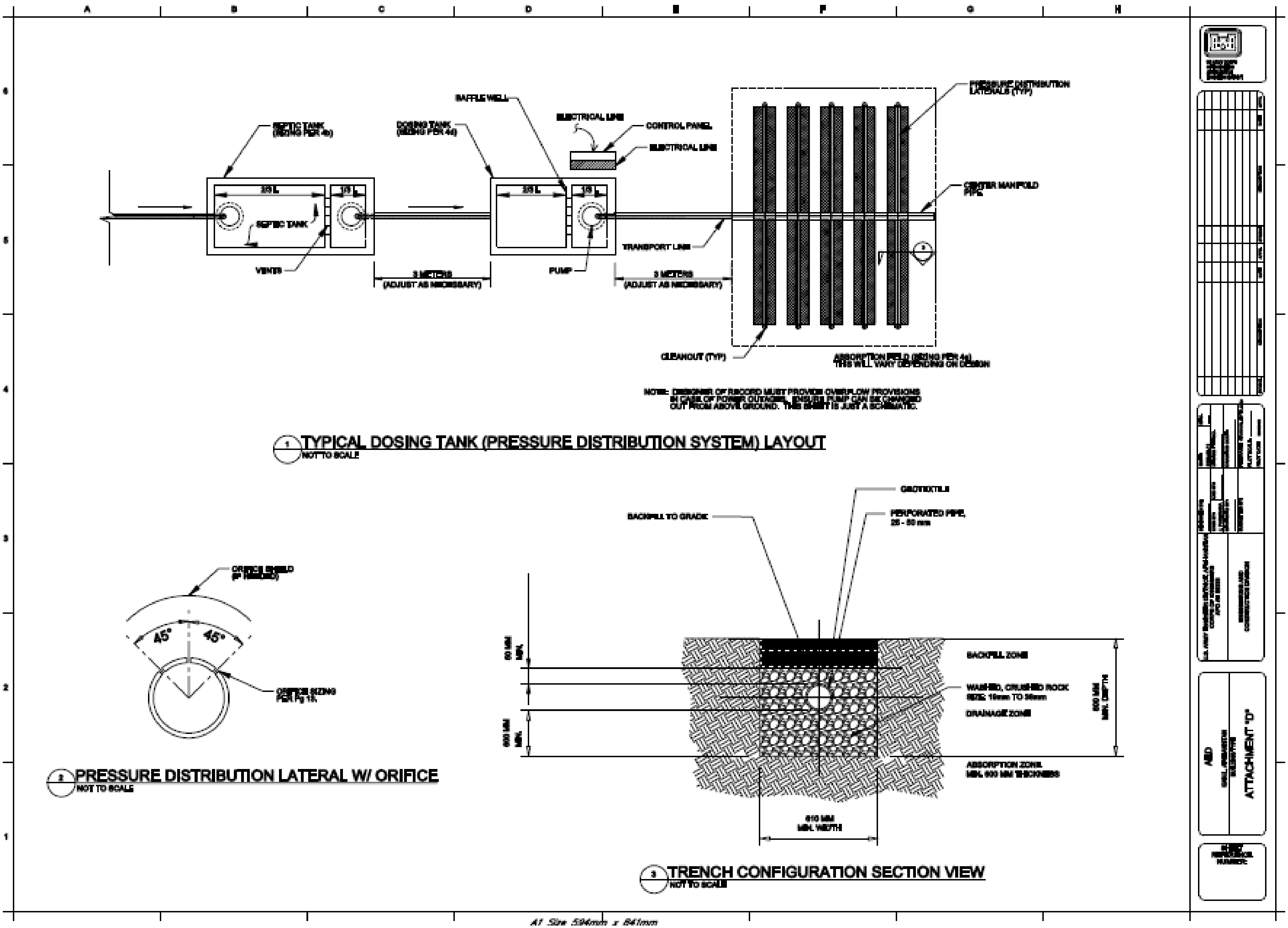
Leaching Chambers Option 1A and 1B (Traffic Loading)

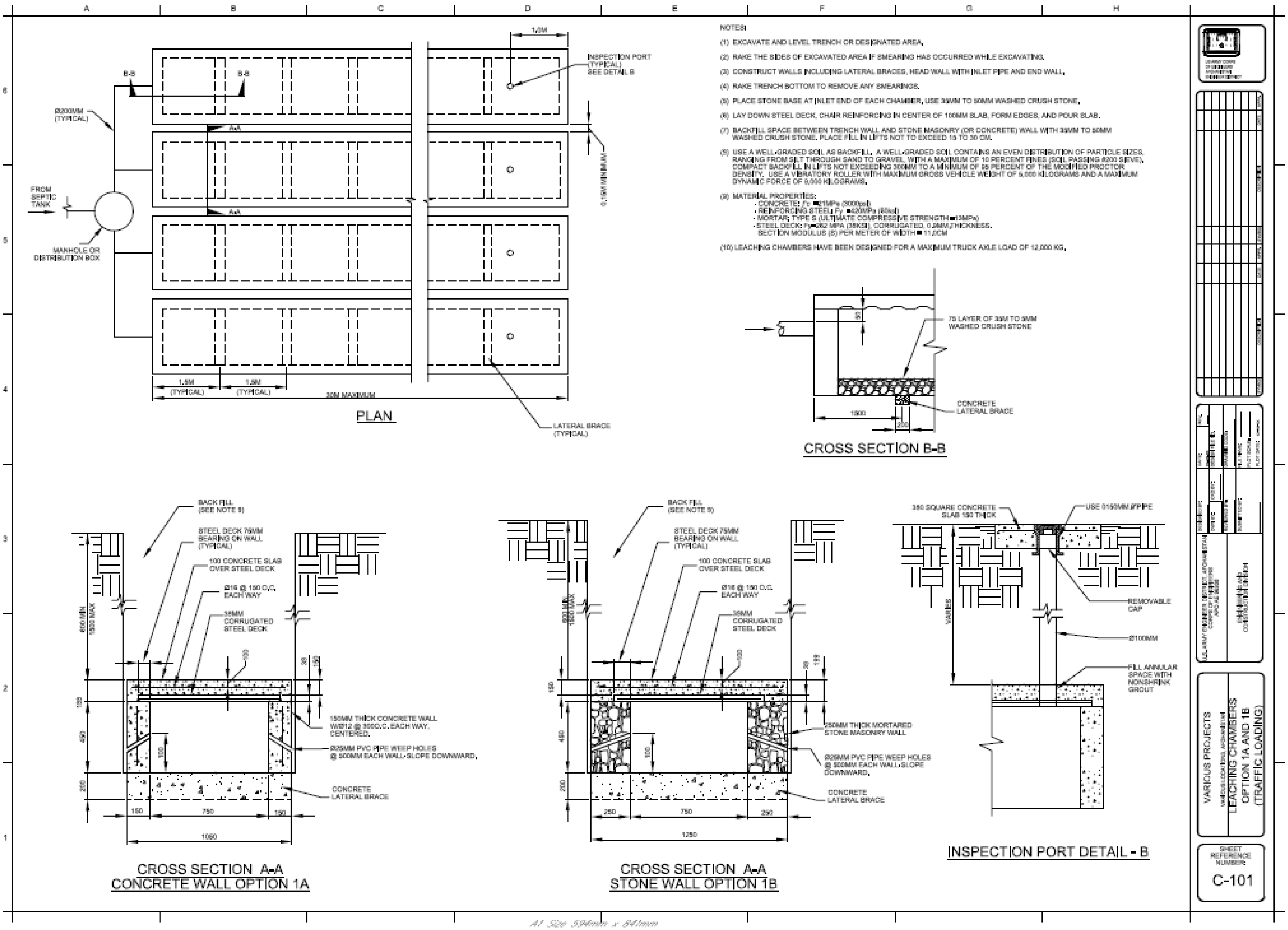
Leaching Chambers Option 2

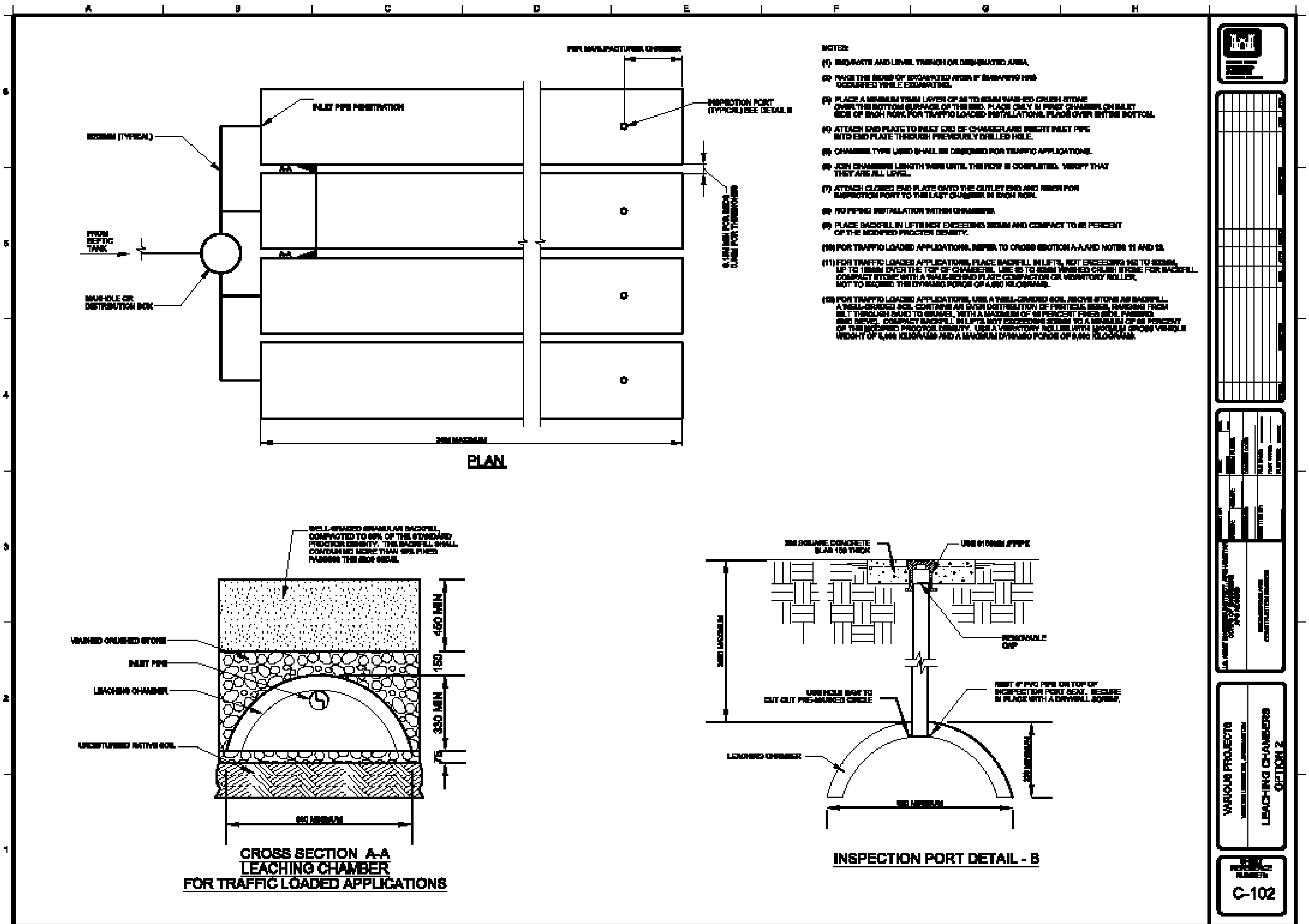

TOP SLAB PLAN
FOUNDATION SLAB PLAN
TYP WALL SECTION
SECTION
DISTRIBUTION BOX DETAIL
NOT TO SCALE
NOTE:
1. PROVIDE PROPER ORIENTATION OF INLET AND OUTLET CONNECTIONS ON PROJECT CIVIL SITE PLANS.
2. TANK DIMENSIONS SHALL HAVE A LENGTH TO WIDTH RATIO OF BETWEEN 2:1 AND 3:1.
ABSORPTION FIELD DETAIL
NOT TO SCALE
BACKFILL
FINISH GRADE
BACKFILL ZONE
GEOTEXTILE
DRAINAGE ZONE
ABSORPTION ZONE
PREFORATED PIPE SECTION
NOT TO SCALE
NOTE:
1. THREE (3) ROWS OF 13 MM DIA. HOLES.
2. EACH ROW OF HOLES SHALL BE SPACED AT A DISTANCE OF 150 MM CENTER-TO-CENTER.
3. HOLES SHALL BE ON THE BOTTOM HALF OF THE PIPE, AS SHOWN.
4. USE MINIMUM 100 MM DIA. SCHEDULE 40 PVC.
ACCESS MANHOLE CAST IRON COVER WITH FRAME
INLET PIPE
OUTLET PIPE
WATER LEVEL
CONCRETE SEPTIC TANK PLAN VIEW
NOT TO SCALE
CONCRETE SEPTIC TANK SECTION
NOT TO SCALE
NOTE: ALL DIMENSIONS IN MILLIMETERS, UNLESS OTHERWISE NOTED.
A1 Size 594mm x 841mm

AED Design Requirements
Sanitary Sewer & Septic Systems




BAFFLE WALL
ELECTRICAL LINE
CONTROL PANEL
ELECTRICAL LINE
SEPTIC TANK (SIZING PER 4a)
DOSING TANK (SIZING PER 4d)
SEPTIC TANK
VENTS
PUMP
3 METERS (ADJUST AS NECESSARY)
3 METERS (ADJUST AS NECESSARY)
TRANSPORT LINE
PRESSURE DISTRIBUTION LATERALS (TYP)
CENTER MANIFOLD PIPE
CLEANOUT (TYP)
ABSORPTION FIELD (SIZING PER 4a) THIS WILL VARY DEPENDING ON DESIGN
NOTE: DESIGNER OF RECORD MUST PROVIDE OVERFLOW PROVISIONS IN CASE OF POWER OUTAGES. ENSURE PUMP CAN BE CHANGED OUT FROM ABOVE GROUND. THIS SHEET IS JUST A SCHEMATIC.
1 TYPICAL DOSING TANK (PRESSURE DISTRIBUTION SYSTEM) LAYOUT
NOT TO SCALE
ORIFICE SHIELD (IF REQUIRED)
45°
45°
ORIFICE SIZING PER Pg 15.
2 PRESSURE DISTRIBUTION LATERAL W/ ORIFICE
NOT TO SCALE
GEOTEXTILE
BACKFILL TO GRADE
PERFORATED PIPE, 25 - 50 mm
BACKFILL ZONE
WASHED, CRUSHED ROCK SIZE: 19mm TO 38mm
DRAINAGE ZONE
ABSORPTION ZONE MIN. 600 MM THICKNESS
610 MM MIN. WIDTH
3 TRENCH CONFIGURATION SECTION VIEW
NOT TO SCALE
AED
ATTACHMENT "D"
A1 Size 594mm x 841mm


NOTES:
(1) EXCAVATE AND LEVEL TRENCH OR DESIGNATED AREA.
(2) RAKE THE SIDES OF EXCAVATED AREA IF SMEARING HAS OCCURRED WHILE EXCAVATING.
(3) CONSTRUCT WALLS INCLUDING LATERAL BRACES, HEAD WALL WITH INLET PIPE AND END WALL.
(4) RAKE TRENCH BOTTOM TO REMOVE ANY SMEARINGS.
(5) PLACE STONE BASE AT INLET END OF EACH CHAMBER, USE 35MM TO 50MM WASHED CRUSH STONE.
(6) LAY DOWN STEEL DECK, CHAIR REINFORCING IN CENTER OF 100MM SLAB, FORM EDGES, AND POUR SLAB.
(7) BACKFILL SPACE BETWEEN TRENCH WALL AND STONE MASONRY (OR CONCRETE) WALL WITH 35MM TO 50MM WASHED CRUSH STONE. PLACE FILL IN LIFTS NOT TO EXCEED 15 TO 30 CM.
(8) USE A WELL-GRADED SOIL AS BACKFILL. A WELL-GRADED SOIL CONTAINS AN EVEN DISTRIBUTION OF PARTICLE SIZES, RANGING FROM SILT THROUGH SAND TO GRAVEL, WITH A MAXIMUM OF 10 PERCENT FINES (SOIL PASSING #200 SIEVE). COMPACT BACKFILL IN LIFTS NOT EXCEEDING 300MM TO A MINIMUM OF 95 PERCENT OF THE MODIFIED PROCTOR DENSITY. USE A VIBRATORY ROLLER WITH MAXIMUM GROSS VEHICLE WEIGHT OF 5,000 KILOGRAMS AND A MAXIMUM DYNAMIC FORCE OF 9,000 KILOGRAMS.
(9) MATERIAL PROPERTIES:
- CONCRETE: f'c = 21MPa (3000psi)
- REINFORCING STEEL: Fy = 420MPa (60ksi)
- MORTAR: TYPE S (ULTIMATE COMPRESSIVE STRENGTH=13MPa)
- STEEL DECK: Fy=262 MPA (38KSI), CORRUGATED, 0.8MM THICKNESS. SECTION MODULUS (S) PER METER OF WIDTH = 11.0CM
(10) LEACHING CHAMBERS HAVE BEEN DESIGNED FOR A MAXIMUM TRUCK AXLE LOAD OF 12,000 KG.
PLAN
FROM SEPTIC TANK
MANHOLE OR DISTRIBUTION BOX
Ø200MM (TYPICAL)
INSPECTION PORT (TYPICAL) SEE DETAIL B
LATERAL BRACE (TYPICAL)
1.5M (TYPICAL)
30M MAXIMUM
CROSS SECTION B-B
75 LAYER OF 35M TO 5MM WASHED CRUSH STONE
CONCRETE LATERAL BRACE
CROSS SECTION A-A CONCRETE WALL OPTION 1A
BACK FILL (SEE NOTE 8)
STEEL DECK 75MM BEARING ON WALL (TYPICAL)
100 CONCRETE SLAB OVER STEEL DECK
Ø16 @ 150 O.C. EACH WAY
38MM CORRUGATED STEEL DECK
150MM THICK CONCRETE WALL W/Ø12 @ 300O.C. EACH WAY, CENTERED.
Ø25MM PVC PIPE WEEP HOLES @ 500MM EACH WALL-SLOPE DOWNWARD.
CONCRETE LATERAL BRACE
CROSS SECTION A-A STONE WALL OPTION 1B
250MM THICK MORTARED STONE MASONRY WALL
INSPECTION PORT DETAIL - B
350 SQUARE CONCRETE SLAB 150 THICK
USE Ø150MM Ø PIPE
REMOVABLE CAP
Ø100MM
FILL ANNULAR SPACE WITH NONSHRINK GROUT
VARIOUS PROJECTS
LEACHING CHAMBERS OPTION 1A AND 1B (TRAFFIC LOADING)
SHEET REFERENCE NUMBER
C-101
A1 Size 594mm x 841mm



A1 Size 594mm x 841mm

AED Design Requirements
Sanitary Sewer & Septic Systems

US Army Corps
of Engineers
Afghanistan Engineer District

AED Design Requirements:
Site Layout Guidance

Various Locations,
Afghanistan

MARCH 2009

TABLE OF CONTENTS

AED DESIGN REQUIREMENTS
FOR
SITE LAYOUT GUIDANCE
VARIOUS LOCATIONS,
AFGHANISTAN

Figures

Tables

1. General

The purpose of this document is to provide requirements to Contractors for any project requiring site layout design and construction.

2. Site Layout

The site layout defines building location, vehicular circulation and parking, pedestrian circulation, utility systems design, and physical security. It also includes the identification and evaluation of the site layout. Identification includes defining site specific goals and objectives, verifying the program requirements, developing functional relationships, defining spatial relationships, and providing an inventory of the area. Evaluation includes the development of a site analysis that graphically shows the developmental opportunities and constraints for the site. Alternative conceptual plans are developed for evaluation and a determination of a final site plan is accomplished. The resulting site layout provides the basis for the preparation of construction drawings. The design criteria discusses building design, location and orientation, vehicular circulation and parking, pedestrian circulation, surface water management, utility systems design, lighting design, landscape design, and physical security.

3. Building Location

The building location on the site may be determined by considering the following factors.

a) Dimensional Factors. Dimensional factors include the building dimensions or footprint and the following factors:

1) Buffer Zones. Buffer zones provide setbacks and safety protection from airfield and helipad, explosive storage areas and storage and handling of hazardous materials. Additionally, buffer zones provide noise abatement and separation of incompatible land use or functions as well as physical security clearances.

2) Spacing Requirements. Spacing between buildings and functions is normally determined by their functional relationships, operational efficiency, fire protection clearances, physical security requirements, parking requirements, future expansion requirements, and open space requirements.

b) Environmental Factors. The location and condition of such elements as geology, soils, drainage, and vegetation may create areas that should be excluded from development because they are unbuildable for structural, economic or environmental reasons, they require protection or they require preservation.

c) Orientation Factors. Building location may be influenced by orientation to enhance energy conservation. Orientation to take advantage of or reduce the impact of prevailing winds and solar radiation should be considered when siting buildings.

d) Other Siting Factors. Other site-specific conditions can influence building alignment such as the ability to accomplish the mission, ability to minimize travel time and the ability to control access.

4. Vehicular Circulation

Circulation should promote safe and efficient movement of vehicles and pedestrians. Maintaining maximum separation of vehicles and pedestrians helps promote safety. Safe roadway circulation systems have a perceivable hierarchy of movement, lead to a clear destination and do not interrupt other activities. The road system should be planned to keep groupings of related functions reasonably close to each other and the interrelating land-use areas for maximum efficiency. Additionally, the road system should minimize on site travel and permit the optimum circulation of traffic originating both outside and within the site. The American Association of State Highway and Transportation Officials (AASHTO) places vehicles into two general classes: passenger cars and trucks. The passenger car class includes passenger cars, and light delivery trucks such as vans and pickups. The truck class includes single-unit trucks, recreation vehicles, buses, trucks semi-tractor trailer combinations, and trucks or truck tractors with semi-trailers in combination with full trailers. Roadway layout to provide the maneuverability and traffic safety required by the vehicles that utilize the roadways is necessary. Table 1 lists dimensions for some of the more common vehicles. Table 2 lists minimum turning radii for the same vehicles.

Table 1. Dimensions for Design Vehicles

	Vehicle Dimension		*Bumper Overhang*	
	Width	*Length*	*Front*	*Rear*
Passenger Car (P)	2.1 (7.0)	5.8 (19.0)	0.9 (3.0)	1.5 (5.0)
Single Unit Truck (SU)	2.6 (8.5)	9.2 (30.0)	1.2 (4.0)	1.8 (6.0)
Intermediate Semi-trailer (WB-40)	2.6 (8.5)	15.3 (50.0)	1.2 (4.0)	1.8 (6.0)
Large Semi-trailer (WB-50)	2.6 (8.5)	16.8 (55.0)	0.9 (3.0)	0.6 (2.0)
Single Unit Bus (BUS)	2.6 (8.5)	12.2 (40.0)	2.1 (7.0)	2.4 (8.0)
Motor Home (MH)	2.4 (8.0)	9.2 (30.0)	1.2 (4.0)	1.8 (6.0)

Table 2. Minimum Turning Radii for Design Vehicles

Design Vehicle	*Minimum Design Turning Radius m (ft)*	*Minimum Inside Radius m (ft)*
Passenger Car	7.3 (24.0)	4.7 (15.3)
Single Unit Truck	12.8 (42.0)	8.7 (28.4)
Intermediate Semi-trailer	12.2 (40.0)	6.1 (19.9)
Large Semi-trailer	13.7 (45.0)	6.1 (19.8)
Single Unit Bus	12.8 (42.0)	7.1 (23.2)
Motor Home	12.8 (42.0)	8.7 (284)

Design site entrances and exits, services drives, and other areas with special requirements (e.g., parking lots or loading docks) to accommodate the largest vehicle that will use the facility. This procedure should assure that traffic safety will be accommodated.

a) Access Intersections. Access intersections should be controlled to minimize the conflicts between through traffic on the main road and vehicles entering and exiting the site. Proper layout of access intersections may reduce conflicts between the traffic entering the site and the through traffic aon the main road. Points of conflict can be limited by the following:

Reducing the number of access drives to one (1) two-way drive or a pair of one-way drives for each site. Drives may be added to the site if the daily traffic volume exceeds 5,000 vehicles per day (both directions) or if traffic using one drive would exceed the capacity of a stop-sign-controlled intersection during the peak (highest) traffic hour.

Increasing the space between access intersections and between access intersections and roadway intersections. The correct spacing of access drives will promote safety for vehicular traffic. For arterial roads where access to the road is not limited, the minimum spacing between access roads should be 61 m (200 ft). Table 3 provides acceptable minimum spacing requirements when frontage along an arterial road is limited. Maintain a minimum spacing of 366m to 457m (1,200 to 1,500 ft) between a signaled drive and adjacent signaled intersection. If the signaled drive is a T-intersection, 183m (600 ft) is an acceptable minimum spacing when frontage is limited. Coordinate drive signals within 762m (2,500 ft) of adjacent signals. Maintain a minimum spacing of 10.5m to 15.5m (35 to 50 ft) on low-volume (5,000 vehicles per day), low-speed (48 kph (30 mph)) roads.

Table 3. Minimum Drive Spacing for Arterial Roads with Limited Access

Arterial Speed kph (mph)	*Minimum Separation m (ft)*
32 (20)	25.9 (85.0)
40 (25)	32 (105.0)
48 (30)	38 (125.0)
56 (35)	45.8 (150.0)
64 (40)	56.4 (185.0)
72 (45)	70.2 (230.0)
81 (50)	83.9 (275.0)

Access drives near major intersections adversely affect traffic operations. They may result in unexpected conflicts with vehicles turning at the intersection. Maintain a minimum clearance of 15.2m (50 ft) between access drives and major intersections.

Provide adequate road width and length along the access drive at the intersection to channel vehicles smoothly into the proper lanes. Providing left-turn lane and right-turn and acceleration lane on the main roadway at the access drive.

b) Controlled Entrances. Controlled entrances are to be provided at the entrance to large complexes or secure facilities. Controlled entrances should contain a traffic island, gates and a way to have the vehicles denied entrance to the site an exit without entering the site. The traffic island with curbs should be a minimum of 3.1 meters wide and shall be used to separate incoming and outgoing traffic. A gate house may be provided within the area of the traffic island. The minimum throat length should be long enough to accommodate stacking of vehicles entering and exiting the site without interfering with the traffic flow on adjacent roads. A pull-off area should be provided on the incoming traffic lanes for the close inspection of vehicles prior to entering the site.

c) Sight Distances. Provide safe sight distance for vehicles entering and exiting an access drive. This sight distance increases according to the design speed of the through road. The relationships of speed to sight distances are provided in Table 4.

Table 4. Minimum Sight Distance

Operating Speed (kph (mph))	*32 kph (20 mph)*		*48 kph (30 mph)*		*64 kph (40 mph)*		*81 kph (50 mph)*	
	Left m (ft)	*Right m (ft)*	*Left m (ft)*	*Right m (ft)*	*Left m (ft)*	*Right m (ft)*	*Left m (ft)*	*Right m (ft)*
Passenger car	**64 (210)**	**52 (170)**	**99 (320)**	**112 (360)**	**167 (540)**	**183 (590)**	**279 (900)**	**301 (970)**
Truck	**112 (360)**	**71 (30)**	**161 (520)**	**140 (450)**	**285 (920)**	**285 (920)**	**468 (1510)**	**474 (1530)**

NOTE: Sight distances are based on the following assumptions:

1. Upon turning left or right when exiting the access drive, the vehicle accelerates to the operating speed of the access road without causing approaching vehicles to reduce speed by more than 16 kph (10 mph).

2. Upon turning left when entering the access drive, the vehicle clears the near half of the access road without causing approaching vehicles to reduce speed by more than 16 kph (10 mph).

3. Turns are 90-degree.

4. The access road and the access drive are on level terrain.

When a safe sight distance cannot be met, the designer should consider methods to achieve adequate sight distance. Some methods of achieving adequate sight distance are the removal of sight obstructions, the relocation of the access drive to a more favorable location along the access street or the relocation of the access drive to another access road.

5. Parking

Parking includes on-street parking, off-street parking lots, and parking structures. On-street parking will be limited to parallel parking spaces that include sufficient length and width to allow safe movement into and out of the space and to adequately separate the parked vehicle from the traffic lanes. All parking areas should be within close walking distance of the building that they serve. A recommended minimum 6m wide buffer strip should be provided to separate parking areas from adjacent streets. In areas of limited space provide a minimum distance of 2.4m may be provided.

Off-street parking lots are the principal means of parking on installations. A 90-degree parking layout is preferable. Where a fast rate of turnover is expected or where required by site limitations, a 45-degree or 60-degree angle layout may be used. Design the parking layout to provide regular parking (2.75m x 5.80m) and handicapped parking spaces (4.25m x 5.80m). Such features as curb cuts and access aisles for barrier-free access to sidewalks should be included for handicapped access. The maintenance of two-way traffic in parking lots is encouraged. Dead end parking lots should be avoided. Provide more than one entrance and exit for parking lots with more than 100 parking spaces. Curbs or a painted line at the end of parking stalls should be used to control placement of vehicles. Provide adequate walkway width to allow comfortable pedestrian movement in areas of bumper overhang. The minimum turning radii should accommodate the largest vehicle expected to use the parking lot as identified in Table 2.

Locate islands at the ends of parking stalls and at the intersections of parking aisles. Medians may be placed between adjacent rows of parking stalls. The islands establish turning radii for vehicular movement and protect end stalls. Turning radii to be used is based upon the largest vehicle that will utilize the parking lot. Include turning radii that is sufficient to allow safe traffic movement without conflicting with the island and/or curbing. Islands and medians can be partially or completely paved to service pedestrian traffic. Pedestrians tend to use circulation aisles, especially if medians are not generous and do not allow for comfortable movement between vehicles. If the median is designed as a sidewalk, provide a width that allows for pedestrian movement and vehicle overhang.

Landscaping in islands and medians should be considered to break up the expanse of impermeable and unshaded surface, provide a more pleasing visual and spatial appearance. When placing landscaping in islands or medians, the designer should consider the 1.07m motorist eye level viewing height when providing shrubs and small trees.

Grading of parking lots should maintain a relatively constant grade across the lot that includes no less than the minimum slope of one (1) percent required for positive drainage to properly direct drainage to swales or to drainage inlets. The maximum slope within a 90-degree parking space of five (5) percent from front to rear end of the vehicle and one and one-half (1 ½) percent from side to side. Provide a maximum slope within a 45-degree or 60-degree parking space of five (5) percent from front to rear end and one (1) percent from side to side. Islands and medians may be used to accommodate change in elevation between the access drive and parking areas or between different parking levels.

6. Pedestrian Circulation

Pedestrian circulation involves the movement of people by non-motorized means along sidewalks. Pedestrian circulation should be based on pedestrians' tendency to follow the most direct route when walking between two points. Sidewalk paths may be gridded, curvilinear or organic. Paths should incorporate required and anticipated access. All three systems provide functional access between facilities. Topography and vegetation can be used to direct movement and emphasize sight lines along paths.

A grid path system is composed of straight lines and right angles and tends to provide the most direct access between locations. The grid system is appropriate in formal landscapes and in areas with strong architectural definition. A curvilinear path system is less formal and should be used to encourage pedestrian interaction with the landscape where direct access to facilities is not critical. Organic sidewalk systems are unique in that the sidewalk patterns are defined by the space outside of the sidewalk and therefore vary in width. Because of this, organic sidewalks are less formal and often respond to natural elements in the surrounding landscape.

The space required to accommodate pedestrian movement increases at the point of origin and destination, where movement slows. Pedestrian movement is also interrupted when people meet, gather, wait, or sit. In areas of pedestrian concentration (e.g., building entrances, drop-offs and small outdoor spaces between buildings), the space should be developed to accommodate these needs. General design techniques include widening walkways at the points of origin and destination and providing both shaded and sunny areas for people to congregate or sit on the edge or outside of the pedestrian flow.

Sidewalks should be provided with a smooth and hard surface such as concrete, asphalt or pavers. The minimum width of the sidewalk should be 1.2 meters. As the amount of traffic increases on a sidewalk, the width should be adjusted accordingly. The grade of a sidewalk should follow the natural grade of the ground as nearly as possible. The longitudinal grade along the sidewalk should not be greater than about 12 percent with the cross-slope of the sidewalk not greater than 2.08 percent. Steps should be avoided if possible but will be used where the maximum longitudinal slope would otherwise be too great. Steps should be grouped together rather than spaced as individual steps. Where steps are required, consideration should be given to handicapped ramps to provide continuous handicapped accessability. Sidewalks should be separated from roads by a turfed area at least 0.6 meters wide for low speed roads. As the speed of the roadway increased, the separation distance between the sidewalk and the roadway should increase.

7. Utility System Design

Utility systems should minimize impact to the natural site while meeting basic economic and functional criteria. Utility corridors should be used to minimize environmental disturbance and simplify maintenance. These corridors should be located along a site's perimeter and not cross a site diagonally or indiscriminately because future realignment of existing systems will increase the costs of future development. Utilities should be placed underground wherever possible to avoid conflicts with vegetation, provide protection from storm damage, and enhance the visual quality of the installation. To simplify maintenance, utility lines should not be placed under paved areas, but located at the back of the roadway curb. It is extremely important to determine the potential for future expansion and to allow for upgrading the system when locating utilities. Utility transformers and transclosures for underground utilities shall be located to ensure ease of access for maintenance but not obstruct site primary visual relationships. They should be located with adequate setbacks from vehicular circulation and parking areas.

a) Utility Separation. Water mains should have a minimum horizontal clearance of 3.05 meters from any point of an existing or proposed sanitary sewer or storm drain line. Water mains and sanitary sewers must not be installed in the same trench. If any condition prevents a horizontal separation of 3.50 meters, a minimum horizontal separation of 1.80 meters can be allowed with the bottom of the water main a minimum of 0.30 meters above the top of the sanitary sewer line. Where water mains and sanitary sewers follow the same roadway, they will be installed on opposite sides of the roadway, if possible. Where water mains and sewer lines cross, the sewer line will have no joints within 0.91 meters of the water main unless the sewer line is encased in concrete for a distance of at least 3.05 meters each side of the crossing. If conditions dictate that a water main be laid under a gravity sewer, the sewer pipe will be fully encased in concrete for a distance of 3.05 meters each side of the crossing or will be made of pressure pipe with no joint located within 0.30 meters horizontally of the water main. Pressure sewer pipe will always cross beneath water pipes with a minimum vertical distance of 0.60 meters between the bottom of the water pipe and the top of the pressure sewer pipe.

8. Physical Security

Operational, logistic, and security requirements must be integrated into the overall design of buildings, equipment, landscaping, parking, roads, and other features. The most cost-effective solution for mitigating explosive effects on buildings is to keep explosives as far as possible from them. Standoff distance must be coupled with appropriate building hardening to provide the necessary level of protection. The following standards detail standoff distances that when achieved will allow for buildings to be built with minimal additional construction costs. Where these standoff distances cannot be achieved because land is unavailable, these standards allow for building hardening to mitigate the blast effects.

a) Standard 1. Standoff Distances. The standoff distances apply to all new and existing (when triggered) buildings covered by these standards. The standoff distances are presented in Table 5 and illustrated in Figures 1 and 2 for new buildings and Figures 3 and 4 for existing buildings. Where the standoff distances in the "Conventional Construction Standoff Distance" column of Table 5 can be met, conventional construction may be used for the buildings without a specific analysis of blast effects, except as otherwise required in these standards.

Where the conventional construction standoff distances are not available, an engineer experienced in blast-resistant design should analyze the building and apply building hardening as necessary to mitigate the effects of the explosives indicated in Table 5 at the achievable standoff distance to the appropriate level of protection.

For new buildings, standoff distances of less than those shown in the "Minimum Standoff Distance" column in Table 5 are not allowed. For existing buildings, the standoff distances in the "Minimum Standoff Distance" column of Table 5 will be provided except where doing so is not possible. In those cases, lesser standoff distances may be allowed where the required level of protection can be shown to be achieved through analysis or can be achieved through building hardening or other mitigating construction or retrofit.

1) Controlled Perimeter. Measure the standoff distance from the controlled perimeter to the closest point on the building exterior or inhabited portion of the building.

2) Parking and Roadways. Standoff distances for parking and roadways are based on the assumption that there is a controlled perimeter at which larger vehicle bombs will be detected and kept from entering the controlled perimeter. Where there is a controlled perimeter, the standoff distances and explosive weight associated with parking and roadways in Table 5 apply. If there is no controlled perimeter, assume that the larger explosive weights upon which the controlled perimeter standoff distances are based (explosive weight I from Table 5) can access parking and roadways near buildings. Therefore, where there is no controlled perimeter, use standoff distances from parking and roadways according to the distances and the explosive weight associated with controlled perimeters in Table 5. Measure the standoff distance from the closest edge of parking areas and roadways to the closest point on the building exterior or inhabited portion of the building.

Table 5. Standoff Distances for New and Existing Buildings

Location	Building Category	Standoff Distance Requirements			
		Applicable Level of Protection	Conventional Construction Standoff Distance	Minimum Standoff Distance(1)	Applicable Explosive Weight (2)
Controlled Perimeter or Parking and Roadways without a Controlled Perimeter	Billeting and High Occupancy Family Housing	Low	45 m (3) (148 ft.)	25 m (3) (82 ft.)	I
	Primary Gathering Building	Low	45 m (3) (4) (148 ft.)	25 m (3) (4) (82 ft.)	I
	Inhabited Building	Very Low	25 m (3) (82 ft.)	10 m (3) (33 ft.)	I
Parking and Roadways within a Controlled Perimeter	Billeting and High Occupancy Family Housing	Low	25 m (3) (82 ft.)	10 m (3) (33 ft.)	II
	Primary Gathering Building	Low	25 m (3) (4) (82 ft.)	10 m (3) (4) (33 ft.)	II
	Inhabited Building	Very Low	10 m (3) (33 ft.)	10 m (3) (33 ft.)	II
Trash Containers	Billeting and High Occupancy Family Housing	Low	25 m (82 ft.)	10 m (33 ft.)	II
	Primary Gathering Building	Low	25 m (82 ft.)	10 m (33 ft.)	II
	Inhabited Building	Very Low	10 m (33 ft.)	10 m (33 ft.)	II

The minimum standoff distance for all new buildings regardless of hardening or analysis is the minimum standoff distance in Table 5 for both parking areas and roadways. Where possible, move parking and roadways away from existing inhabited buildings (including leased buildings) in accordance with the standoff distances and explosive weights in Table 5. It is recognized, however, that moving existing parking areas and roadways or applying structural retrofits may be impractical; therefore, the following operational options are provided for existing inhabited buildings.

Controlled parking associated with existing inhabited buildings may be allowed to be as close as the minimum standoff distance in Table 5 without hardening or analysis if access control to the parking

Figure 1. Standoff Distance – Controlled Perimeter

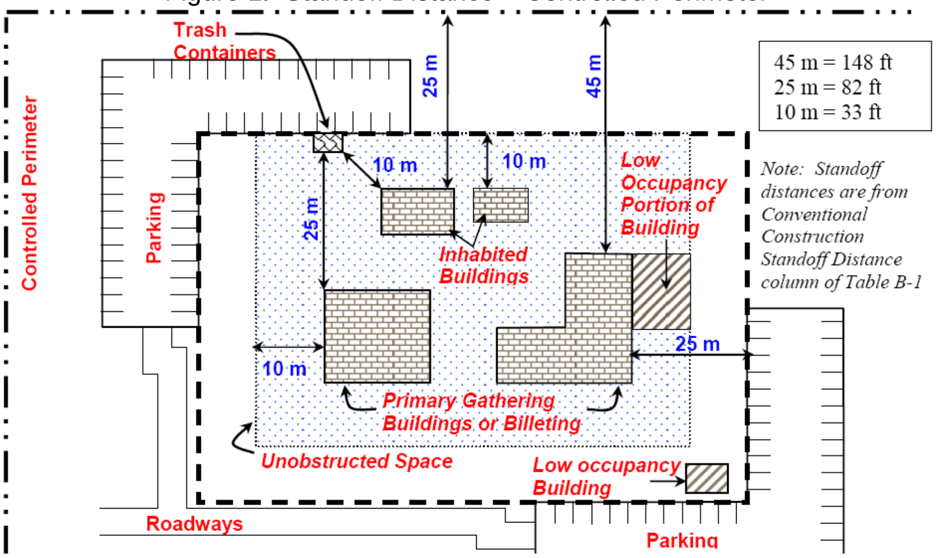


Figure 2. Standoff Distance – No Controlled Perimeter

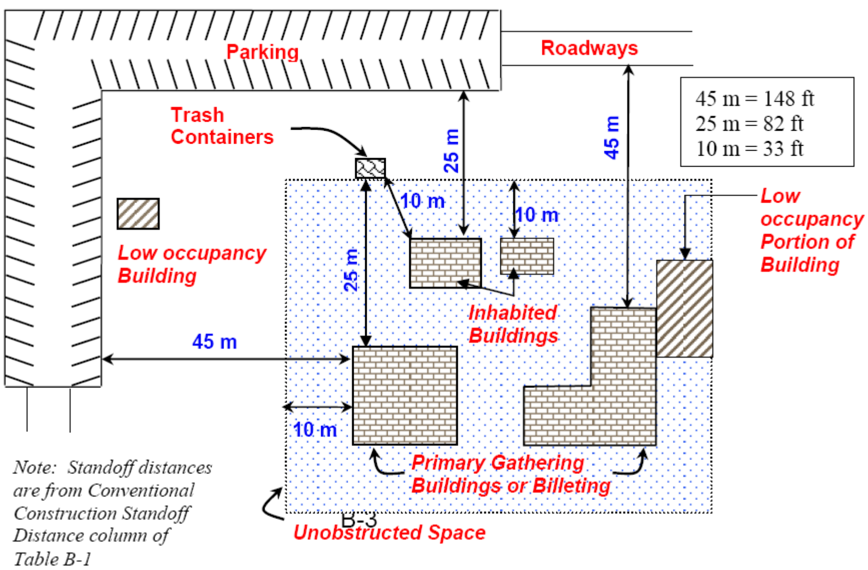


Figure 3. Parking and Roadway Control for Existing Buildings - Controlled Perimeter

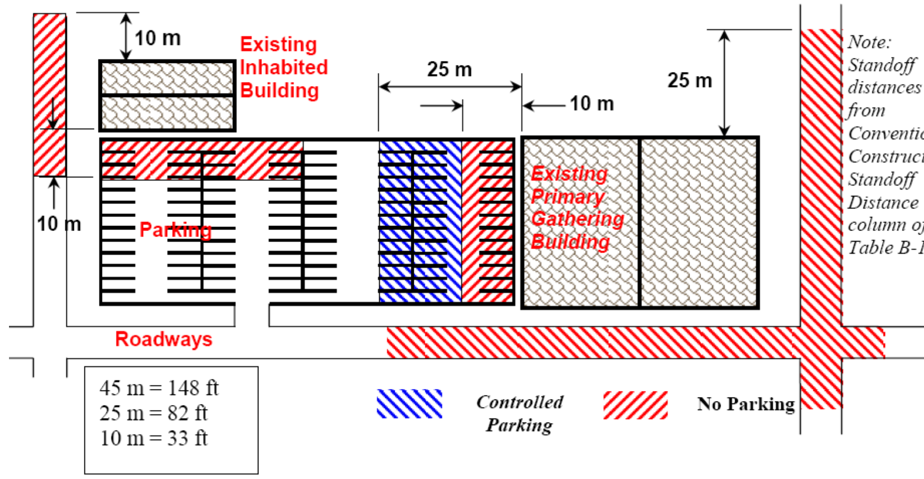
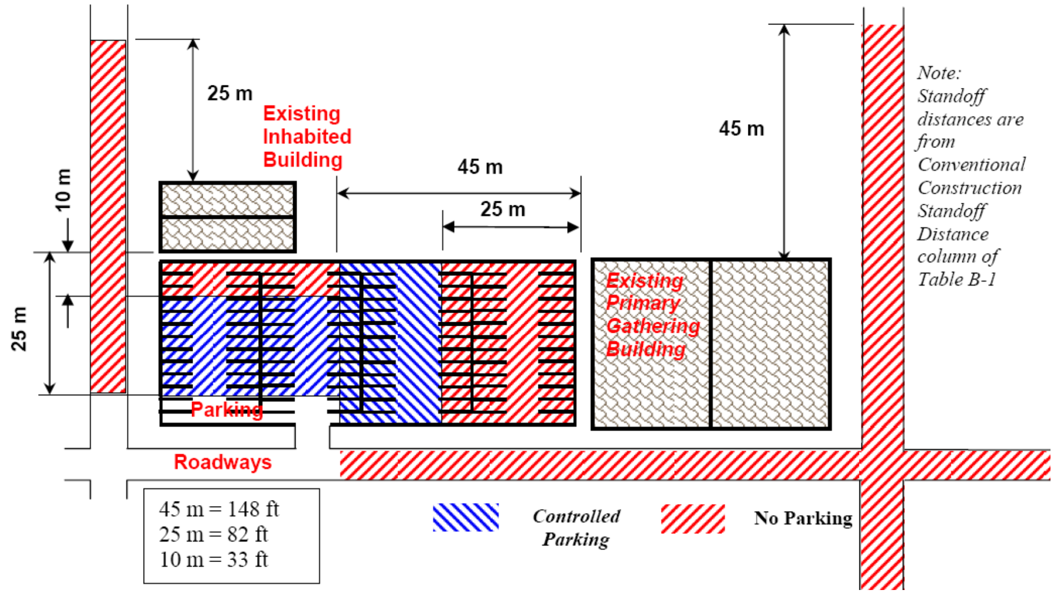


Figure 4. Parking and Roadway Control for Existing Buildings - No Controlled Perimeter



The minimum standoff distance for all new buildings regardless of hardening or analysis is the minimum standoff distance in Table 5 for both parking areas and roadways.

3) Existing Inhabited Buildings. Where possible, move parking and roadways away from existing inhabited buildings (including leased buildings) in accordance with the standoff distances and explosive weights in Table 5. It is recognized, however, that moving existing parking areas and roadways or applying structural retrofits may be impractical; therefore, the following operational options are provided for existing inhabited buildings.

Controlled parking associated with existing inhabited buildings may be allowed to be as close as the minimum standoff distance in Table 5 without hardening or analysis if access control to the parking area is established at the applicable conventional construction standoff distance for parking in Table 5. In cases where the applicable level of protection can be provided (based on hardening or analysis) with a standoff distance between the conventional construction standoff distance and the minimum standoff distance, parking may be allowed as close as the minimum standoff distance in Table 5 if parking is controlled at that lesser applicable standoff distance subject to the following:

Parking Within a Controlled Perimeter. The applicable conventional construction or minimum standoff distance at which access will be controlled will be based on the standoff distances for parking and roadways within a controlled perimeter in Table 5 and illustrated in Figure 3 for the applicable building category.

Parking Without a Controlled Perimeter. The applicable conventional construction or minimum standoff distance at which access will be controlled will be based on the standoff distances for parking and roadways without a controlled perimeter in Table 5 and illustrated in Figure 4 for the applicable building category.

Alternate Situations. Controlled parking may be allowed to be closer to existing inhabited buildings where conditions necessitate it and where it can be shown through analysis that the required level of protection can be provided at a lesser standoff distance or if it can be provided through building hardening or other mitigating measures or retrofits. Allowing any parking closer than the distances established in the paragraphs above should be avoided wherever possible, however.

Parking along roadways is subject to the same standoff considerations as other parking. Ensure that there is no parking on roadways within the required standoff distances (conventional construction or minimum in accordance with Table 5 and illustrated in Figures 3 and 4) along existing roads adjacent to existing buildings covered by these standards.

For high occupancy family housing within a controlled perimeter or where there is access control to the parking area, parking within the required standoff distances may be allowed where designated parking spaces are assigned for specific residents or residences. Do not label assigned parking spaces with names or ranks of the residents, however. Do not encroach upon existing standoff distances where the existing standoff distances are less than the required (conventional construction or minimum in accordance with Table 5) standoff distances. For example, where existing designated parking is only 8 meters from existing family housing, that parking may be retained, but additional parking will not be allowed closer than 8 meters.

4) Parking of Emergency, Command and Operations Support Vehicles. Emergency and command vehicles, as well as operations support vehicles may be parked closer to inhabited buildings than allowed in Table 5 without hardening or analysis if access to them is continuously controlled or as long as they are never removed from a restricted access area, but they may not be parked closer than the distance associated with unobstructed spaces as established in Standard 2. In addition, where standard operation of buildings includes parking emergency vehicles inside them, such as in fire stations, those emergency vehicles may be parked inside the buildings where necessary as long as access to the building is controlled.

5) Parking of Vehicles Undergoing Maintenance. Vehicles undergoing maintenance may be parked inside maintenance buildings closer to inhabited areas of those buildings than allowed in Table 1 while they are undergoing repair where operationally necessary.

6) Adjacent Existing Buildings. Where projects for new and existing buildings designed in accordance with these standards include locating parking, roadways, or trash containers near existing inhabited buildings that are not required to meet these standards, the standoff distances from parking, roadways, and trash containers to the buildings that are not required to comply with these standards should comply with the applicable standoff distances in Table 5. Where those standoff distances are not available, do not allow the parking, roadways, and trash containers to encroach on existing standoff distances to the parking, roadways, and trash containers associated with those existing buildings. For example, if existing parking associated with an existing inhabited building that does not have to comply with these standards is 10 meters from the building, do not allow new parking and roadways associated with a new building closer than

10 meters from the existing building.

7) Parking and Roadway Projects. Where practical, all roadway and parking area projects should comply with the standoff distances from inhabited buildings in Table 5. Where parking or roadways that are within the standoff distances in Table 5 from existing buildings are being constructed, expanded, or relocated, do not allow those parking areas and roadways to encroach on the existing standoff distances of any existing inhabited building. That applies even where such projects are not associated with a building renovation, modification, repair, or restoration requiring compliance with these standards.

8) Trash Containers. Measure the standoff distance from the nearest point of the trash container or trash container enclosure to the closest point on the building exterior or inhabited portion of the building. Where the standoff distance is not available, harden trash enclosures to mitigate the direct blast effects and secondary fragment effects of the explosive on the building if the applicable level of protection can be proven by analysis or testing. Alternatively, if trash containers or enclosures are secured to preclude introduction of objects into them by unauthorized personnel, they may be located closer to the building as long as they do not violate the unobstructed space provisions of Standard 2. Openings in screening materials and gaps between the ground and screens or walls making up an enclosure must not be greater than 150 mm.

b. Standard 2. Unobstructed Space. It is assumed that aggressors will not attempt to place explosive devices in areas near buildings where these explosive devices could be visually detected by building occupants observing the area around the building. Therefore, ensure that obstructions within 10 meters of inhabited buildings or portions thereof do not allow for concealment from observation of explosive devices 150 mm or greater in height. This does not preclude the placement of site furnishings or plantings around buildings. It only requires conditions such that any explosive devices placed in that space would be observable by building occupants. For existing buildings where the standoff distances for parking and roadways have been established at less than 10 meters in accordance with paragraph a.3, the unobstructed space may be reduced to be equivalent to that distance.

1) Electrical and Mechanical Equipment. The preferred location of electrical and mechanical equipment such as transformers, air-cooled condensers, and packaged chillers is outside the unobstructed space or on the roof. However this standard does not preclude placement within the unobstructed space as long the equipment provides no opportunity for concealment of explosive devices.

2) Equipment and Trash Container Enclosures. If walls or other screening devices with more than two sides are placed around trash containers or electrical or mechanical equipment within the unobstructed space, enclose the trash containers or equipment on all four sides and the top. Openings in screening materials and gaps between the ground and screens or walls making up an enclosure will not be greater than 150 mm. Secure any surfaces of the enclosures that can be opened so that unauthorized personnel cannot gain access through them.

c. Standard 3. Drive-Up/Drop-Off Areas. Some facilities require access to areas within the required standoff distance for dropping off or picking up people or loading or unloading packages and other objects. Examples that may require drive-up/drop-off include, but are not limited to, medical facilities, exchanges and commissaries, child care centers, and schools.

1) **Marking.** Where operational or safety considerations require drive-up or drop-off areas or drive-through lanes near buildings, ensure those areas or lanes are clearly defined and marked and that their intended use is clear to prevent parking of vehicles in those areas.

2) **Unattended Vehicles.** Do not allow unattended vehicles in drive-up or drop-off areas or drive-through lanes.

3) **Location.** Do not allow drive-through lanes or drive-up/drop-off to be located under any inhabited portion of a building.

d. Standard 4. Access Roads. Where access roads are necessary for the operation of a building (including those required for fire department access), ensure that access control measures are implemented to prohibit unauthorized vehicles from using access roads within the applicable standoff distances in Table 5.

e. Standard 5. Parking Beneath Buildings or on Rooftops. Eliminate parking beneath inhabited buildings or on rooftops of inhabited buildings. Where very limited real estate makes such parking unavoidable, the following measures must be incorporated into the design for new buildings or mitigating measures must be incorporated into existing buildings to achieve an equivalent level of protection.

1) Access Control. Ensure that access control measures are implemented to prohibit unauthorized personnel and vehicles from entering parking areas.

2) Structural Elements. Ensure that the floors beneath or roofs above inhabited areas and all other adjacent supporting structural elements will not fail from the detonation in the parking area of an explosive equivalent to explosive weight II in Table 5.

11. As-Builts

Upon completion of installing the site features, The Contractor shall submit editable CAD format As-Built drawings. The drawing shall show the final product as it was installed in the field, with the exact dimensions, locations, materials used and any other changes made to the original drawings. Refer to Contract Sections 01335 and 01780A of the specific project for additional details.

US Army Corps
of Engineers
Afghanistan Engineer District

AED Design Requirements:
Vertical Curve Design

Various Locations,
Afghanistan

MARCH 2009

TABLE OF CONTENTS

AED DESIGN REQUIREMENTS
FOR
VERTICAL CURVE DESIGN
VARIOUS LOCATIONS,
AFGHANISTAN

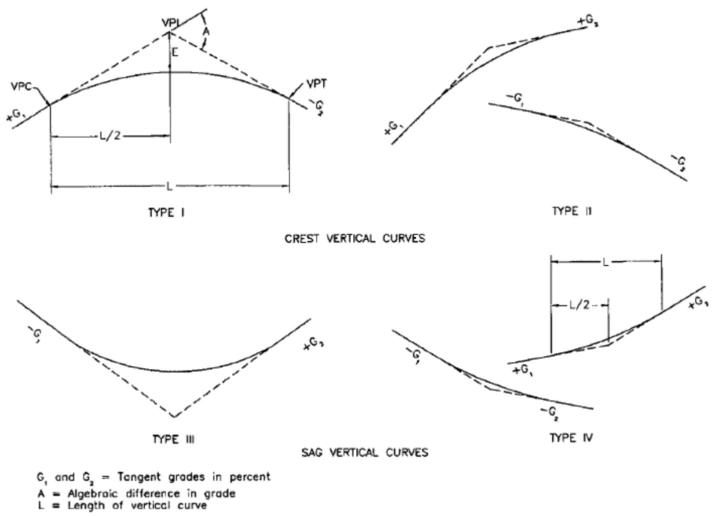
1. General

The purpose of this document is to provide requirements to Contractors for any project requiring the design and construction of vertical curve road design.

2. Vertical Curves

Vertical curves are parabolic curves used to achieve a gradual change between tangent grades (G_1 and G_2) and may be either a crest curve or a sag curve as shown in Exhibit 1.

Exhibit 1. Types of Vertical Curves



3. Crest Vertical Curves Stopping Sight Distance

The major control for safe operation on a crest vertical curve is the sight distance required. At the minimum, the stopping sight distance for the road design speed provided in Table 1 should be provided for all crest vertical curves. Wherever practical, larger stopping sight distances should be used.

Table 1. Stopping Sight Distance

Metric				
Design speed (km/h)	Brake reaction distance (m)	Braking distance on level (m)	Stopping sight distance Calculated (m)	Stopping sight distance Design (m)
20	13.9	4.6	18.5	20
30	20.9	10.3	31.2	35
40	27.8	18.4	46.2	50
50	34.8	28.7	63.5	65
60	41.7	41.3	83.0	85
70	48.7	56.2	104.9	105
80	55.6	73.4	129.0	130
90	62.6	92.9	155.5	160
100	69.5	114.7	184.2	185
110	76.5	138.8	215.3	220
120	83.4	165.2	248.6	250
130	90.4	193.8	284.2	285

Equations 3-1 and 3-2 provide the general equations for calculating the minimum length of crest vertical curves based on the required sight distance and the algebraic difference in grade. Equation 3-1 is to be used if the required sight distance is less than the length of the vertical curve and Equation 3-2 is to be used if the required sight distance is greater than the length of the vertical curve.

Equation 1 $L=(AS)^2/100((2h_1)^{1/2}+(2h_2)^{1/2})^2$ (S<L)
Equation 2 $L=2S-(200(h_1^{1/2}+h_2^{1/2})^2)/A)$ (S>L)

Where:

L=length of vertical curve (m)
S=sight distance (m)
A=algebraic difference in grades (%)
h_1=height of eye above roadway surface (m)
h_2=height of object above roadway surface (m)

When the height of the eye and the height of the object are 1.08 meters and 0.6 meters respectively, as used for stopping sight distance, general equation 3-1 and 3-2 become the requires crest curve length for stopping sight as shown in equations 3-3 and 3-4 respectively.

Equation 3 $L=AS^2/658$ (S<L)
Equation 4 $L=2S-(658/A)$ (S>L)

Where:

L=length of vertical curve (m)
S=sight distance (m)
A=algebraic difference in grades (%)

The rate of vertical curvature (K) is equal to the length of the vertical curve (L) divided by the algebraic difference in the tangent grades (A) in percent (K=L/A). For a given design speed the minimum length of the crest vertical curve for stopping sight distance can be verified by determining the rate of vertical curvature and checking this value against the rate of vertical curvature provided in Table 2 for the design speed of the road. An alternative method to determining the minimum length of a crest vertical curve (L) for stopping sight distance is to multiply the rate of vertical curvature (K) for the design speed of the roadway by the algebraic difference in the tangent grades (A) in percent (L=K*A).

Table 2. Design Controls for Stopping Sight Distance and for Crest Vertical Curves

Metric			
Design speed (km/h)	Stopping sight distance (m)	Rate of vertical curvature, K^a	
		Calculated	Design
20	20	0.6	1
30	35	1.9	2
40	50	3.8	4
50	65	6.4	7
60	85	11.0	11
70	105	16.8	17
80	130	25.7	26
90	160	38.9	39
100	185	52.0	52
110	220	73.6	74
120	250	95.0	95
130	285	123.4	124

Example 1: With a two-lane crest vertical curve with entering and exiting tangent grades of +2.00% and -3.75% respectively and a design speed of 100 km/h, calculate the minimum vertical curve length for stopping sight distance.

From Table 2 with a 100 km/h design speed, the required stopping sight distance is 185 meters and the rate of vertical curvature is 52. Using Equation 3-3 the length of the vertical curve can be determined.

$L=AS^2/658=[(2.00+3.75)*185^2]/658=299.08$ meters.

Since the sight distance (185 meters) is less that the length of the vertical curve (299.08 meters) we can verify that the rate of vertical curvature meets the design requirements.

K=L/A=299.08/(2.00+3.75)=52.01 > 52

The rate of vertical curvature for the 299.08 meter long vertical curve meets or exceeds the required rate of vertical curvature from Table 2 the curve length is satisfactory.

Example 2: With a two-lane crest vertical curve with entering and exiting tangent grades of +8.00% and +4.15% respectively and a design speed of 80 km/h, calculate the minimum vertical curve length.

From Table 2 with an 80 km/h design speed, the required stopping sight distance is 130 meters and the rate of vertical curvature is 26. Using Equation 3 the length of the vertical curve can be determined.

$L=AS^2/658=[(8.00-4.15)*130^2]/658=98.88$ meters.

Since the sight distance (130 meters) is larger that the length of the vertical curve (98.88 meters) calculated with Equation 3 the required length of the vertical curve is calculated with Equation 4.

*L=2S-(658/A)=2*130-(658/(8.00-4.15)=89.09 meters.*

With the calculated sight distance known, we can verify that the rate of vertical curvature meets the design requirements.

K=L/A=89.09/(8.00-4.15)=23.14 < 26

Since the rate of vertical curvature for the 89.09 meter long vertical curve does not meets the required rate of vertical curvature from Table 2 the vertical curve length is determined by the rate of vertical curvature.

L=KA=26(8.00-4.15)=100.10 meters.*

The minimum vertical curve length should be 100.10 meters.

4. Crest Vertical Curve Passing Sight Distance

Design values of crest vertical curves for passing sight distance differ from those for crest stopping sight distance because of the different sight distance and object height criteria. The required passing sight distance for various design speeds can be obtained from Table 3 shown below.

Table 3. Passing Sight Distance for Design of Two-Land Highways

Metric				
Design speed (km/h)	Assumed speeds (km/h)		Passing sight distance (m)	
	Passed vehicle	Passing vehicle	From Exhibit 3-6	Rounded for design
30	29	44	200	200
40	36	51	266	270
50	44	59	341	345
60	51	66	407	410
70	59	74	482	485
80	65	80	538	540
90	73	88	613	615
100	79	94	670	670
110	85	100	727	730
120	90	105	774	775
130	94	109	812	815

The height of the object for passing sight distance increases to 1.08 meters from 0.60 meters for stopping sight distance. These two factors result in crest passing sight distances that are substantially longer than the crest stopping sight distance. When the height of the eye and the height of the object are both 1.08 meters, as used for padding sight distance, Equation 1 and Equation 2 become the requires crest curve length for passing sight as shown in Equation 5 and Equation 6 respectively.

Equation 5 $L=AS^2/864$ $(S<L)$
Equation 6 $L=2S-(864/A)$ $(S>L)$

Where:

L=length of vertical curve (m)
S=sight distance (m)
A=algebraic difference in grades (%)

Again, the rate of vertical curvature (K) is equal to the length of the vertical curve (L) divided by the algebraic difference in the tangent grades (A) in percent (K=L/A). For a given design speed the minimum length of the crest vertical curve for passing sight distance can be verified by determining the rate of vertical curvature and checking this value against the rate of vertical curvature provided in Table 4 for the design speed of the road. An alternative method to determining the minimum length of a crest vertical curve (L) for passing sight distance is to multiply the rate of vertical curvature (K) for the design speed of the roadway by the algebraic difference in the tangent grades (A) in percent (L=K*A).

Table 4. Design Controls for Crest Vertical Curves Based on Passing Sight Distance

Metric		
Design speed (km/h)	Passing sight distance (m)	Rate of vertical curvature, K* design
30	200	46
40	270	84
50	345	138
60	410	195
70	485	272
80	540	338
90	615	438
100	670	520
110	730	617
120	775	695
130	815	769

Example 3: With a two-lane crest vertical curve with entering and exiting tangent grades of +2.00% and -3.75% respectively and a design speed of 100 km/h, calculate the minimum vertical curve length for stopping sight distance.

From Table 4 with a 100 km/h design speed, the required passing sight distance is 670 meters and the rate of vertical curvature is 520. Using Equation 5 the length of the vertical curve can be determined.

$L=AS^2/864=[(2.00+3.75)*670^2]/864=2987.47$ *meters.*

Since the sight distance (670 meters) is less that the length of the vertical curve (2987.47 meters) we can verify that the rate of vertical curvature meets the design requirements.

$K=L/A=2987.47/(2.00+3.75)=519.56 < 520$

Since the rate of vertical curvature for the 2987.47 meter long vertical curve does not meets the required rate of vertical curvature from Table 4 the vertical curve length is determined by the rate of vertical curvature.

L=KA=520(2.00+3.75)=2990.00 meters.*

The minimum vertical curve length should be 2990.00 meters.

5. Sag Vertical Curves

At least four different criteria for establishing the length of sag vertical curves are recognized to some extent. These are headlight sight distance, passenger comfort, drainage control, and general appearance. Of these four criteria, the headlight sight distance is the basis for determining the length of sag vertical curves. Equation 7 and Equation 8 show the general equations for sag vertical curve stopping sight distance based on an eye and object heights of 1.08 meters and 0.6 meters respectively.

Equation 7 $L=AS^2/[200(h_1+S(\tan z))]$ $(S<L)$
Equation 8 $L=2S-[(200(h_1+S(\tan z)))/A]$ $(S>L)$

Where:
L=length of vertical curve (m)
S=sight distance (m)
A=algebraic difference in grades (%)
h_1=height of headlight (m)
z=upward divergence of headlight beam (^{o})

A headlight height of 0.60 meters and a 1-degree upward divergence of the light beam from the longitudinal axis of the vehicle are commonly assumed. Equations 7 and 8 become Equations 9 and 10 respectively, with the known relationship between the length of the sag vertical curve (L) in meters, the algebraic difference in grades (A) in percent and the distance between the vehicle and point where the 1-degree upward angle of the light beam intersects the surface of the roadway (s) in meters.

Equation 9 $L=AS^2/(120+3.5S)$ $(S<L)$
Equation 10 $L=2S-[(120+3.5S)/A]$ $(S>L)$

Where:
L=length of vertical curve (m)
S=sight distance (m)
A=algebraic difference in grades (%)

For overall safety, a sag vertical curve should be long enough that the light beam distance in nearly the same as the stopping sight distance. Accordingly, it is appropriate to use the stopping sight distances for different design speeds as the value of S in the above equations. As in the case of crest vertical curves, it is convenient to express the design control in terms of the rate of vertical curvature (K). Again the rate of vertical curvature is equal to the length of the vertical curve (L) divided by the algebraic difference in the tangent grades (A) in percent (K=L/A). For a given design speed the minimum length of the sag vertical curve can be verified by determining the rate of vertical curvature and checking this value against the rate of vertical curvature provided in Table 5 for the design speed of the road. An alternative method to determining the minimum length of a sag vertical curve (L) is to multiply the rate of vertical curvature (K) for the design speed of the roadway by the algebraic difference in the tangent grades (A) in percent (L=K*A).

Table 5. Design Control for Sag Vertical Curves

Metric			
Design speed (km/h)	Stopping sight distance (m)	Rate of vertical curvature, K^a	
		Calculated	Design
20	20	2.1	3
30	35	5.1	6
40	50	8.5	9
50	65	12.2	13
60	85	17.3	18
70	105	22.6	23
80	130	29.4	30
90	160	37.6	38
100	185	44.6	45
110	220	54.4	55
120	250	62.8	63
130	285	72.7	73

Example 4: With a two-lane sag vertical curve with entering and exiting tangent grades of -2.50% and +4.00% respectively and a design speed of 100 km/h, calculate the minimum vertical curve length.

From Table 5 with a 100 km/h design speed, the required stopping sight distance is 185 meters and the rate of vertical curvature is 45. Using Equation 9 the length of the vertical curve can be determined.

$L=AS^2/(120+3.5S)=[(2.50+4.00)*185^2]/(120+(3.5*185)=289.85$ meters.

Since the sight distance (185 meters) is less that the length of the vertical curve (289.85 meters) we can verify that the rate of vertical curvature meets the design requirements.

$K=L/A=289.85/(2.50+4.00)=44.59 < 52$

Since the rate of vertical curvature for the 289.85 meter long vertical curve does not meets the required rate of vertical curvature from Table 5 the vertical curve length is determined by the rate of vertical curvature.

$L=KA=45*(2.50+4.00)=292.50$ meters.

The minimum vertical curve length should be 292.50 meters.

Example 5: With a two-lane sag vertical curve with entering and exiting tangent grades of -8.00% and -5.30% respectively and a design speed of 80 km/h, calculate the minimum vertical curve length.

From Table 5 with an 80 km/h design speed, the required stopping sight distance is 130 meters and the rate of vertical curvature is 30. Using Equation 9 the length of the vertical curve can be determined.

$L=AS^2/(120+(3.5S)=[(8.00-5.30))*130^2]/(120+(3.5*130)=79.36$.

Since the sight distance (130 meters) is larger that the length of the vertical curve (79.36 meters) calculated with Equation 9 the required length of the vertical curve is calculated with Equation 3-10.

$L=2S-[(120+(3.5S))/A]=2*130-[(120+(3.5*130))/(8.00-5.30)]=47.03$ meters.

With the calculated sight distance known, we can verify that the rate of vertical curvature meets the design requirements.

$K=L/A=47.03/(8.00-5.30))=17.42 < 30$

Since the rate of vertical curvature for the 47.03 meter long vertical curve does not meets the required rate of vertical curvature from Table 5 the vertical curve length is determined by the rate of vertical curvature.

L=KA=30(8.00-5.30)=81.00 meters.*

The minimum vertical curve length should be 81.00 meters.

6. Design Considerations

The following design considerations, in addition to the criteria listed above, should be reviewed for all horizontal curves to endure a safe design.

The "roller-coaster" type of profile should be avoided. Such profiles generally occur on relatively straight horizontal alignments where the roadway profile closely follows a rolling natural ground line. This type of profile is avoided by the use of horizontal curves or by more gradual grades.

A "broken-back" gradeline (two vertical curves in the same direction separated by a short tangent section) should be avoided, particularly in sags. "Broken-back" gradelines can be avoided by changing the grade lines or the lengths of the vertical curves.

Sag vertical curves should be avoided in cut sections unless adequate drainage can be provided.

7. As-Builts

Upon completion of construction of the roadway, The Contractor shall submit editable CAD format As-Built drawings. The drawing shall show the final product as it was installed in the field, with the exact dimensions, locations, materials used and any other changes made to the original drawings. Refer to Contract Sections 01335 and 01780A of the specific project for additional details.

US Army Corps
of Engineers
Afghanistan Engineer District

AED Design Requirements:
Water Tanks & System Distribution

Various Locations,
Afghanistan

SEPTEMBER 2009 Version 1.2

TABLE OF CONTENTS

AED DESIGN REQUIREMENTS
FOR
WATER TANKS
VARIOUS LOCATIONS,
AFGHANISTAN

Figures

Tables

Appendices

1. General

The purpose of this document is to provide water storage and distribution design requirements to Contractors for projects at USACE-AED projects. This is a summary of design and testing requirements for USACE-AED construction. Design procedures and examples can be found in the documents listed in the reference section.

2. Water Tanks & Distribution Systems

Water tanks may be required where a new water distribution system is proposed or as an upgrade to an existing water distribution system. Reference 1 provides design guidance specifically for water storage. Storage capacity of the water tank should meet peak flow requirements, equalize system pressures, and provide emergency water supply. The water supply system must provide flows of water sufficient quantity to meet all points of demand in the distribution system. To do so, pressure levels within the distribution system must be high enough to provide suitable pressure, and water distribution mains must be large enough to carry these flows. Reference 2 provides design guidance specifically for water distribution. Water storage facilities are constructed within a distribution network to meet the peak flow requirements exerted on the system and to provide emergency storage. Water supply systems must be designed to satisfy maximum anticipated water demands. The peak demands usually occur on hot, dry, and summer days when larger than normal amounts of water are used for irrigation and washing vehicles and equipment. In addition, most industrial processes, especially those requiring supplies of cooling water, experience greater evaporation on hot days, thus requiring more water. The necessary storage can be provided in elevated, ground, or a combination of both types of storage. Requirements for storage are discussed further in Section 7.

3. Water Distribution System Requirements

The Contractor shall install water distribution mains, branches, laterals, lines and service connections to include all pipe, valves, fittings and appurtenances, and pipe thrust restraint. Exterior water line construction shall include service to all buildings as described in the contract Scope of Work Section 01010. Distribution system designs must consider system operating pressure range; the pipe size and material, including joint construction and fittings; disinfection, and construction testing.

Pipe material is of importance from the standpoint of constructability, service life, and ease of maintenance. In USACE-AED projects PVC Schedule 80 pipe is the preferred material. This pipe has well documented experience in previous projects; it has superior strength and durability of over time to other thermoplastic pipe material, it is easily repaired, and materials including fittings are readily available in Afghanistan.

a) System Pressure Requirements.

Distribution systems shall provide system pressures that are neither too low 70 KPa (10 psi) for operating plumbing fixtures nor too high such that they are damaged. Pressure are measured at the building service connection; therefore pressures at this location must be generally in excess of 207 KPa (30 psi) for one to two story buildings considering internal pipe friction head losses.

1) Minimum pressures. Water distribution system, including pumping facilities and storage tanks or reservoirs, should be designed so that water pressures of at least 275 KPa (40 psi) at ground level will be maintained at all points in the system, including the highest ground elevations in the service area. Minimum pressures of 207 KPa (30 psi) under peak fixture flow conditions can be tolerated at the building farthest from the water source (tank or booster pump) as long as all peak fixture flow requirements can be satisfied at all locations. During firefighting flows, water pressures should not fall below 138 KPa (20 psi) at the hydrants, in new systems. Fire fighting capability is provided using hose streams at only a limited number of projects as specified in the contract.

2) Maximum pressure. Maximum water pressures in distribution mains and service lines should not normally exceed 517 KPa (75 psi) at ground elevation. Higher pressures require pressure reducing valves on feeder mains or individual service lines to restrict maximum service pressures to 517 KPa (75 psi).

3) Multiple pressure levels. If an extensive area has pressures higher than 517 KPa (75 psi) lower than 275 KPa (40 psi) under a single pressure level zone, it may be appropriate to divide the system into two or more separate zones, each having different pressure levels. Within each level, pressures within the distribution system should range from 275 KPa to 517 KPa (40 to 75 psi) at ground elevation.

b) Pipe Size and Material.

Pipe diameter is related to the design of adequate system pressure because the larger the pipe diameter the lower the friction head loss and therefore more service pressure availability. In addition, USACE water system planning technical criteria recognize good engineering judgment includes providing some safety factor in the design. The contract's minimum size is desirable for future growth that the contractor cannot account for in their analysis, and their water distribution analysis cannot be verified until after construction; and even then at great effort. Water system models do not account for all system losses and operational circumstances. Furthermore, sizing water mains for the bare minimum when new means the system will be under sized as it ages in the future when pipe leaks and scale occur and other components such as valves and flow meters deteriorate. Unnecessarily small water main pipe diameters increase the booster pump horsepower requirements, energy costs for operation, and ultimately make water system sustainability more of an issue.

1) Pipe diameters and velocities. The minimum pipe diameter in the distribution system shall be 100mm (4 inch). The maximum velocity shall be 5 feet per second (1.5 meters per second) at 150% of the fixture unit flow or 2 times the average daily flow (8-hour basis), whichever is greater. The Contractor shall provide a water distribution system described as follows: Pipe diameters used in the network shall be 100mm or greater, as required to maintain proper system velocities and pressures between 275 KPa (40 psi) and 275 KPa (75 psi). Pipes for building service connections may be smaller diameter and shall be sized based on the fixture unit flows required for each building.

2) Pipe materials. The Contractor shall provide pipe of adequate strength, durability and be corrosion resistant with no adverse effect on water quality. Water distribution pipe material shall be PVC or Ductile Iron (DI). Ductile iron pipe shall conform to AWWA C104. DI fittings shall be suitable for 1.03MPa (150psi) pressure unless otherwise specified. Fittings for mechanical joint pipe shall conform to AWWA C110. The exterior surface of the pipe must be corrosion resistant. If DI pipe is installed underground pipe shall be encased with polyethylene in accordance with AWWA C105. Fittings and specials shall be cement mortar lined (standard thickness) in accordance with C104. Fittings for use with push-on joint pipe shall conform to AWWA C110 and C111. Polyvinyl Chloride (PVC) pipe shall conform to ASTM D 1785. Plastic pipe coupling and fittings shall be manufactured of material conforming to ASTM D 1784, Class 12454B. PVC screw joint shall be in accordance with ASTM D 1785 Schedules 80 and 120. PVC pipe couplings and fittings shall be manufactured of material conforming to ASTM D 1784, Class 12454B. Pipe for building service, less than 80mm (3 inch) may be screw joint and shall conform to dimensional requirements of ASTM D schedule 80. Elastomeric gasket-joint, shall conform to dimensional requirements of ASTM D 1785 Schedule 80, All pipe and joints shall be capable of 1.03 MPa (150psi) working pressure and 1.38 MPa (200psi) hydrostatic test pressure.

The only time HDPE and PVCu will be allowed in any AED project, including facility designs, water transmission pipelines, sewer force mains and non pressure pipe applications such as storm water or gravity sewers, is through an approved variation request submitted in accordance with Section 01335 of the contract. The variation request shall be submitted to

AED Engineering for approval. PVCu pipe material shall be specifically manufactured in accordance with specifications and labeled as Schedule 80 pipe according to ASTM D1785 specifications. Variation request shall include (among other items stated in Section 01335 3.6.4) the pipe material cell classification used in the product, the standard dimension ratio (SDR), the type of jointing being used, and the proposed use for the product (such as well casing or water distribution pipe lines). To be considered for a variation, HDPE pipe shall conform to Deutsche Institut fur Normung (DIN) 8074 Polyethylene Pipe - Dimensions and DIN 8075 Polyethylene Pipes – General quality requirements and testing (August 1999). Installation shall be as specified in AWAA M55 PE Pipe – Design and Installation. In the absence of products or installation methods not meeting these standards, the contractor shall provide documentation using the variations process in the contract section 01335 for approval prior to installation.

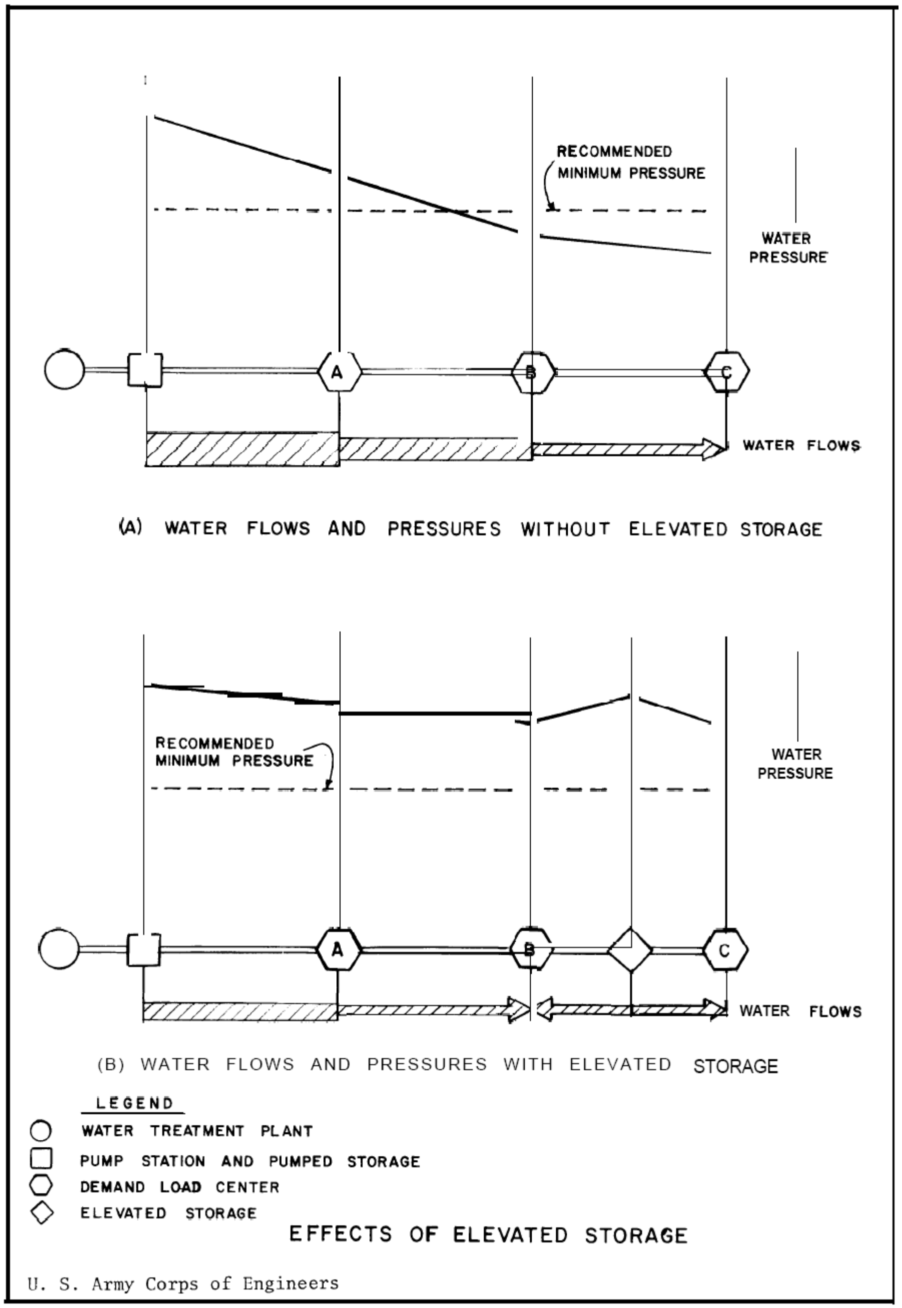
c) Pressure Provisions.

1) Elevated storage. Within the distribution system, elevated storage permits the well pumping to a tank to operate at uniform rates and without frequent start/stop cycles. The usefulness of elevated storage is shown in Figure 1. The system illustrated in Figure 1 (A) (without elevated storage) requires storage at the plant sufficient to provide for system demand rates in excess of the plant production rate, assuming the plant is operated at a uniform rate. The pump station forces water into the service main, through which it is carried to three load areas: A, B, and C. Since all loads on the system are met without the use of elevated storage, the pump station must be capable of supplying the peak rates of water use to Areas A, B, and C, simultaneously, while maintaining the water pressure to Area C at a sufficient level. The minimum recommended pressure in the distribution system under non peak nonemergency flow conditions is 275 KPa (40 psi). Figure l (B) assumes the construction of an elevated storage tank on the service main between Areas B and C, with peak loads in Area C and part of the peak load in Area B being satisfied from this tank. The elevation of the tank ensures adequate pressures within the system. The storage in the tank is replenished when water demands are low and the well (or pump station in the figure) can fill the tank while still meeting all flow and pressure requirements in the system. The Figure 1 (B) arrangement reduces required capacity of the booster pumps.

2) Booster pump pressure. Booster pump stations are sited downstream of ground level water tanks in order to provide the system operating pressure. Therefore they operate at the position shown in Figure 1A except that the pump total dynamic head (TDH) must be sufficient at the pump discharge to elevate the system pressure above the minimum pressure requirement at every location in the water system. Therefore compared to systems that have elevated storage, there is less uniformity in the system pressure and generally greater energy used to maintain system pressure than in a centrally located water tank. Booster pumping applications within AED shall have either a bladder style expansion or a hydropneumatic tank.

3) Most elevated storage tanks “float” on the distribution system. That is, the elevated tank is hydraulically connected to the distribution system, and the volume of water in the tank tends to maintain system pressures at a uniform level. When water use is high and booster pumping facilities cannot maintain adequate pressures, water is discharged from elevated tanks. Conversely, when water use is low, the booster pumps, which operate within a reasonably uniform head-capacity range, supply excess water to the system and the elevated storage is refilled. This condition is not normally encountered in designs in Afghanistan since it assumes that the booster pumps draw water from a source other than the elevated tank being filled.

Figure 1. Effects of Elevated Storage



d) Valves and Thrust Restraint

1) System Isolation Valves. Valves (Gate valves w/box) shall be placed at all pipe network tees and cross intersections, and the number of valves shall be one less than the number of lines leading into and away from the intersection. For isolation purposes, valves shall be spaced not to exceed 3600 mm (12 feet) from tees or crosses. Gate valves shall be in accordance with AWWA C 500 and/or C509. Butterfly valves (rubber seated) shall be in accordance with C504. The valves and valve boxes shall be constructed to allow a

normal valve key to be readily used to open or close the valve. Provide traffic-rated valve boxes and concrete pad, 1 meter (3'-4") square, for all valve boxes.

2) Air Release Valves. Air release valves are required to evacuate air from the main high points in the line when it is filled with water, and to allow the discharge of air accumulated under pressure. Vacuum relief valves are needed to permit air to enter a line when it is being emptied of water or subjected to vacuum. Contractor shall submit manufacturer's data for properly sized combination air and vacuum release valves and determine their locations on the distribution system subject to review and approval of the Contracting Officer.

3) Blow-off Valves. The Contractor shall provide 40-50mm (1-5/8" – 2") blow-off valves at ends of dead end mains. Valves should be installed at low points in the mains where the flushing water can be readily discharged to natural or manmade drainage ditches, swales or other.

Thrust restraint is required for pipe diameters 100 mm in diameter or larger. Restraint may be achieved by the type of joint system selected or by thrust blocking.

4. Types of Storage

Required storage capacity at military installations is met by use of elevated or ground storage. Examples of standard water tank construction at USACE-AED projects are shown in Figures 2 and 3. Elevated storage, feeds the water distribution system by gravity flow. Storage which must be pumped into the system is generally in ground storage tanks. Clear-well storage, which is usually part of a water treatment plant, is not included in computing storage unless sufficient firm pumping capacity is provided to assure that the storage can be utilized under emergency conditions, and then only to the extent of storage in excess of the 24-hour requirements of the treatment plant. Clear-well storage is used to supply peak water demand rates in excess of the production rate, and to provide a reservoir for plant use, filter backwash supply, and water supply to the system for short periods when plant production is stopped because of failure or replacement of some component or unit of treatment.

a) Ground Storage. Ground storage is usually located remote from the treatment plant (if one exists) but within the distribution system. Ground storage is used to reduce well or treatment plant peak production rates and also as a source of supply for re-pumping to a higher pressure level. Such storage for re-pumping is common in distribution systems covering a large area, because the outlying service areas are beyond the range of the primary pumping facilities. An example of a ground level reinforced concrete tank at USACE-AED projects is given in Figure 2. Ground level water tanks may be either reinforced concrete or steel construction. Ground storage tanks or reservoirs, below ground, partially below ground, or constructed above ground level in the distribution system, may be accompanied by pump stations if not built at elevations providing the required system pressure by gravity. There are a few projects in the USACE-AED project inventory that have partially below ground-level tanks. However, if the terrain permits, the design location of ground tanks at an elevation sufficient for gravity flow is preferred. Concrete reservoirs are generally built no deeper than 6.1-7.6 meters (20-25 feet) below ground surface. If rock is present, it is usually economical to construct the storage facility above the rock level. In a single pressure level system, ground storage tanks should be located in the areas having the lowest system pressures during periods of high water use. In multiple pressure level systems, ground storage tanks are usually located at the interface between pressure zones with water from the lower pressure zones filling the tanks and being passed to higher pressure zones through adjacent pump stations.

b) Elevated Storage. Elevated storage is provided within the distribution system to supply peak demand flow rates and equalize system pressures. In general, elevated storage is more effective and economical than ground storage because of the reduced pumping requirements, and the storage can also serve as a source of emergency supply since system pressure requirements can still be met temporarily when pumps are out of service. The most common types of elevated storage are elevated steel tanks, and standpipes. An example of an elevated steel tank at USACE-AED projects is given in Figure 3. Elevated storage tanks should be located in the areas having the lowest system pressures during intervals of high water use to be effective in maintaining adequate system pressures and flows during periods of peak water demand. These are those of greatest water demand or those farthest from pump stations. Elevated tanks are generally located at some distance from the pump station serving a distribution pressure level, but not outside the boundaries of the service area, unless the facility can be placed on a nearby hill. Additional considerations for locating elevated storage are conditions of terrain, suitability of subsurface soil and/or rock for foundation purposes, and hazards to low-flying aircraft. Elevated tanks are built on the highest available ground, up to static pressures of 517 KPa (75 psi) in the system, so as to minimize the required construction cost and heights.

Figure 2. Ground Level Storage Tank

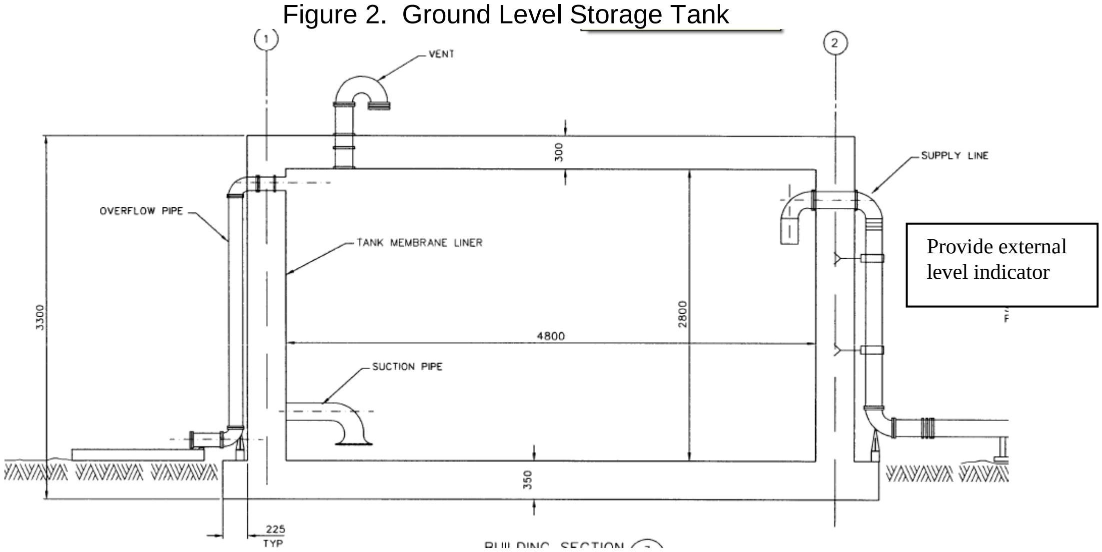
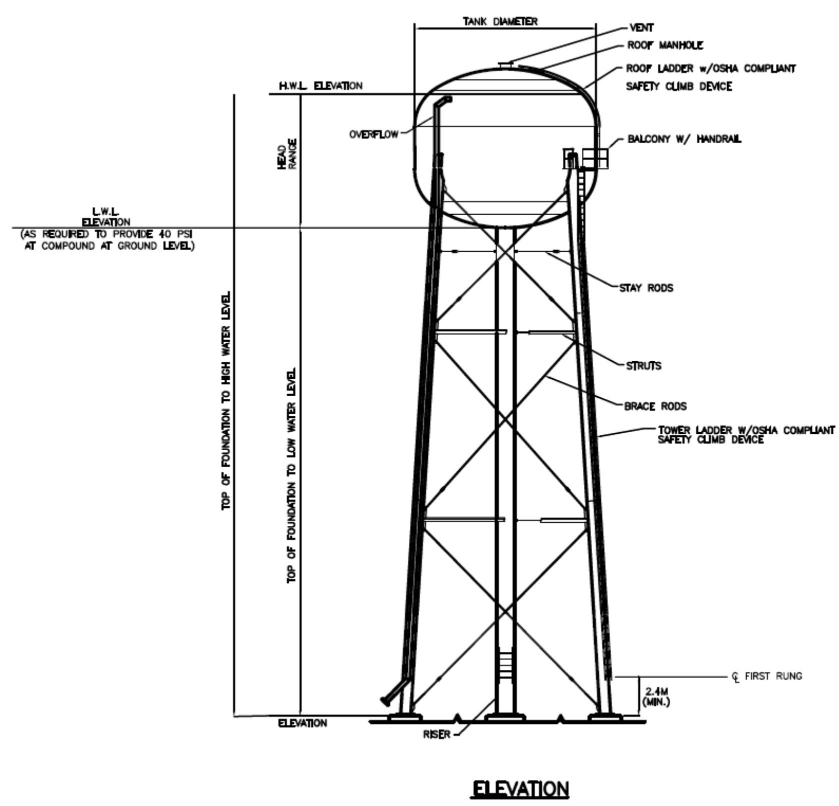


Figure 3. Standard Elevated Storage Tank



5. Water Requirements

The design of the water distribution systems shall be sized to provide flow and discharge based on a fixture unit basis or the basis of the average daily demand multiplied by the capacity factor, whichever is the greater. This flow is to be used to design facilities on an installation and is called the Average Daily Flow (ADF). The ADF is used to design installation water and wastewater systems and is calculated as the effective population x ADD x CF. These terms are defined below.

a) Domestic Requirements. The daily, per capita water requirements/allowances used in the design of facilities in Afghanistan are derived from Table 1 below unless stated differently in the contract technical requirements. These allowances do not include special purpose water uses, such as industrial, aircraft-wash, air-conditioning, irrigation, or extra water demands at desert stations. The term Average Daily Demand (ADD) and Domestic Water Allowance are terms used to quantify the volume of water used by an average individual at the facility being designed. These terms DO NOT include a capacity factor CF, described below in Table 2. If an individual ADD or water allowance is defined in the Scope of Work or Technical Requirements, that value in the Contract shall be used for design calculations and not the value provided in Table 1. In either case, a capacity factor (see Paragraph 6, Table 2) should be applied when making design and sizing calculations.

b) Fire-Flow Requirements. Fire flow demand will generally not be included in the sizing of water storage facilities except where specifically stated in the contract technical requirements. In those cases a stand-alone water storage tank may be required in the technical requirements. In this case only, the system must be capable of supplying the fire flow specified plus any other demand that cannot be reduced during the fire period at the required residual pressure and for the required duration. The requirements of each system must be analyzed to determine whether the capacity of the system is fixed by the domestic requirements, by the fire demands, or by a combination of both. Where fire-flow demands are relatively high, or required for long duration, and population and/or industrial use is relatively low, the total required capacity will be determined by the prevailing fire demand. In some exceptional cases, this may warrant consideration of a special water system for fire purposes, separate, in part or in whole, from the domestic system. However, such separate systems will be appropriate only under exceptional circumstances and, in general, are to be avoided.

Table 1. Domestic Water Allowance/Average Daily Demand (ADD)

	Liters/Capita/Day (Gallons/Capita/Day)	
	Enduring Base	Contingency Base
U.S. Forces	285 (75)	190 (50)
Coalition Forces	190 (50)	115 (30)
ANA	155 (41)	95 (25)
ANP	155 (41)	95 (25)
Dining Facility[1]	# of meals x rate/meal	# of meals x rate/meal
Wash Racks[2]	# of vehicles x rate/vehicle	# of vehicles x rate/vehicle
Vehicle Maintenance	20 (5)/vehicle	20 (5)/vehicle

Notes:
[1]Rate/meal shall be 2 liters (0.5 gallons) for breakfast, 4 liters (1 gallon) for lunch and 8 liters (2 gallons) for dinner.
[2]Rate/vehicle shall be 75 liters (20 gallons) for cars small trucks, 115 liters (30 gallons) for large trucks and 190 liters (50 gallons) for aircraft.

c) Irrigation Requirements. The allowances indicated in Table 1 include water for limited watering for planted and grassed areas. However, these allowances do not include major lawn or other irrigation uses. Lawn irrigation provisions for facilities, such as family quarters and temporary structures, in all regions will be limited to hose bibs on the outside of buildings and risers for hose connections. Where substantial irrigation is deemed necessary and water is available, underground sprinkler systems may be considered. Where irrigation requirements are justified in arid or semi-arid regions, such irrigation quantities will be included as an industrial water requirement and not as a domestic requirement.

6. Capacity of Water Supply System

In order to account for fluctuations in water use at facilities, a safety or capacity factor (CF) is introduced into the design calculations. Capacity factors, as a function of "Effective Population" are shown in Table 2, as follows:

Table 2. Capacity Factors (CF)

Effective Population	Capacity Factor
5,000 or less	1.50
10,000	1.25
20,000	1.15
30,000	1.10
40,000	1.05
50,000 or more	1.00

Per the UFCs, the "Capacity Factor" will be used in planning water supplies for all projects, including general hospitals. The proper "Capacity Factor" as given in Table 2 is multiplied by the "Effective Population" to obtain the "Design Population." For example, a facility with a planned (effective population) of 93 persons would be considered to have a design population of 93 x 1.5 = 140. Capacity factors and Design Populations will be used in calculating the ADF, capacity of the supply works, supply lines, treatment works, principal feeder mains and storage reservoirs. Taking this into account, the required storage volume for a facility with 93 assigned personnel, would be 93 persons multiplied by the ADD of 155 liters per person per day (Table 1), multiplied by the capacity factor (CF) of 1.5 (Table 2), which means, 93 x 155 x 1.5 = 21.62 cubic meters (5,710 gallons). It should be stressed again that ADD values provided in the contract documents shall be used when given, but that capacity factors must be applied unless specifically excluded in those contract documents. When necessary, arithmetic interpolation should be used to determine the appropriate Capacity Factor for intermediate project population. (For example, for an "Effective Population" of 7,200 in interpolation, obtain a "Capacity Factor" of 1.39.) Capacity factors will NOT be used for hotels and similar structures that are acquired or rented and troop housing. Capacity factors will NOT be applied to fire flows, irrigation requirements, or industrial demands.

7. Storage Requirements

The amount of water storage provided will conform to the requirements set forth herein. Storage requirements for MILCON projects are explained in Reference 1. Requirements for ANP and ANA vary, but are typically a minimum provision of one (1) day average daily flow which is the ADD multiplied by the effective population (c) multiplied by the capacity factor (ADD x c x CF). In all projects, the storage requirements stated in the contract technical requirements (Section 01015) shall be multiplied by the capacity factor unless specifically stated otherwise.

In general for MILCON projects, total storage capacity, including elevated and ground storage, will be provided in an amount not less than the greatest of the following items.

Item 1: One hundred percent (100%) of the ADF (ADD x c x CF) plus all industrial requirements. This will provide minimum operational storage needed to balance average daily peak demands on the system and to provide an emergency supply to accommodate essential water needs during minor supply outages of up to a one-day duration. For the purposes of this item, essential water needs do not include the fire demand.

Item 2: The fire demand is the required fire flow needed to fight a fire in the facility (including water required to support fire suppression systems) which constitutes the largest requirement for any facility served by the water supply system; plus 50 percent of the average domestic demand rate plus any industrial or other demands that cannot be reduced during a fire period. This amount will be reduced by the amount of water available under emergency conditions during the period of the fire. The fire demand quantity must be maintained in storage for fire protection at all times except following a fire fighting operation when the fire demand quantity would be depleted. It is recognized that during daily periods of peak consumption due to seasonal demands, the amount of water in storage will be less than full storage capacity; however, conservation methods will be instituted to prevent drawdown of water in storage below the fire demand quantity. Fire demand flow may not be included in the project; check the Section 01015 technical requirements for provision of fire flow demand.

8. Amount of Water Available Under Emergency Conditions.

Where the water supply is obtained from wells, all of which are equipped with standby power and located within the distribution system, the emergency supply will be considered as the quantity available from all but one of the wells. Where one well has a capacity greater than the others, that one will be assumed out of service. Where only 50 percent of the wells have standby power, the emergency supply will reconsidered as the quantity available from the wells having standby power.

9. Design and Construction of Water Storage Facilities

All treated water reservoirs must be covered to prevent contamination by dust, birds, leaves, and insects. These covers will be, insofar as possible, watertight at all locations except vent openings. Special attention should be directed toward making all doors and manholes watertight. Vent openings must be protected to prevent the entry of birds and insects; and vent screens should be kept free of ice or debris so that air can enter or leave the reservoir area as temperature and water levels vary. All overflows or other drain lines must be designed so as to eliminate the possibility of flood waters or other contamination entering the reservoir. Reservoir covers also protect the stored water from sunlight, thus inhibiting the growth of algae. Further prevention of algae growth or bacterial contamination, due to the depletion of the chlorine residual, can be obtained by maintaining sufficient flow through the reservoir so that water in the reservoir does not become stagnant. Minimal flows through the reservoir also help to prevent ice buildup during cold periods.

All storage tanks will be provided with external level indicators to prevent overflows during filling. Depending upon the contract requirements, either level controls to the well pump motor control panel or altitude valves shall be used to control overflows of the water tank. If the contract technical requirements specifically state that altitude valves shall be used, these altitude valves will be installed in concrete pits having provision for draining either by gravity or pumping. Water tank drains and overflow piping will not be connected to sanitary sewers. Every precaution will be taken to prevent the collection of water from any source in valve pits.

Storage measurements are used for monitoring, inventory, and system controls. Elevated and ground storage measurements will be made by either external mechanical level indicators or pressure sensitive instruments directly connected by static pressure lines at points of no flow. Underground storage measurements will be made by air bubbler back pressure sensitive instruments or by float actuated instruments. The direct pressure measurements of elevated tanks will be suppressed to readout only the water depth in the elevated bowl. High and low level pressure sensitive switches will be used for alarm status monitoring and for pump cut-off controls. Intermediate level switches, pressure or float actuated, will be used for normal pump controls. High storage level will initiate the shut-down of supply pumping units and actuation of an overflow alarm in that order. Low storage level will initiate startup of supply pumping or well pumping units or distribution pumping unit shutdown.

Potable water storage facilities, associated piping, and ancillary equipment must be disinfected before use. Disinfection will be accomplished following procedures and requirements of the contract specifications. In no event will any of the above equipment or facilities be placed in service prior to verification by the supporting medical authority, by bacteriological tests, that disinfection has been accomplished.

Leakage tests shall be conducted prior to acceptance of the completed water system. Procedures for storage tank leakage testing are contained in Appendix B.

10. Water Distribution System Design Capacities and Requirements

The sizing and location of water mains, booster pump stations, and elevated storage facilities are dependent upon hydraulic analyses of the water distribution system.

Features of the water system shall be sized to provide flow or storage capacity as follows:

- Water Well Pump Capacity - Capacity and total dynamic head (TDH) shall be based on an adjusted ADF (ADD, times the population, times the capacity factor over a 16 hour period).

- Water Tanks - Capacity shall be based on ADF (ADD x c x CF). (NOTE: If a minimum volume of storage is provided in the contract documents, that value is to be taken as the average daily storage capacity and will be multiplied by the capacity factor to determine the actual required storage volume for the facility.)

- Booster Pumps – For installations with fewer than 400 persons, the capacity of each pump shall be 50% of the installation wide, total fixture unit flow. For installations with greater than 400 persons, the capacity of each pump shall be 50% of the installation wide, total fixture unit flow or 2 times the adjusted average daily flow (16 hour basis), whichever is greater. Provide three identical pumps. Each pump shall be sized to deliver the calculated capacity. Pumps shall automatically alternate to distribute wear. Provide variable frequency drives or pressure controller and automatically turn pumps on and off based on flow demand and system pressures. Unless stated otherwise in the contract documents, the total dynamic head (TDH) of the booster pumps shall be calculated to maintain a minimum, system pressure of 40 psi at all points in the distribution system assuming 2 pumps are operating at the specified flow. Either a bladder style expansion tank or a hydro-pneumatic tank shall be supplied when booster pumps are used in the water system.

- Hydro-pneumatic tanks – Volume and pressure regulation to maintain a pressure range provided in the technical requirements based on a rate equal to the ADF (ADD x c x CF).

- Water Mains – Diameter based on the installation fixture unit flow or two times the ADF (ADD x c x CF) and velocity requirements per this guide unless a minimum diameter is specified which is adequate to provide flow and meet the specified maximum velocity. The flow through the system shall be distributed on the basis of fixture unit flow in each the buildings serviced or per contract

- Water Service Lines - Diameter based on fixture units of the building serviced or per contract

Technical requirements for water distribution systems design are provided in Reference 2 and may also be summarized in the contract technical requirements. Other AED Design Guides discuss the sizing of booster pumps and hydro pneumatic tanks (see references). For all but single pipe water transmission lines to one or two individual buildings, water distribution computer programs should be used to evaluate the water system hydraulic design. An excellent program developed by the U.S. Environmental Protection Agency is available on the internet at the location shown in Reference 5.

Water distribution models are based on assumed values for many parameters, simplifications of the actual physical system, and unknowns concerning future demands. They are therefore inexact representation of the real world and must be used with conservatism applied to the results. Water system models do not account for all system losses and operational circumstances. For example partially closed gate valves, fitting losses, and the growth of scale in the pipes due to the very hard water quality in Afghanistan are not considered in water models. Furthermore, sizing water mains for the bare minimum when it is new means the system will be under sized as it ages in the future when pipe leaks occur and other components deteriorate.

Appendix A contains an example of the information to be provided for USACE-AED project design analysis reports. The critical information that shall be shown is listed below:

1. Network model representation – a drawing (with accompanying graphic scale) showing the valves, fittings, water tanks, pumps, demand nodes, and pipes (both gridiron and dead end) lines that convey water to demand nodes;
2. A table showing the water demand flow rates at each demand node, the node ground elevations, the pipe length, diameter, and pipe hydraulic roughness coefficients assumed for the model, the tank ground elevations and water level above ground elevations, pump capacity and total dynamic head rating based on the proposed pump curve for the pump
3. A pipe table showing the flow velocity obtained during the simulation
4. Design assumptions such as the basis of the flow rates, the water tank water level, and the number of pumps in operation (e.g. duty and jockey pumps)
5. For existing water distribution systems where the project(s) are being upgraded with additional facilities (barracks or admin buildings, maintenance facilities, or recreational facilities) the existing system pressures as measured or estimated based on booster pump gauges or water level elevations in existing water tanks shall be documented.

Water pressure measurements at existing facilities should include gauge readings taken at locations as close to the new water use facilities. Equipment for monitoring pressure is shown in Appendix B.

11. Shop Submittals and As-Builts

After the completion of any water distribution system all piping and water storage facilities, testing shall be provided per contract specification. Water pipes shall be tested for leakage and hydrostatic pressure performance. Storage tanks shall be tested for leakage performance. Appendix B provides test examples and report forms for use at USACE-AED projects.

Shop submittal shall include the following tests, products and materials:

- Substitutions of pipe material and fittings different from contract technical requirements
- Water pipe pressure and leakage tests reports (see Appendix A for example)
- Water tank leakage tests reports (see Appendix B for example)
- Tank installation drawings with complete details of steel, pipe and concrete work if different than standard drawings. Note for site adapt projects where standard drawings and specifications for water tanks have been prepared; substitutions are not allowed.
- Certifications that internal linings or coatings that come in contact with the potable water comply with NSF 61 (see reference 10). Section 5 Barrier Materials specifically covers products and materials such as coatings and paints applied to storage tanks; linings, bladders and diaphragms in hydro-pneumatic tanks; and constituents of concrete and cement

mortar, blended sealers and admixtures that are field applied or factory applied to precast or cast in place concrete.

Upon completion of installing the water tank system, The Contractor shall submit editable CAD format As-Built drawings. The drawing shall show the final product as it was installed in the field, with the exact dimensions, locations, materials used and any other changes made to the original drawings. Refer to Contract Sections 01335 and 01780A of the specific project for additional details.

12. References

1. UFC 3-230-09a Water Supply: Water Storage, January 2004
2. UFC 3-230-04a Water Distribution, January 2004
3. UFC 3-230-03a Water Supply, January 2004
4. Comprehensive Water Distribution Systems Analysis Handbook, MWH Soft
5. US EPA. EPANET – Users Manual, EPA-600/R-00/057, September 2000 Available at http://www.epa.gov/nrmrl/wswrd/dw/epanet.html
6. AED Design Requirements - Booster Pumps, March 2009
7. AED Design Requirements - Chlorinators, March 2009
8. AED Design Requirements - Hydro-Pneumatic Tanks, March 2009
9. AED Design Requirements - Jockey Pumps, March 2009
10. National Science Foundation, NSF/ANSI 61 – 2008, Drinking Water System Components – Health Effects
11. UFC 3-230-13a Water Supply Pumping Stations, January 2004

Appendix A Example Water Distribution Analysis

The following illustration shows the proposed water plan for a project:

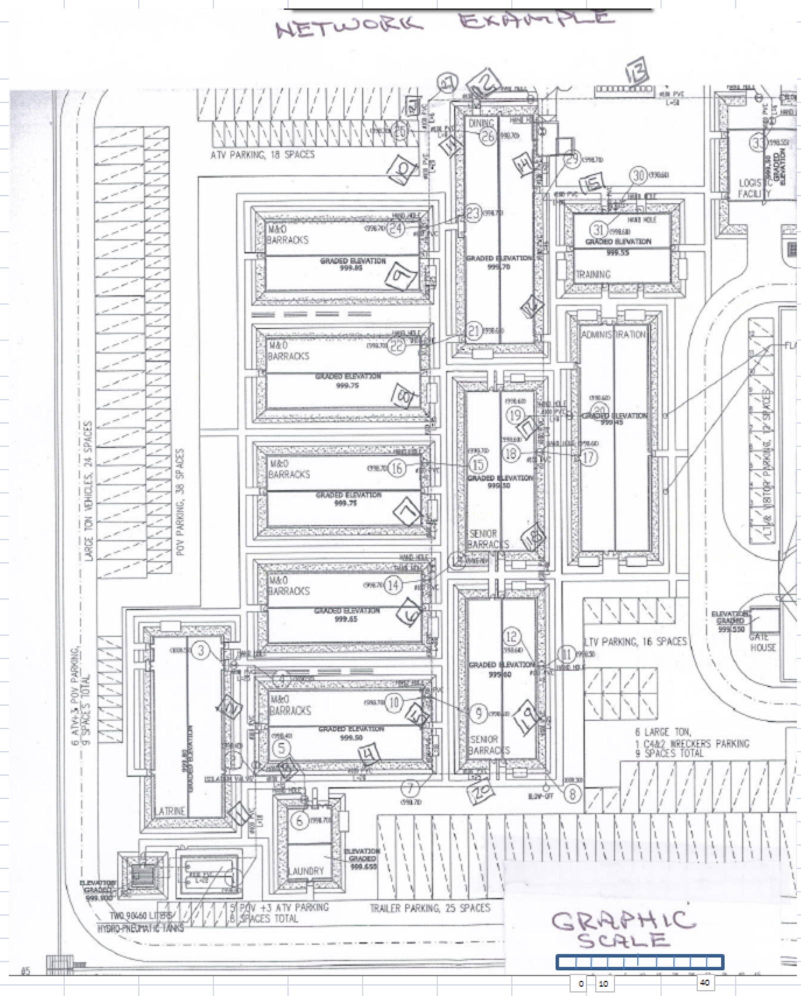
Water Distribution Hydraulic Analysis Example Network Description Table

Node #	Type	Name	Ground Elevation, m	Fixture Demand, l/s	Fixture Units	Fixture Fraction	Pipe #	Diameter, mm	Length, m	Rougness, C	% Total Demand	2x ADD Demand, l/s
1	Junction	BP Sta	998.80				1	100	18	145		
2	Junction		998.40									
3	Demand	Latrine	1000.50	5.01	376	0.52	2	100	18	145	51.51%	0.777
6	Demand	Laundry	998.70	1.20	90	0.12	3	100	11	145	12.33%	0.186
7	Junction		998.70				4	100	28	145		
10	Demand	Middle Barracks	998.70	0.16	12	0.02	5	100	18	145	1.64%	0.025
14	Demand	Middle Barracks	998.70	0.16	12	0.02	6	100	25	145	1.64%	0.025
16	Demand	Middle Barracks	998.70	0.16	12	0.02	7	100	25	145	1.64%	0.025
22	Demand	Middle Barracks	998.70	0.16	12	0.02	8	100	25	145	1.64%	0.025
24	Demand	Middle Barracks	998.70	0.16	12	0.02	9	100	25	145	1.64%	0.025
25	Junction		998.70				10	100	20	145		
27	Junction		998.70				21	100	6	145		
26	Demand	Dining	998.70	0.43	32	0.04	9	100	8	145	4.38%	0.066
28	Junction		998.70				12	100	31	145		
32	Demand	Logistics	998.60	0.16	12	0.02	13	100	56	145	1.64%	0.025
29	Junction		998.70				14	100	21	145		
30	Demand	Training	998.60	0.16	12	0.02	15	100	18	145	1.64%	0.025
19	Demand	Admin	998.60	0.80	60	0.08	16	100	46	145	8.22%	0.124
18	Demand	Senior Barracks	998.60	0.59	44	0.06	17	100	8	145	6.03%	0.091
12	Demand	Senior Barracks	998.60	0.59	44	0.06	18	100	46	145	6.03%	0.091
8	Junction		1000.30				19	100	25	145		
7	Junction		998.70				20	100	25	145		
		Sum		9.72	730				503		100.00%	1.51

Demand comparison for booster pump

Based on fixture basis		Factor (2x ADD)		
35.00	m^3/h		6.4	0.16
9.72	**l/s**			
Based on 2 x ADD basis (16 hour operation)				
5.43	m^3/h		1.0	
1.51	**l/s**			

population	305	capita
water usage	190	per capita -day
capacity factor	1.5	ratio
total ADD	86,925	liter /day
	86.925	m^3/day



The objectives of the simulation for the system include:

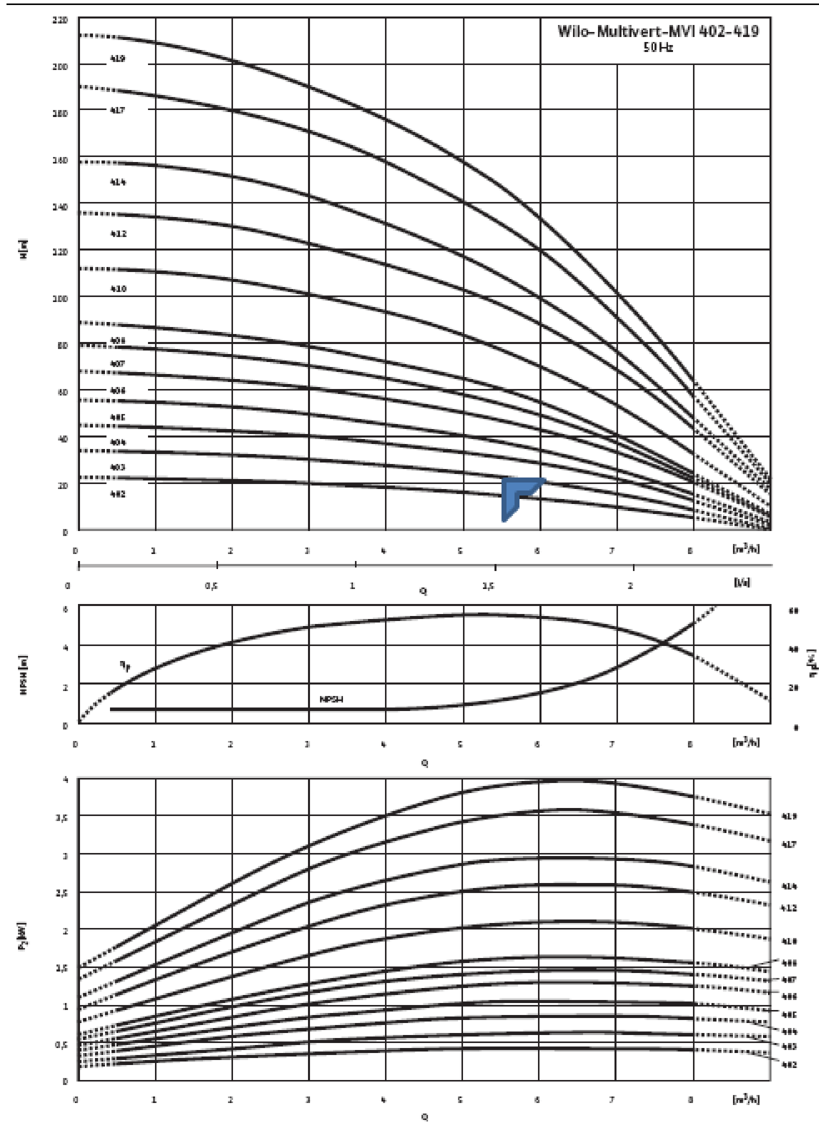
1 Verifying the minimum diameter for the distribution piping is large enough to minimize head loss throughout the system such that the technical criterion for minimum pressure is achieved. This is to be done with the minimum amount of elevated tank or booster pump energy requirements to minimize operation and maintenance cost, for example the booster pump energy required to operate the generators powering the motors.
2 Verifying the maximum flow velocities are not excessive per the technical requirements to minimize peak pressure surges associated with equipment and system operation (for example pump shutoff) that can separate joints and damage plumbing fixtures if excessive pressures occur.
3 Verifying booster and jockey pump selection.
4 Verifying the heights assumed for elevated tanks are adequate.

The project EPANET file is created (see user's manual for detailed instructions, Reference 5) by using the following steps:

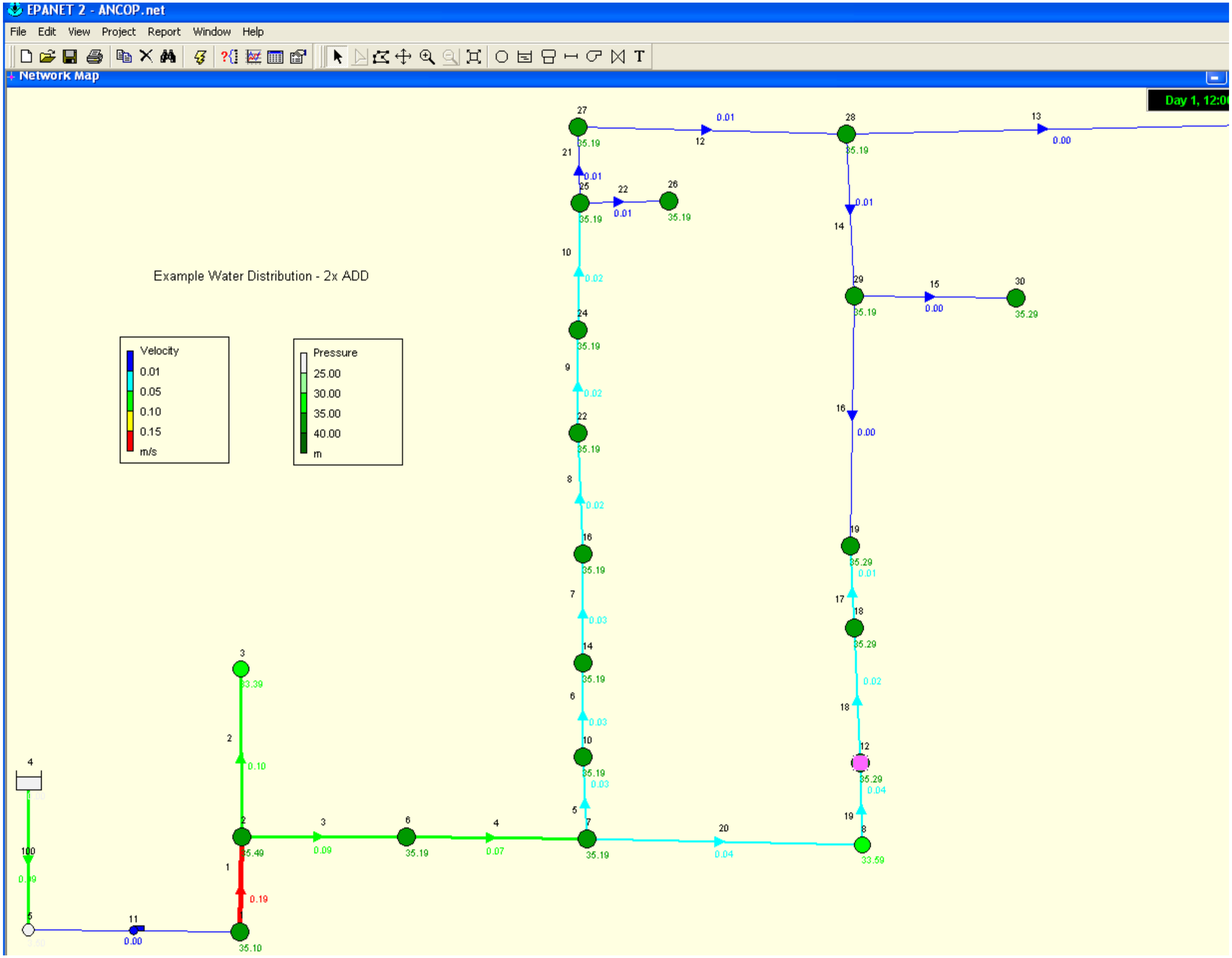
1 Set the design units to SI and defaults and labels
    a. Under project pull down tab edit the following
        i. Hydraulics – set head loss form to Hazen Williams (H-W) and flow units to liter per second (L/S)
        ii. Properties – set pipe length to 25 (meters) and roughness to 145 (conservative value for C value to approximate fitting losses not included)
    b. Under summary – add project title and notes
2 Add pipe junctions (nodes) using the node creation tool on the tool bar – note EPANET automatically enters a number for each node when enters using the tool which can be edited to make the EPANET model numbering scheme identical to the designers numbering scheme on the water site plan. Enter for each node:
    a. Grade elevations for each node location
    b. Base water demands
    c. Additional nodes to later connect the reservoir to the booster station
3 Connect the nodes with pipes using the pipe creation tool on the tool bar
    a. Check that the correct roughness (Hazen William C value) appears in the Browser dialogue box
    b. Adjust the length of the pipe to the correct value determined from the site plan
4 Add a water tank or reservoir as required by the project using the appropriate tool from the tool bar
    a. Use a water tank if the project includes an elevated tank; a reservoir is the choice if the project contains a ground-level reservoir
    b. Set the grade elevation, initial water level and minimum and maximum water levels in the tank
    c. Set the total head (HGL elevation) for the ground level reservoir
5 Enter the pump by using the pump tool to drag a connection between the first downstream node from the reservoir to the first node in the water distribution system connected to the pump. Enter pump information in the Brower dialogue box. Create a pump curve number to enter information for the booster pump from the manufacturer's pump curve.
6 The first simulation should be based on the booster pump design criteria
7 A second simulation (not shown here) should be based using a low flow rate for the jockey pump operation

System operating pressure objective		60	**psi**	**42**	**m**
		50	**psi**	**35**	**m**
Engineer's Pump Curve:					
__**Tabulated data from pump curve**__					
	Pump Curve (2XADD)				
Model MVI-404	m^3/h	L/s	H, m		
	1	0.28	45		
	4	1.11	37.5		
	5	1.39	34		
	5.4	**1.51**	**31.5**		
	6	1.67	28		
	7	1.94	21		
Required for 2 X ADD					

Wilo-Multivert MVI 402 – 419



Pump curves in accordance with ISO 9906, class 2


EPANET 2 - ANCOP.net
File Edit View Project Report Window Help
Network Map
Day 1, 12:0
Example Water Distribution - 2x ADD
Velocity
0.01
0.05
0.10
0.15
m/s
Pressure
25.00
30.00
35.00
40.00
m

Appendix B Procedures for Leakage and Pressure Testing

Hydro static pressure test form

This form is to be used only for hydrostatic tests per AWWA C600

Procedure

1. Following the installation of any new pipeline, all newly laid pipe or valved section shall be subjected to a pressure and leakage test
 The section being tested shall be described on the test form. The information to be included shall be as shown on the form. Do not state "not applicable"
2. Each valved section of pipeline shall be slowly filled with water. The specfied test pressure in the contract techncial requirements shall be applied using a suitable pump.
3. Before applying the final test pressure (1,378 Kpa or 200 psi) unless stated otherwise in contract, air shall be expelled completely from the pipeline section under test
4. The pipeline shall be allowed to stabilize at the test pressure. The test pressure shall not be allowed to vary by plus or minus 34.5 Kpa (5 psi) for a period of one (1) hour.
5. A pressure test apparatus simlar to the one shown in the Figure 1 shall be connected to the test section at either a hose bib or blow off valve location using appropriate fittings
6. Test pressure shall be maintained within th the tolerance stated in step 4 for a minimum one hour duration (or longer if required in the cntract techncial requirements)
7. Pressure readings at 15 minute intervals shall be recorded as shown on the hydrostatic pressure test form example in Figure 2; a blank form is provided in Figure 4.
8. Leakage test shall be conducted followng the pressure test. Leakage test pressure shall be 1034 Kpa (150 psi) unless state otherwise in the contract.
9. If any air has been introduced into the line, expell the air as it could affect the ability to conduct a successful test.
10. The test leakage shall not exceed the volume computed as shown below in Figure 3 for a period of two (2) hours.

Figure 1 Hydraulic pressure and leakage test apparatus

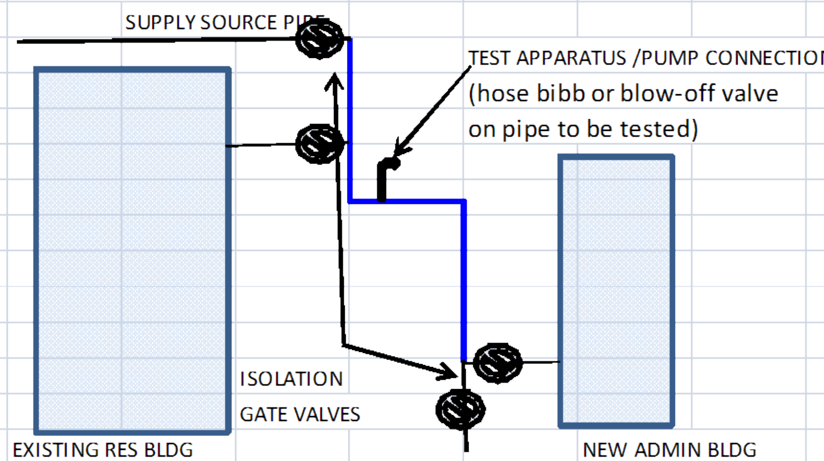
Note: pressure test pump connection shall be made to the inlet
tee (run) using threaded connection from the pump
Provide sufficient reservoir of water to run test including
filling pipe line (if empty) plus allowable leakage in Figure 3

Figure 2 Hydraulic pressure and leakage test form completion example

Test Information (shall be printed including names)

Project name and location	____________
Contract number	____________
Date of test	____________
Location of test	____________
Name of hydraulic test supervisor:	____________
Name of witness	____________

Test site sketch (example)



Pressure test data

Interval	*Time (h:m)*	*Test pressur*
Start of test		1378
1st 15 min	____	____
2nd 15 min	____	____
3rd 15 min	____	____
Completion of test	____	____

Leakage test data

Interval	*Time (h:m)*	*Test pressur*
Start of test		1034
1st 15 min	____	____
2nd 15 min	____	____
3rd 15 min	____	____
4th 15 min	____	____
5th 15 min	____	____
6th 15 min	____	____
7th 15 min	____	____
Completion of test	____	____

Pipe Test Data

Length	100	m
Diameter	100	mm

Figure 3 Allowable leakage

Based on formula:

Allowable leakage in liters per hour, L=(1/794797)*S*D*P^0.5

S= length, D = pipe diameter P= 1034 Kpa

Nominal diameter, mm	Nominal diameter, in	Length of pipe, m	allowable leakage, liter/hour	allowable leakage, liter/ 2hour	milliliters
50	2.0	50	0.1011	0.2023	202
		100	0.2023	0.4046	405
		150	0.3034	0.6069	607
		200	0.4046	0.8092	809
		250	0.5057	1.0114	1,011
		300	0.6069	1.2137	1,214
75	3.0	50	0.1517	0.3034	303
		100	0.3034	0.6069	607
		150	0.4552	0.9103	910
		200	0.6069	1.2137	1,214
		250	0.7586	1.5172	1,517
		300	0.9103	1.8206	1,821
100	3.9	50	0.2023	0.4046	405
		100	0.4046	0.8092	809
		150	0.6069	1.2137	1,214
		200	0.8092	1.6183	1,618
		250	1.0114	2.0229	2,023
		300	1.2137	2.4275	2,427
110	4.3	50	0.2225	0.4450	445
		100	0.4450	0.8901	890
		150	0.6676	1.3351	1,335
		200	0.8901	1.7802	1,780
		250	1.1126	2.2252	2,225
		300	1.3351	2.6702	2,670
120	4.7	50	0.2427	0.4855	485
		100	0.4855	0.9710	971
		150	0.7282	1.4565	1,456
		200	0.9710	1.9420	1,942
		250	1.2137	2.4275	2,427
		300	1.4565	2.9130	2,913
150	5.9	50	0.3034	0.6069	607
		100	0.6069	1.2137	1,214
		150	0.9103	1.8206	1,821
		200	1.2137	2.4275	2,427
		250	1.5172	3.0343	3,034
		300	1.8206	3.6412	3,641

Figure 4 Blank Test Form

Test Information (shall be printed including names)

Project name and location ______________

Contract number ______________

Date of test ______________

Location of test ______________

Name of hydraulic test supervisor: ______________

Name of witness ______________

Test site sketch

Pipe Test Data

Length ________ m

Diameter ________ mm

Pressure test data

Interval	*Time (h:m)*	*Test pressur*
Start of test		1378
1st 15 min		
2nd 15 min		
3rd 15 min		
Completion of test		

Leakage test data

Interval	*Time (h:m)*	*Test pressur*
Start of test		1034
1st 15 min		
2nd 15 min		
3rd 15 min		
4th 15 min		
5th 15 min		
6th 15 min		
7th 15 min		
Completion of test		

Tank Leakage Test

Reference: Underwriters Laboratory Method 142, 2007

Procedure

1. Following the installation of any new steel (welded or bolted water tank), a leakage test shall be performed in accordance with the manufacturer's recommendation. If there are no manufacturer's recommendation, **one** of the leakage test methods based on UL 142 as decribed below shall be used.

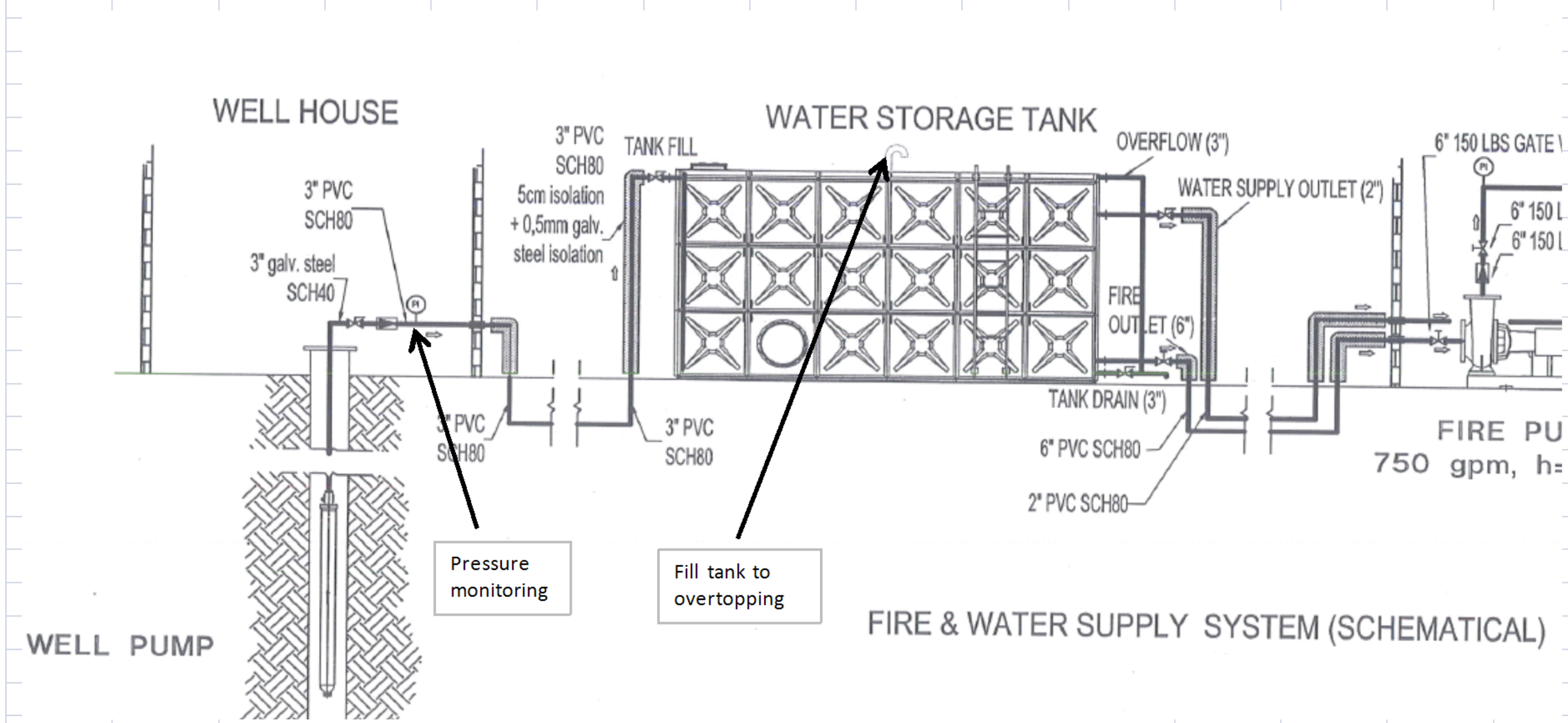
Low Pressure Hydrostatic Test Option

a) A low pressure hydrostatic test shall be conducted on the full tank. The tank shall be filled to overflow and the outlet and drain valves securely closed
A sump pump shall be used to fill the tank by pumping from a barrel of chlorinated fresh water through a hose inserted into the air vent pipe above
until the vent starts to overflow - then the pump shall be shut off. The pressure gauge at the well should be allowed to exceed 55 to 69 KPa (8 to 10 psi)
The pressure gauge at the supply source shall be monitored for drop in pressure for one hour during which time the seams and joints will be
monitored for water leaks; leaks shall be marked with either spray paint of water proof marking pens and noted on the test form
If the pressure drops more than 14 Kpa (2 psi) in the one hour test period, the first trial leakage test shall be indicated as having failed on the test form
A second trial shall be repeated once. A second failure shall be cause for rejection of the tank watertightness integrity

Low Pressure Air Test Option

b) A low pressure air test shall be conducted on the empty tank. The tank shall be drained of water using the tank drain
All external vent pipes and drains that are not closed by valves shall be blinded with threaded caps, inflatable pipe plugs or gasketed blind flanges
An air compressor shall be used to pressurized the tank to between 10 to 14 KPa (1.5 to 2 psi) using a fitting on the end of the drain pipe
A soap-based water solution consisting of one part hand soap mixed with 10 parts clear water shall be made and applied to the all joints and tank seams
A pressure gauge at the air supply source shall be monitored for drop in pressure for one hour during which time the seams & joints will be
monitored for bubbles indicating air leakage from these joints; leaks shall be marked with spray paint of marking pens - and noted on the test form
The visual identification of air bubbles at the seams/ joints or a drop of pressure to zero will indicate leakage which
shall be cause for rejection of the tank watertightness integrity

Figure 1 Hydraulic leakage test setup



WATER TANK AND WELL SOURCE DIAGRAM (Mark leaks or provide other sketch as needed)

Figure 2 Hydraulic leakage test form completion example

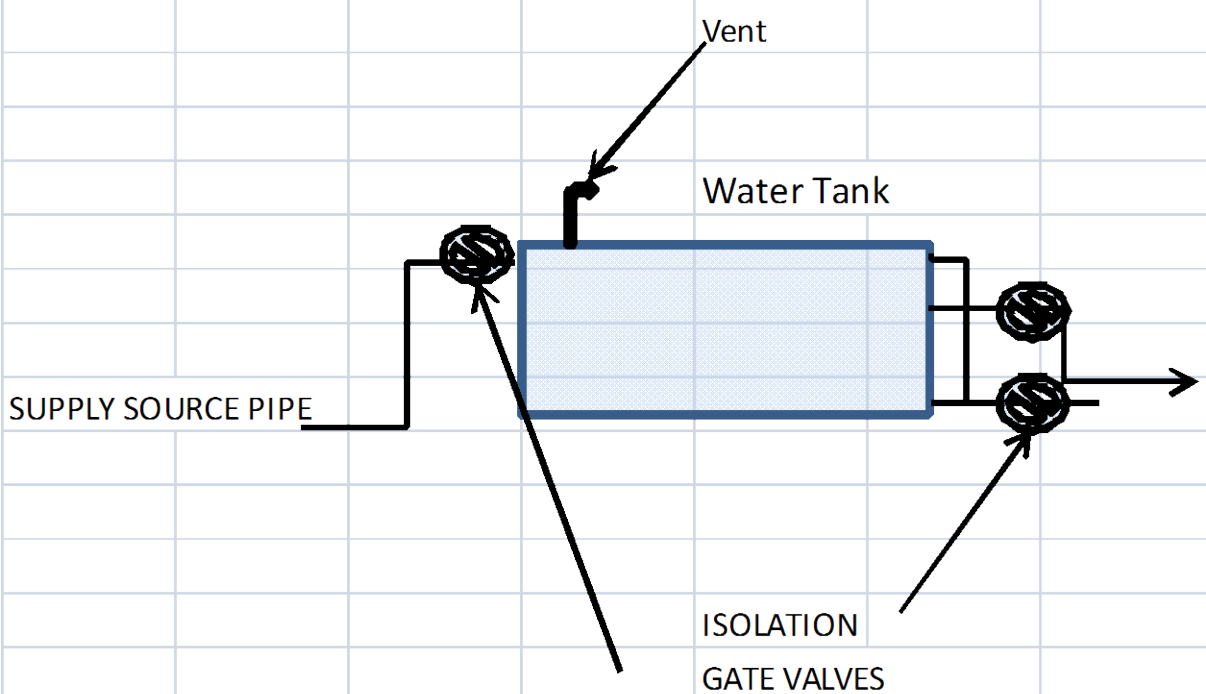
Test Information (shall be printed including names)

Project name and location	____________
Contract number	____________
Date of test	____________
Location of test	____________
Name of leakage test supervisor:	____________
Name of witness	____________

Pressure test data

Interval	*Time (h:m)*	*Test pressure (Kpa or Psi)*
Start of test		34
1st 15 min		
2nd 15 min		
3rd 15 min		
Completion of test		

Test site sketch (example)



Leakage test data

Interval	*Time (h:m)*	*Location of leak*
Start of test		(show on tank diagram above)
1st 15 min		
2nd 15 min		
3rd 15 min		
4th 15 min		
5th 15 min		
6th 15 min		
7th 15 min		
Completion of test		