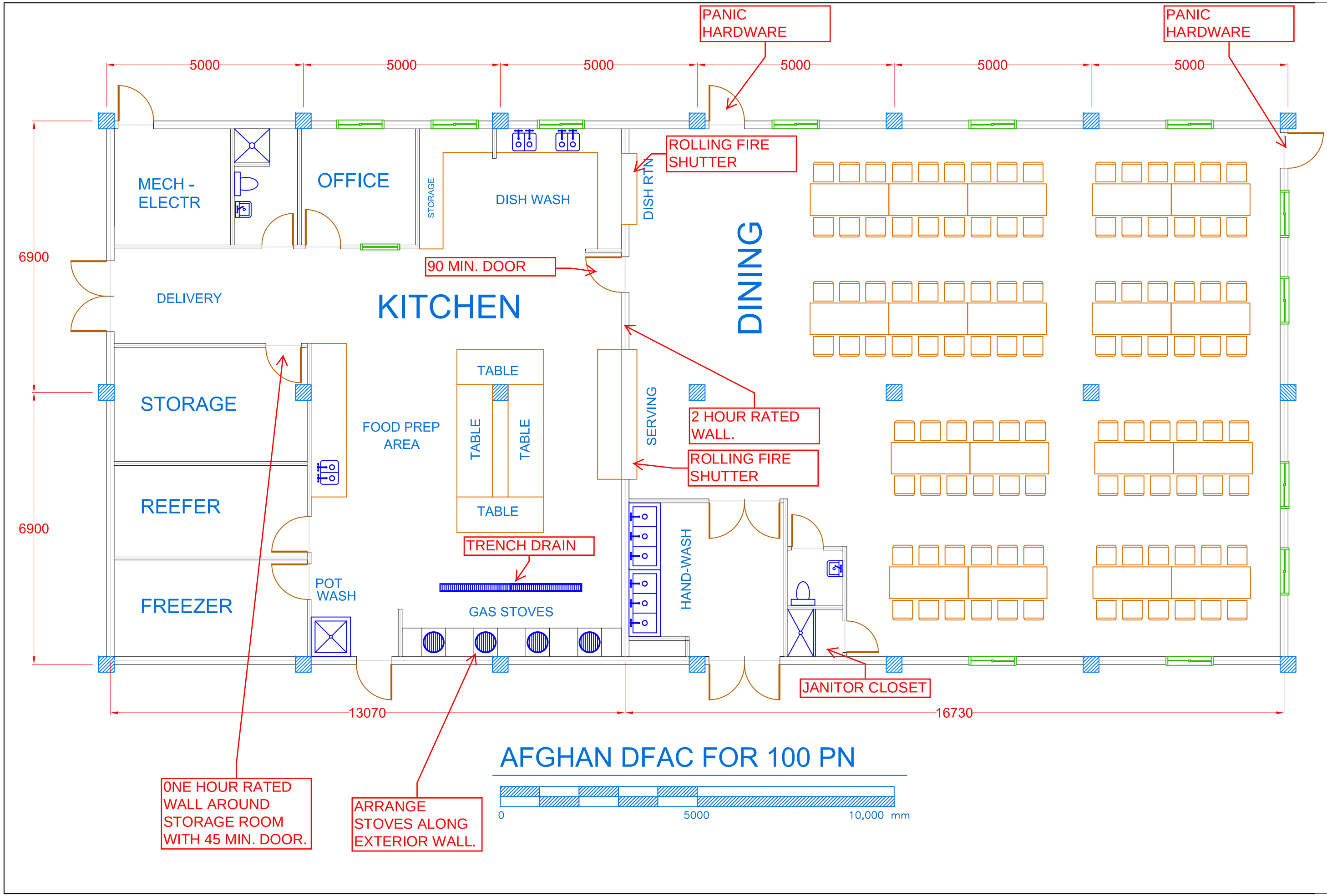
US Army Corps
of Engineers
Afghanistan Engineer District

AED Design Requirements: 100 PN DFAC

Various Locations,
Afghanistan

MARCH 2009



REVISIONS

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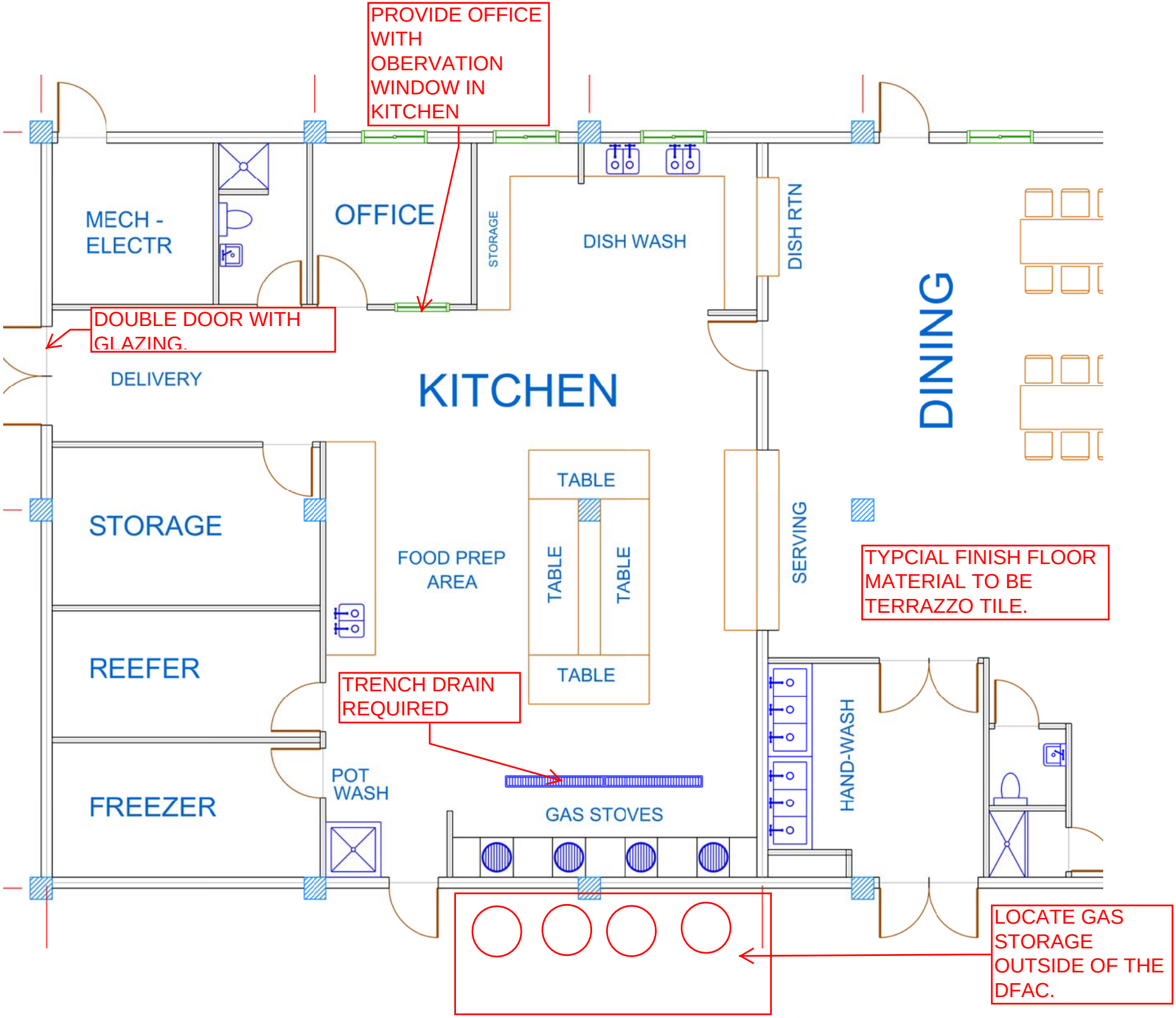
US ARMY CORPS OF ENGINEERS
AFGHAN ENGINEER DISTRICT
ENGINEERING BRANCH
KABUL, AFGHANISTAN

W917PM-09-C-XXXX
ENGINEERING BRANCH
AFGHAN NATIONAL ARMY
100 PN DFAC PLAN

A-1

SHEET:
1 OF 1


PROVIDE OFFICE WITH OBERVATION WINDOW IN KITCHEN
MECH - ELECTR
OFFICE
STORAGE
DISH WASH
DISH RTN
DINING
DOUBLE DOOR WITH GLAZING.
DELIVERY
KITCHEN
STORAGE
FOOD PREP AREA
TABLE
TABLE
TABLE
TABLE
SERVING
TYPCIAL FINISH FLOOR MATERIAL TO BE TERRAZZO TILE.
REEFER
TRENCH DRAIN REQUIRED
HAND-WASH
POT WASH
FREEZER
GAS STOVES
LOCATE GAS STORAGE OUTSIDE OF THE DFAC.

DFAC architectural standards

---requires two hour wall separation between kitchen and dining area; this wall also requires one 900 mm width 90 minute rated door (no glazing). Wall must extend to the roof, not ceiling. We want grout filled 200 mm CMU for the fire rated wall
---all other interior walls are 100 mm CMU for DFAC, no GWB is to be used.
---same separation wall requires two service counter openings, one for dish wash and one for food serving. Openings depend on the size of the DFAC but general rule is that the serving area opening is twice the size of the dish wash return opening. Openings are 1000 mm AFF and 1200 mm in height (width depends on the size of the DFAC). An alarm activated fire rated shutter is required.
---standard DFAC dining room ceiling height is 3000 mm AFF, kitchen can be reduced, depending on size. Suspended ceilings (WP GWB) or concrete slabs can either be used.
---storage rooms require one hour wall (to roof) with 45 minute doors.
---delivery area requires cover canopy for weather protection and double doors with glazing.
---exit doors for dining area require panic hardware.
---general ratios for serving: one stove serves 25 diners, one hand-wash station serves 15 diners, 4 stoves require one pot wash
---use gas stoves, not wood. Locate gas storage OUTSIDE the DFAC bldg, with 200 mm wall separation. Storage area requires fencing, with lockable gate and 24 gauge corrugated metal roof cover. Tanks require chains to hold them upright.
---for kitchen area in food preparation and stove areas, trench drains are required and must be connected to OUTSIDE grease trap. Dining area requires floor drains.
---provide one small unisex toilet with lavatory for the dining area.
---provide toilets and showers for the kitchen staff, size dependent on the number of workers per one shift. Also, one janitor closet is required.
---design kitchen per flow of operations as follows: (1) delivery of food (2) storage of food (3) food preparation (4) cooking (5) food serving (6) dish return, dish wash and dish storage
---SST backing (height = 2000 mm AFF) for stove area. Stoves located on exterior wall. Stoves have hood duct fan exhaust thru the ceiling and roof. Exhaust stack 1000 mm above roof for grease dispersal.
---kitchen should be no more than 40% of overall DFAC.
---floor finish terrazzo tile for all areas in DFAC, no ceramic tile except for toilet rooms. Sealed concrete for storage rooms.
---Interior columns are OK for dining area; clear spans are not needed
---provide office for kitchen with observation window

US Army Corps
of Engineers
Afghanistan Engineer District

AED Design Requirements:
Booster Pumps

Various Locations,
Afghanistan

September 2009 Version 1.0

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AED DESIGN REQUIREMENTS
FOR
BOOSTER PUMPS
VARIOUS LOCATIONS,
AFGHANISTAN

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AED Design Requirements
Booster Pumps

1. General

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

2. Booster Pump Layout

Booster pumps may be required anywhere in a water distribution system to increase the pressure in the pipeline. Booster pump stations are usually located remote from the main pump station or elevated storage, as in hilly topography where pressure zones are required. Booster pumps may be needed to handle peak flows in a distribution system which can otherwise handle the normal flow requirements. Booster pumps may be above ground or underground. Pump and control selection for in-line booster pumps will consider minimum suction pressure and automatic discharge cut-off pressure.

3. Pump Selection

a) System Head Curve. The first step in the pump selection process is to determine the system head curve for the piping system. The system head curve is determined from the total head (H) required in the piping system at varying flow rates. The total head (H) is the total discharge head (h_d) minus the total suction head (h_s).

$$H=h_d-h_s$$

The total discharge head (h_d) is the sum of the static discharge head (h_{sd}), the discharge velocity head (h_{vd}) at the pump discharge flange and the friction head (h_{fd}) in the discharge line.

$$H_d=h_{sd}+h_{vd}+h_{fd}$$

The total suction head (hs) is the sum of the static suction head (h_{ss}) and the suction velocity head (h_{vs}) at the pump suction flange minus the total friction head (h_{fs}) in the suction line.

$$H_s=h_{ss}+h_{vs}+h_{fs}$$

Each type of head for the discharge and suction sides of the pump are calculated in the same manner.

b) Static Head. Static head is the vertical distance in meters from the pump inlet centerline to the highest point of free discharge. The static discharge head (h_{sd}) is easily determined by simply subtracting the elevation at the centerline of the pump from the elevation of the highest discharge point in the system on the discharge side of the pump. The static suction head (h_{ss}) is the elevation difference between the centerline of the pump and the free water surface elevation of the supply reservoir. If the suction side of the pump is supplied by a pressurized water system, the static suction head will be the minimum pressure in the suction line at the pump flange, converted to head. Theses value will remain constant for all flow rates.

c) Velocity Head. Velocity head is the head of a fluid as a result of its kinetic energy as shown by the following equation.

$$h_v=v^2/2*g_c$$

Where:

v=Q/A=flow velocity at the supply or discharge flanges of the pump (m/sec)
g_c=gravitational constant=9.81 m/sec^2
Q=flow (m^3/sec)

A= pipe area (m^2)=π*(D^2/4)
D= pipe diameter (m).

The velocity head is a function of the fluid velocity which is related to the flow rate. The discharge velocity head (h_{vd}) will need to be calculated for specific flow rates within the system. The suction velocity head (h_{vs}) will need to be calculated for the minimum flow velocity at the pump flange on the suction side of the pump.

d) Friction Head. Friction head (h_f) is the head required to overcome resistance to flow in the pipe and fittings. It is dependent upon the size, condition and type of pipe, number and type of pipe fittings, flow rate and nature of the liquid.

$$h_f=L[(v/(kC))*(4/D)^{0.63}]^{1/0.54}$$

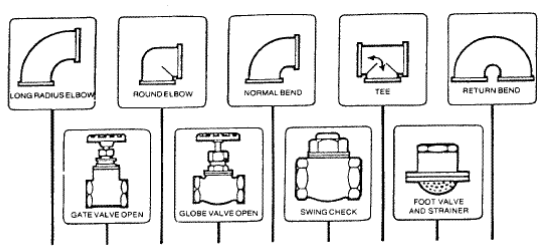
Where:

L=equivalent length of pipe and fittings (m)
v=flow velocity in pipe (m/sec)
k=0.85.
C=Hazen-Williams coefficient.
D=pipe diameter (m)

The discharge friction head (h_{fd}) will need to be calculated for specific flow rates within the system. Since the suction side of the pump will be supplied by a pressurized water system that will supply water to the pump, the suction friction head (h_{fs}) can be assumed to be zero.

e) Equivalent Pipe Length. Equivalent pipe length is the sum of the pipe length and the minor losses associated with all fittings along the pipe. Numerous minor loss tables are available which expressed the minor losses associated with various fittings as an equivalent pipe length. The designer should make sure that all minor losses are accounted for in the system and that the proper equivalent lengths are used for each type of fitting. A sample of equivalent length of straight pipe for various fitting of differing sizes id provided in Table 1.

Table 1. Minor Losses



Pipe size mm	Equivalent length of straight pipe in metres, for calculating friction loss								
20	0.3	0.3	0.6	6.7	0.5	1.5	1.5	1.5	1.5
25	0.3	0.3	0.8	8.2	0.5	2.0	1.8	2.3	2.0
32	0.3	0.6	0.9	11.3	0.8	2.6	2.4	2.7	2.6
40	0.4	0.6	1.1	13.4	0.9	3.1	2.7	3.4	3.1
50	0.5	0.8	1.4	17.4	1.1	4.0	3.4	4.6	4.0
65	0.6	0.9	1.7	20.1	1.4	5.2	4.3	5.5	4.6
80	0.8	1.1	2.1	26.0	1.5	6.1	5.2	6.7	5.5
100	1.1	1.5	2.7	34.0	2.1	8.2	6.7	8.8	7.3
125	1.2	1.8	3.7	43.0	2.7	10.0	8.2	11.0	9.5
150	1.5	2.1	4.3	49.0	3.4	12.2	10.0	14.0	11.0
200	2.1	3.1	5.5	67.0	4.3	16.5	13.4	18.0	15.0
250	2.4	3.7	7.3	85.4	5.5	20.0	16.5	22.0	19.0
300	3.1	4.3	8.5	98.0	6.7	24.4	20.0	27.4	23.0

f) Hazen-Williams Coefficient. As with minor loss tables, numerous Hazen-Williams coefficient tables are available. The designer should select the appropriate coefficient based on the pipe material and age. A sample Hazen-Williams coefficient table is provided in Table 2.

Hazen–Williams Coefficient C_{hw}	
Pipes extremely straight & smooth	140
Pipes very smooth	130
Smooth wood, smooth masonry	120
New riveted steel, vitrified clay	110
Old cast iron, ordinary brick	100
Old riveted steel	95
Old iron in poor condition	60–80

g) Pump Curve. With the system head curve defined, it is possible to select a pump to deliver the required capacity. Manufacturer's published pump head-capacity curves will be used for this process. Since these pump head-capacity curves usually apply to a particular impeller and pump design, different manufacturers may show slightly different performance for the same type and size of pump. The system head curve is plotted on a manufacturer's pump head-capacity curve to determine the operating characteristics of the pump. The point where the system head curve and the pump head-capacity curve intersect will be the theoretical point at which the pump will operate. If the intersection point of the system head curve and the pump head-capacity curve is not satisfactory for the intended purpose, a different pump must be selected.

4. Booster Pump Valves

Gate valves will be used on the suction and discharge sides of the booster pump to isolate the booster pump from the remainder of the distribution system for maintenance purposes. A check valve will be provided between the pump and gate valve on the discharge side of the pump to protect the pump from excessive back pressure and to prevent liquid from running backward through the pump in case of a power failure. Pressure relief valves, commonly diaphragm activated globe or angle type, will be installed on the discharge piping system for flow control and/or pressure regulation, and to protect pump equipment and piping system from excessive surge pressures. Pump control system valves range from single hand-operated valves to highly advances, automatic flow control or pump speed control systems. In an unattended high head booster pump system the control valve may have a controller to close automatically when the pump is stopped and to open once the pump has reached a specified speed after the pump is started. Control valves are installed to prevent surge pressures, which cause water hammer and high pressure. Designers should use their best judgment when selecting control valves. Minor losses for all valves on the discharge side of the pump should be included in the system head curve calculations.

5. Booster Pump Controls

Pump controls will have the capacity to provide the desired flow rates and pressures, to provide protection from pump and piping system damage, and to serve as a tool to find system problems which may need operational adjustments, repair or maintenance. Control systems consist of the following:

Sensing and measuring elements (primary device).
Comparison and relaying element (controller).
Final control element to produce the required change including an actuator to move control elements.

a) Sensing and Measuring Elements. Sensing and measuring elements consist of automatic pump control and valve operation equipment that will detect changes in the flow and emit a signal. The

most common sensing and measuring elements for booster pumps are pressure sensors and flow meters.

b) Comparison and Relaying Elements. Comparison and relaying elements are the transducers and transmitters, which are usually housed together in the controller which often is physically separated from the primary device and convert the signal from the sensing and measuring equipment into another signal medium that is necessary for actuation of the final control element.

c) Final Control Elements. For the final control element, valves and pumps serve for no-off control and modulating needs in the system. A control valve is a valve that modulates the flow through it to provide the desired pressure within the system. The term control valve means a specialized type of power-activated valve designed to modulate flow to meet system demands. The term pump as a final control element is a pump provided with automatic variable speed control drive to maintain an essentially fixed flow rate and for controlled flow rate increase/decrease at start/stop of pumping to minimize surge in the system. Variable speed pumps will be considered only for large pumps and only if justified by an accurate economic analysis. Instrumentation for a booster station will supervise and monitor the routine operation of pumps, their drives and accessories to sustain a desired level of performance and reliability. Alarm situations will be identifies, such as low flow and low pressure, pump failure, power failure and low suction head (water loss). The selection of all booster pump controls should be based on the system layout and the designer's experience. When possible, the designer should select a packaged booster pump system.

6. Design Capacities.

For installations with fewer than 400 persons, the capacity shall be based the installation wide, total fixture unit flow. For installations with greater than 400 persons, the capacity shall be based on the installation wide, total fixture unit flow or 2 times the average daily flow, whichever is greater. Three identical pumps shall be provided which are all sized to deliver 50% of the calculated capacity. Pumps shall automatically alternate to distribute wear and shall automatically turn on and off based on demand and system pressures. The total dynamic head (TDH) of the booster pumps shall be calculated to maintain a minimum, residual system pressure of 40 psi at the calculated capacity unless stated otherwise in the contract documents.

7. As-Builts

Upon completion of installing the booster pump 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.

Example – Booster Pump sizing and Selection

Determine the system head curve for a flow range from 0.00 to 0.10 cubic meters per second.

Given:

Pump elevation=23.50 meters
Discharge elevation=87.65 meters
Static suction head=0.00 meters
Pipe diameter=0.20 meters
Pipe length=700 meters
Hazen-Williams coefficient=130
Fittings:
- 1-Fully open gate valve
- 1-Swing check valve
- 3-Line flow flanged tees

Calculated:

1. Determine static discharge head (h_{sd}) which is the discharge elevation minus the pump elevation.

   h_{sd}=87.65 meters-23.50 meters=64.15 meters.

2. Determine pipe area (A)=3.14159*(0.2^2/4)=0.0314 m^2

3. Determine pipe velocity for variable flow rates.

   Velocity (v)=flow/pipe area

Flow	Velocity
0.00	0.00
0.02	0.64
0.04	1.27
0.06	1.91
0.08	2.55
0.10	3.18

4. Determine the discharge velocity head (h_{vd})=v^2/2*g_c

Flow	h_{vd}
0.00	0.00
0.02	0.02
0.04	0.08
0.06	0.19
0.08	0.33
0.10	0.52

5. Determine the minor losses:
   1-Fully open gate valve @ 0.15=0.15 meters
   1-Swing check valve @ 2.00=2.00 meters
   3- Line flow flanges tees @ 0.20=0.60 meters
   Total minor losses=2.75 meters

6. Determine equivalent pipe length which is the sum of the pipe length plus the minor losses.

Equivalent pipe length=700 meters+2.75 meters=702.75 meters.

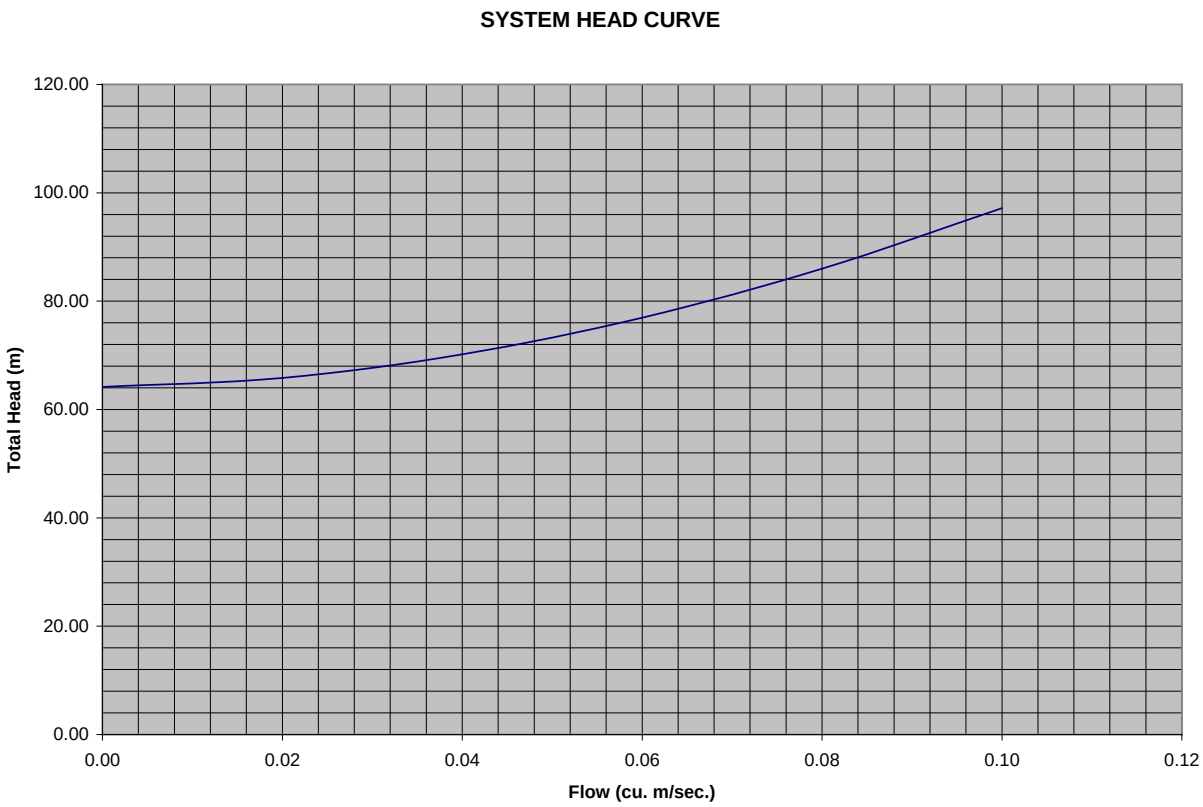
7. Determine the discharge friction head $h_{fd})=L[(v/(kC))*(4/D)^{0.63}]^{1/0.54}$

Flow	h_{fd}
0.00	0.00
0.02	1.65
0.04	5.96
0.06	12.62
0.08	21.50
0.10	32.50

8. Determine total discharge head (h_d) which is the sum of the static discharge head, discharge velocity head and the discharge friction head.

Flow	+	h_{sd}	+	h_{vd}	+	h_{fd}	=	h_d
0.00		64.15		0.00		0.00		64.15
0.02		64.15		0.02		1.65		65.82
0.04		64.15		0.08		5.96		70.19
0.06		64.15		0.19		12.62		76.96
0.08		64.15		0.33		21.50		85.98
0.10		64.15		0.52		32.50		97.17

9. Plot the system head curve.



US Army Corps
of Engineers
Afghanistan Engineer District

AED Design Requirements:
Chlorinators

Various Locations,
Afghanistan

MARCH 2009

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AED DESIGN REQUIREMENTS
FOR
CHLORINATORS
VARIOUS LOCATIONS,
AFGHANISTAN

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1. General

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

2. Hypochlorinators

A hypochlorinator is a piece of equipment used to feed liquid chlorine (bleach) solutions. Hypochlorinators used on small water systems are very simple and relatively easy to install. Typical installations are shown in figures below. Hypochlorinator systems usually consist of a chemical solution tank for the hypochlorite, diaphragm-type pump.

Figure 1. Typical Hypochlorinator Installation

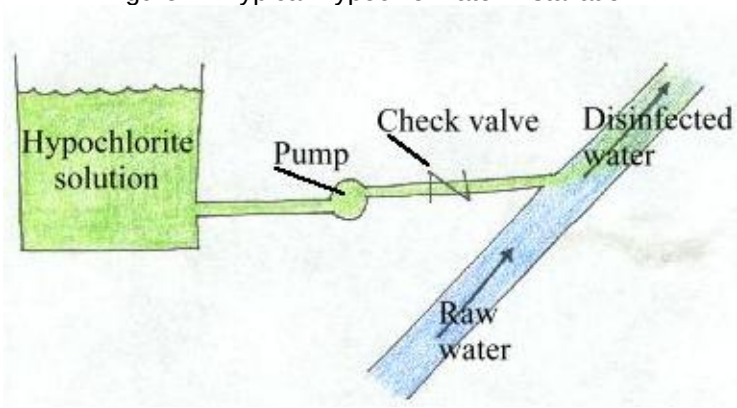
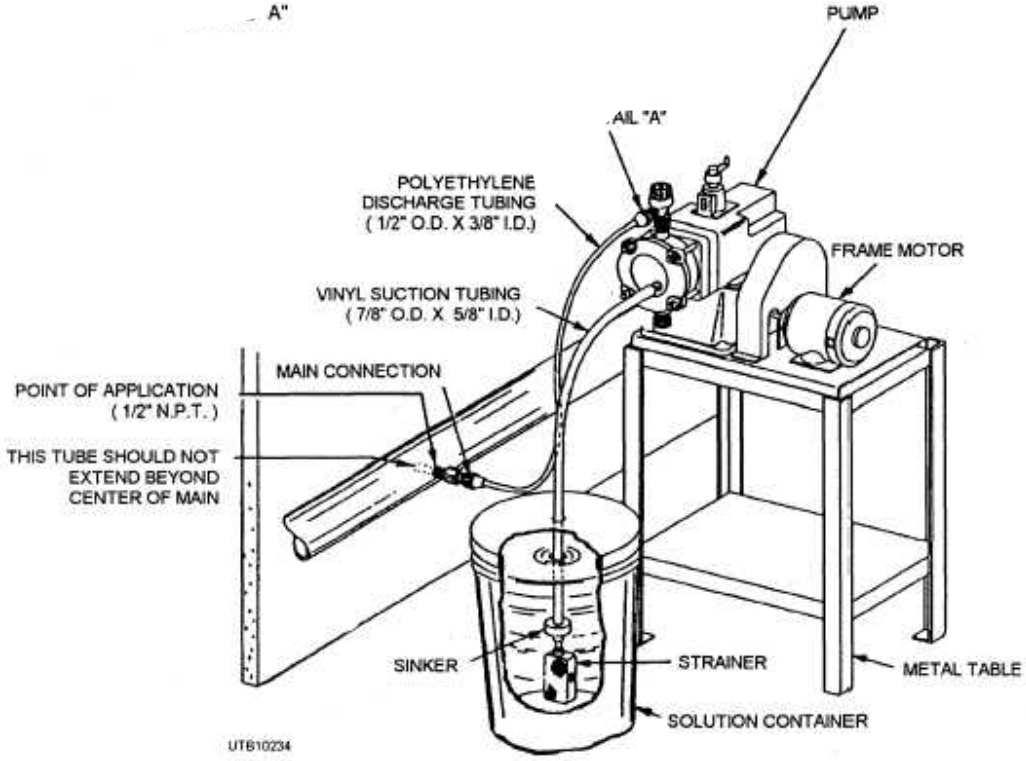


Figure 2. Hypochlorinator Paced by a Propeller-Type Water Meter



3. Hypochlorinator Operation

When operating a hypochlorinator you should compare the actual chlorine dose applied to the water being treated with the desired chlorine dose in milligrams per liter. The actual dose is calculated by determining the amount of chlorine actually used and the amount of water treated. The amount of chlorine used is found by measuring the amount of hypochlorite solution used and knowing the strength of the hypochlorite solution. The amount of water used is determined from a flowmeter.

To calculate the amount of water treated, determine the amount in gallons from a flowmeter and convert this amount from gallons to pounds.

Water, lbs = (Water Treated, gal)(8.34 lbs/gal)

Hypochlorite, gallons = (0.785)(Diameter, ft)^2(Depth, ft)(7.48 gal/cu ft)

To determine the pounds of chlorine used to disinfect the water being treated, we have to convert the hypochlorite used from gallons to pounds of chlorine by considering the strength of the hypochlorite solution.

Chlorine, lbs = (Hypochlorite, gal)(8.34 lbs/gal)(Hypochlorite, %/100%)

Finally, to estimate the actual chlorine dose in milligrams of chlorine per liter of water treated, we divide the pounds of chlorine used by the millions of pounds of water treated. Pounds of chlorine per million pounds of water is the same as parts per million or milligrams per liter (ppm=mg/L).

Chlorine Dose, mg/L = Chlorine Used, lbs/Water Treated, Million lbs

4. As-Builts

Upon completion of installing the culvert, 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.

Example 1. Check Adequacy of Hypochlorinator

Water pumped from a well is disinfected by a hypochlorinator. A chlorine dose of 1.2 mg/L is necessary to maintain adequate chlorine residual throughout the system. During a one-week time period, the water meter indicated that 2,289,000 gallons of water were pumped. A two percent sodium hypochlorite solution is stored in a three-foot diameter plastic tank. During this one-week period, the level of hypochlorite in the tank dropped 2 feet 8 inches (2.67 feet). Does the chlorine feed rate appear to be too high, too low, or about OK?

Known:
Desired Chlorine Dose=1.2 mg/L
Water Pumped=2,289,000 gal
Hypochlorite= 2%
Chemical Tank Diameter= 3 ft
Chemical Drop in Tank= 2.67 ft

Unknown:
Actual Chlorine Dose, mg/L
Is Actual Dose OK

1. Calculate the pounds of water disinfected.
Water = (Water Pumped, gallons)(8.34 lbs/gal)

= (2,289,000 gal)(8.34 lbs/gal)
= 19,090,000 lbs
= 19.09 Million lbs

2. Calculate the volume of two percent sodium hypochlorite used in gallons.

Hypochlorite = (0.785)(Diameter, ft)^2(Depth, ft)(7.48 gal/cu ft)
= (0.785)(3 ft)^2(2.67 ft)(7.48 gal/cu ft)

= 141.1 gallons

3. Determine the pounds of chlorine used to disinfect the water.

Chlorine = (Hypochlorite, gal)(8.34 lbs/gal)(Hypochlorite, %/100%)
= (141.1 gal)(8.34 lbs/gal)(2%/100%)
= 23.5 lbs Chlorine

4. Estimate the chlorine dose.

Chlorine Dose = Chlorine, lbs/Water, Million lbs
= 23.5 lbs Chlorine/19.09 Million lbs Water
= 1.23 mg/L

Since the actual estimated chlorine dose (1.23 mg/L) was similar to the desired dose of 1.2 mg/L, the chlorine feed rate appears OK.

US Army Corps
of Engineers
Afghanistan Engineer District

AED Design Requirements:
Culverts & Causeways

Various Locations,
Afghanistan

JULY 2009, Version 1.3

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FOR
CULVERT /CASUEWAY DESIGN
VARIOUS LOCATIONS,
AFGHANISTAN

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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.

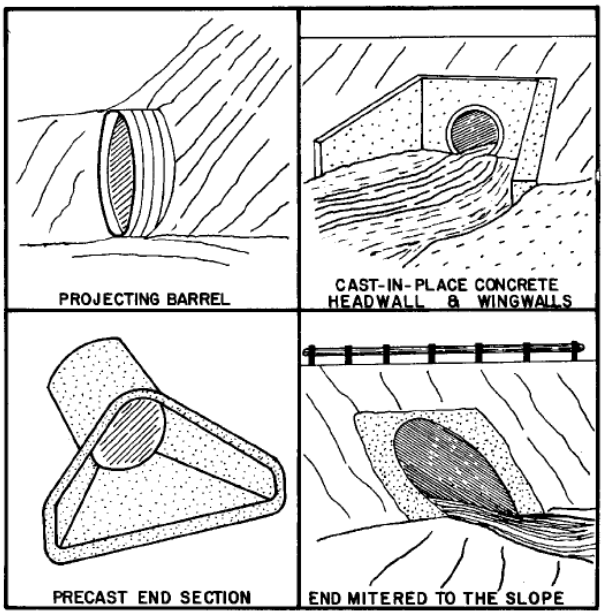
9) Contractors are encouraged to adapt the geometry (i.e. shape and slope) of the standard MRRD culvert and causeway design drawings to site conditions where applicable. For example, reinforced box culvert cross slopes should be adjusted to existing channel grade

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.

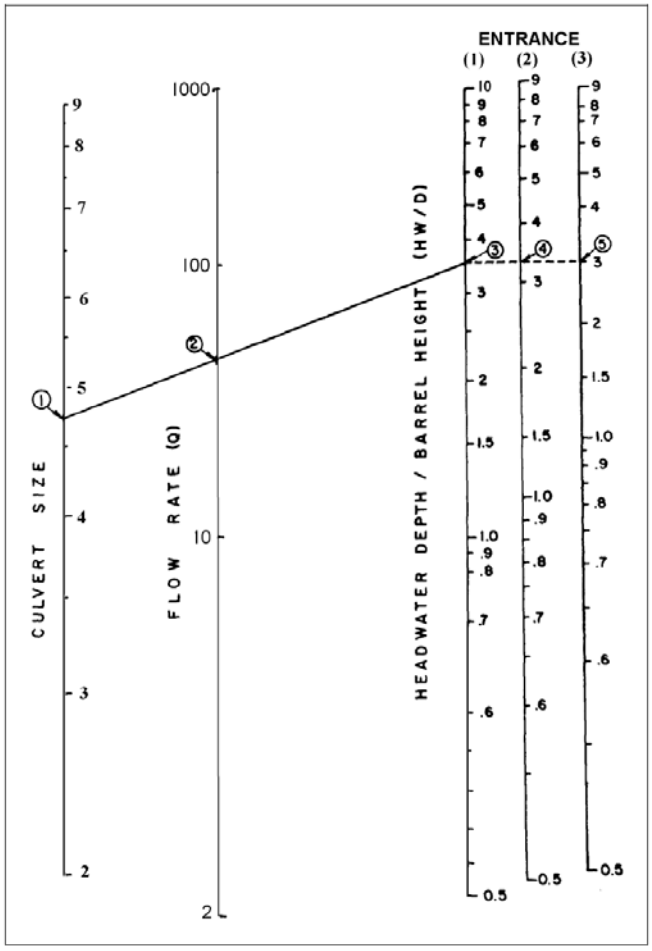
Appendix A – Culvert Design Procedure

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

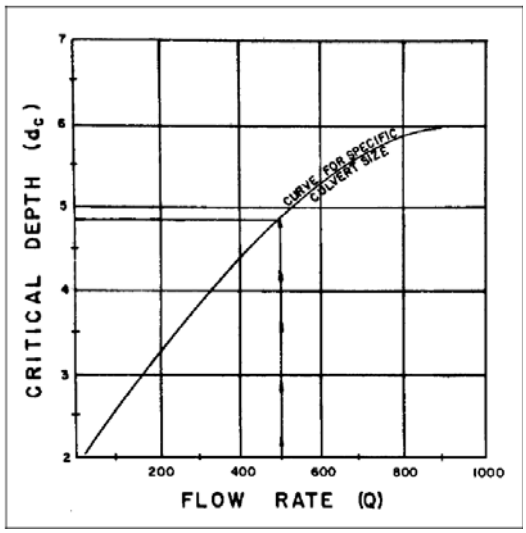
The outlet control calculations result in the headwater elevation required to convey the design discharge through the selected culvert in outlet control. The critical depth charts and outlet control nomographs are used in the outlet control design process. The following steps should be completed to calculate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

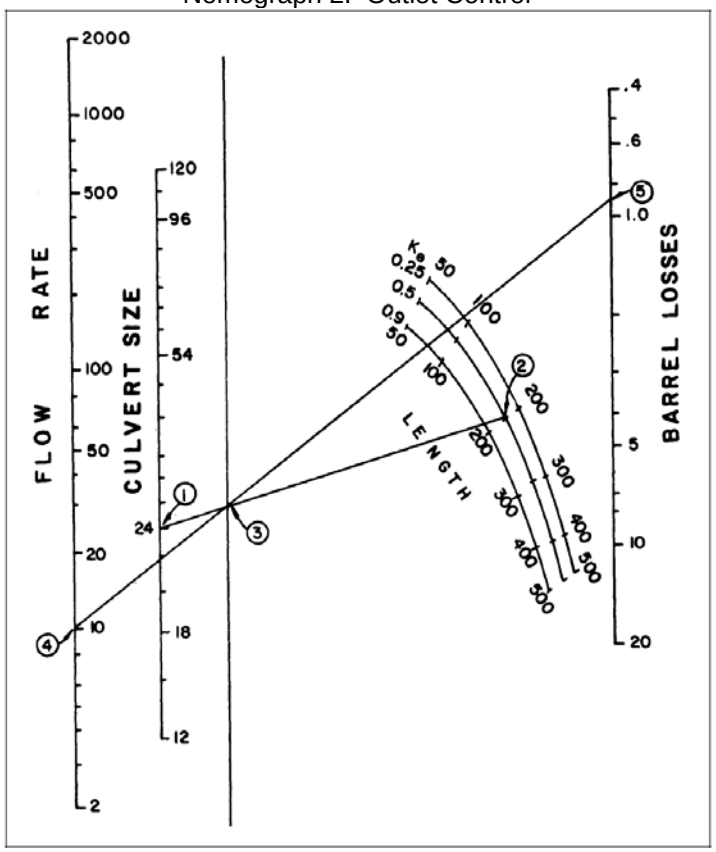
5) Determine the appropriate entrance loss coefficient, k_e, for the culvert inlet configuration. An entrance loss coefficient table is shown below.

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° 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 conform 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 D/12 or B/12 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 D/12 or beveled top edge	0.2
Wingwall at 10° to 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

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	
	0.5	0.3	Tailwater variation determined by
	1	0.51	using cross sections upstream and
	1.5	0.64	downstream of culvert to compute
Based on channel n value=		0.035	water surface profile using standard
			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			

Technical Notes per Design Chart Explanation========>			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

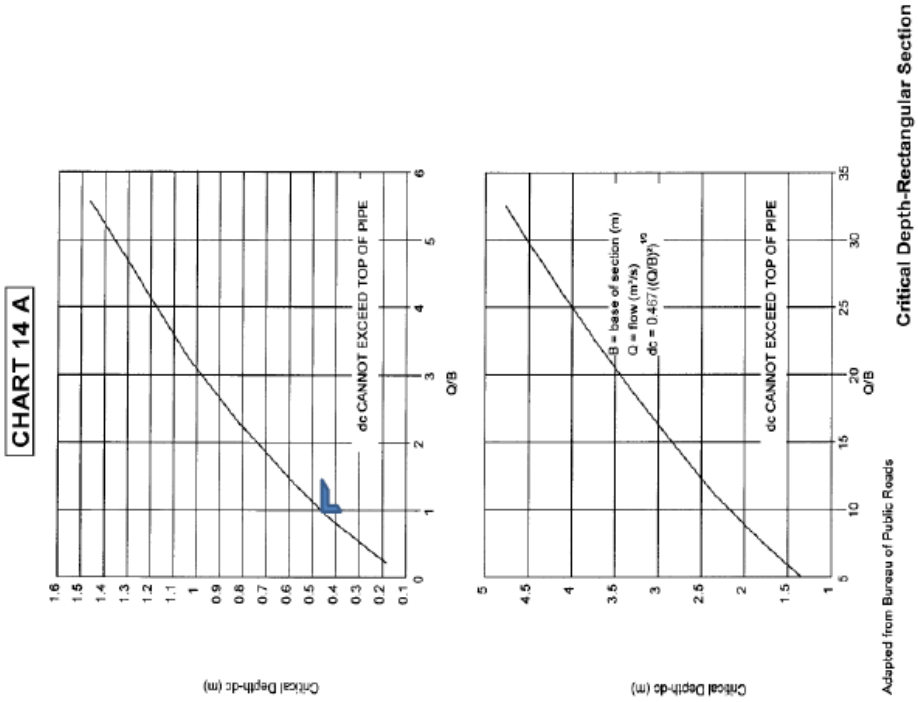
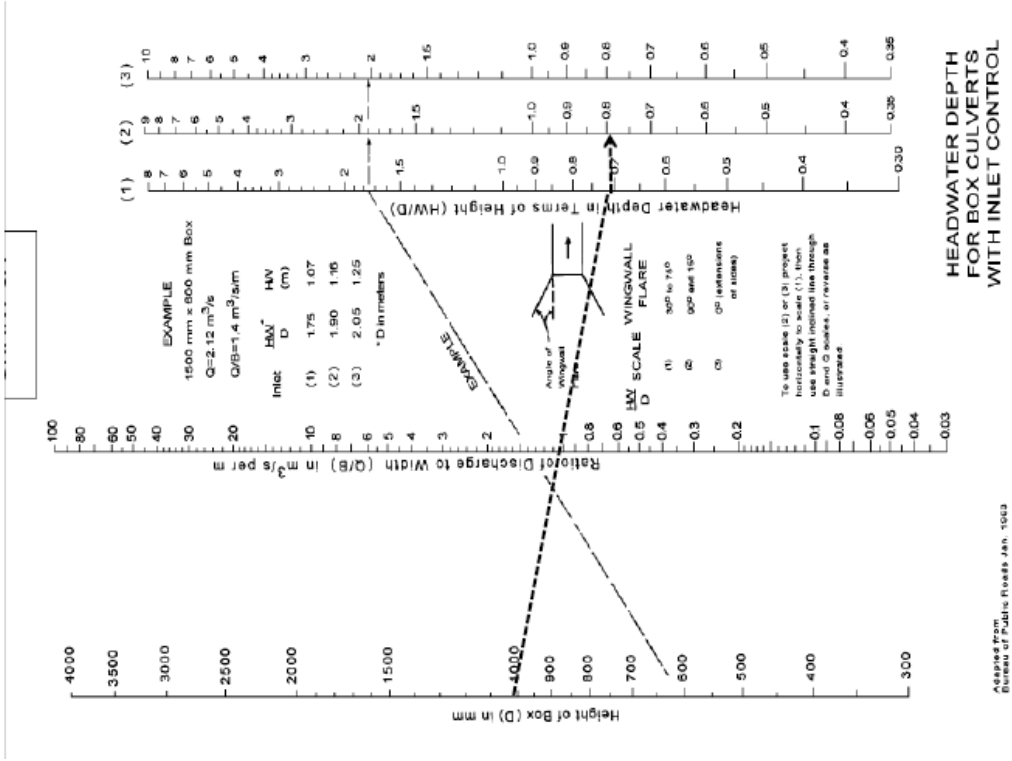
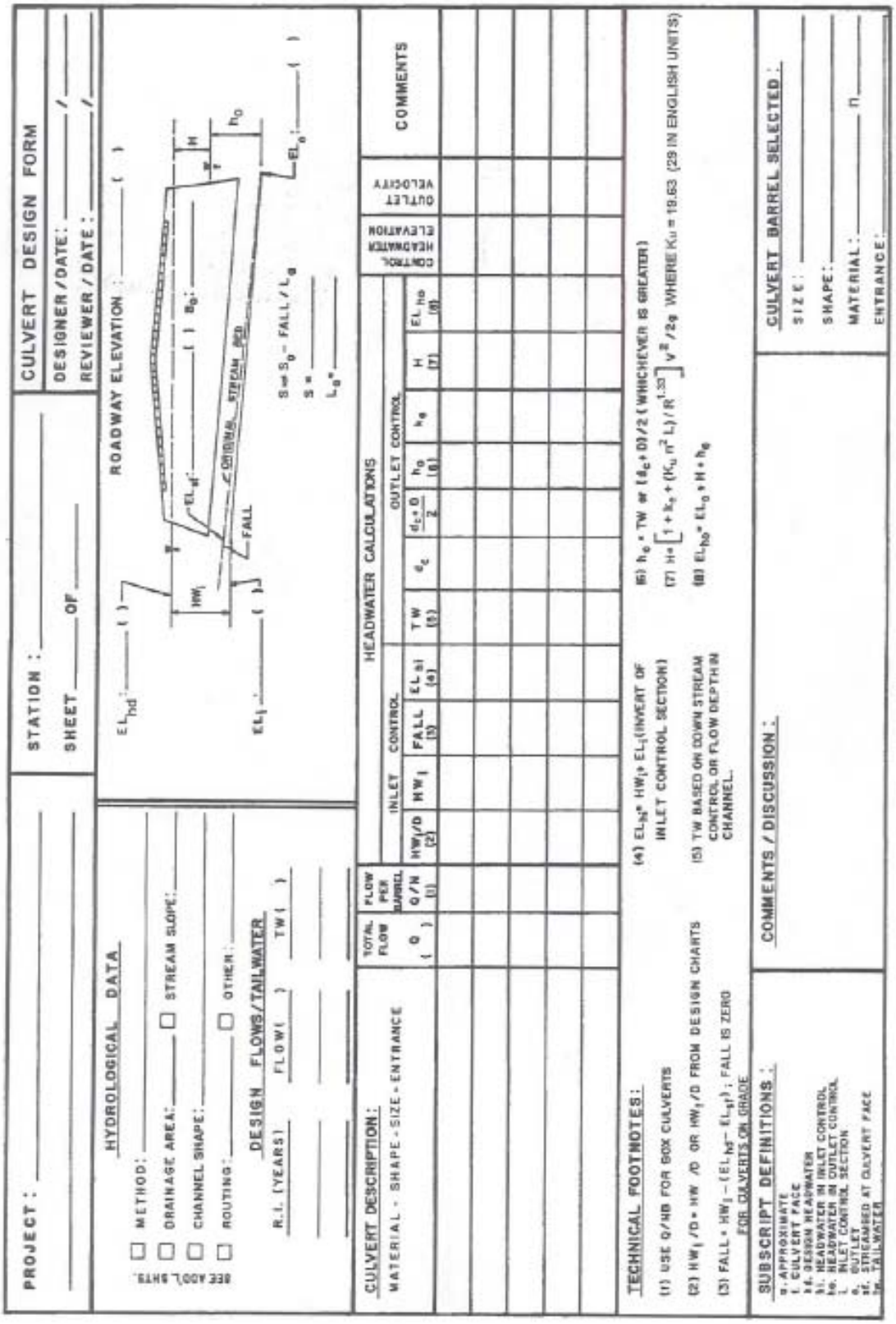


CHART 14 A
dc CANNOT EXCEED TOP OF PIPE
Q/B
Critical Depth-dc (m)
B = base of section (m)
Q = flow (m³/s)
dc = 0.467 (Q/B)^0.6
dc CANNOT EXCEED TOP OF PIPE
Q/B
Critical Depth-dc (m)
Adapted from Bureau of Public Roads
Critical Depth-Rectangular Section


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 600 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
EXAMPLE
Angle of Wingwall Flare
HW/D SCALE WINGWALL FLARE
(1) 90° to 75°
(2) 90° and 180°
(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
Adapted from Bureau of Public Roads Jan. 1963
HEADWATER DEPTH FOR BOX CULVERTS WITH INLET CONTROL

Culvert Design Form



PROJECT : _____

STATION : _____

SHEET _____ OF _____

CULVERT DESIGN FORM

DESIGNER / DATE: _____ / _____

REVIEWER / DATE : _____ / _____

HYDROLOGICAL DATA

☐ METHOD: _____

☐ DRAINAGE AREA: _____ ☐ STREAM SLOPE: _____

☐ CHANNEL SHAPE: _____

☐ ROUTING: _____ ☐ OTHER: _____

SEE ADDTL. SHTS.

DESIGN FLOWS/TAILWATER

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

_____ _____ _____

ROADWAY ELEVATION : _____ ( )

EL_{hd}: _____ ( )

HW_i

EL_{id}: _____ ( ) B_o: _____

FALL

ORIGINAL STREAM BED

EL_i: _____ ( )

h_o

H

EL_o: _____ ( )

$S = S_o$ - FALL / L_a

S = _____

L_a = _____

CULVERT DESCRIPTION:

MATERIAL - SHAPE - SIZE - ENTRANCE

HEADWATER CALCULATIONS

TOTAL FLOW Q ( )	FLOW PER BARREL Q/N (1)	INLET CONTROL HW/D (2)	INLET CONTROL HW_i	INLET CONTROL FALL (3)	INLET CONTROL EL_{si} (4)	TW (5)	OUTLET CONTROL d_c	OUTLET CONTROL $\frac{d_c+D}{2}$	OUTLET CONTROL h_o (6)	OUTLET CONTROL h_e	OUTLET CONTROL H (7)	OUTLET CONTROL 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_{id}) ; FALL IS ZERO FOR CULVERTS ON GRADE

(4) EL_{hd} = 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 Ku = 19.63 (29 IN ENGLISH UNITS)

(8) EL_{ho} = EL_o + H + h_e

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
st. 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

$EL_{hi}= HW_i+EL_i$

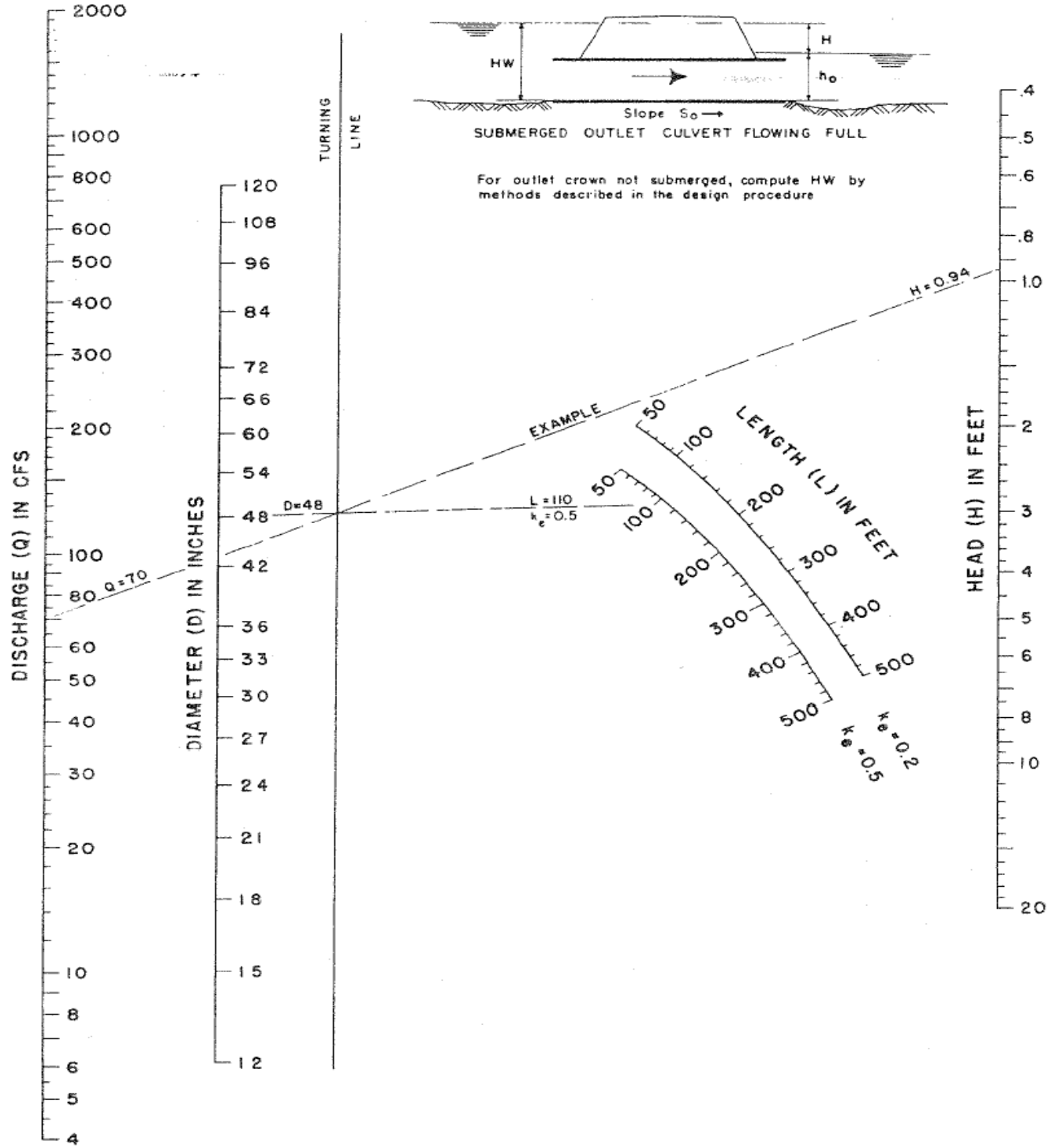
$HW_i= D[HW/D]$

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

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				$EL_o=EL_i - S_o *L$											
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 اورو 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 تحت 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 سئل		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						

97	1	Q /barrel=Q/(N*B)=	3.26928	cm/sm	N= nose of barrel B= wide of barrel	
98	2.a	Hwi/D	3.048		from chart A8 acurdance to Q/B(sqm/sca)	
99	2.b	Hwi	3.048	m		
100	3	Fall	0			
101	4	Elhi=Hwi+Eli	33.528	m	inlet control head water elevation	
102	5.a	TW	0.51	m		
103	5.b	dc	1	m	chart 14A acurdance to Q/B(sqm/sca	
104	5.c	(dc+D)/2	1	m		
105	6.a	greater TW or (dc+D)/2,Ho=He	1	m	*Reference: U.S. Department of Transportation Federal Highway Administration.*	
106	6.b	ko=	0.5		haydrolic	
107	7	H=	28.87	m	by this (H=[1+Ke+(ku*n^2*L/R^1.33)]*(V^2/2g) farmula	
108	8 outlet control elvation	Elho=Elo+H+He	25.45	m		
109	inletecontrol headwater elevation		33.528			
110	full box outlet velocity	Q/(B*D)	3.26928	m/s		
111	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

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.

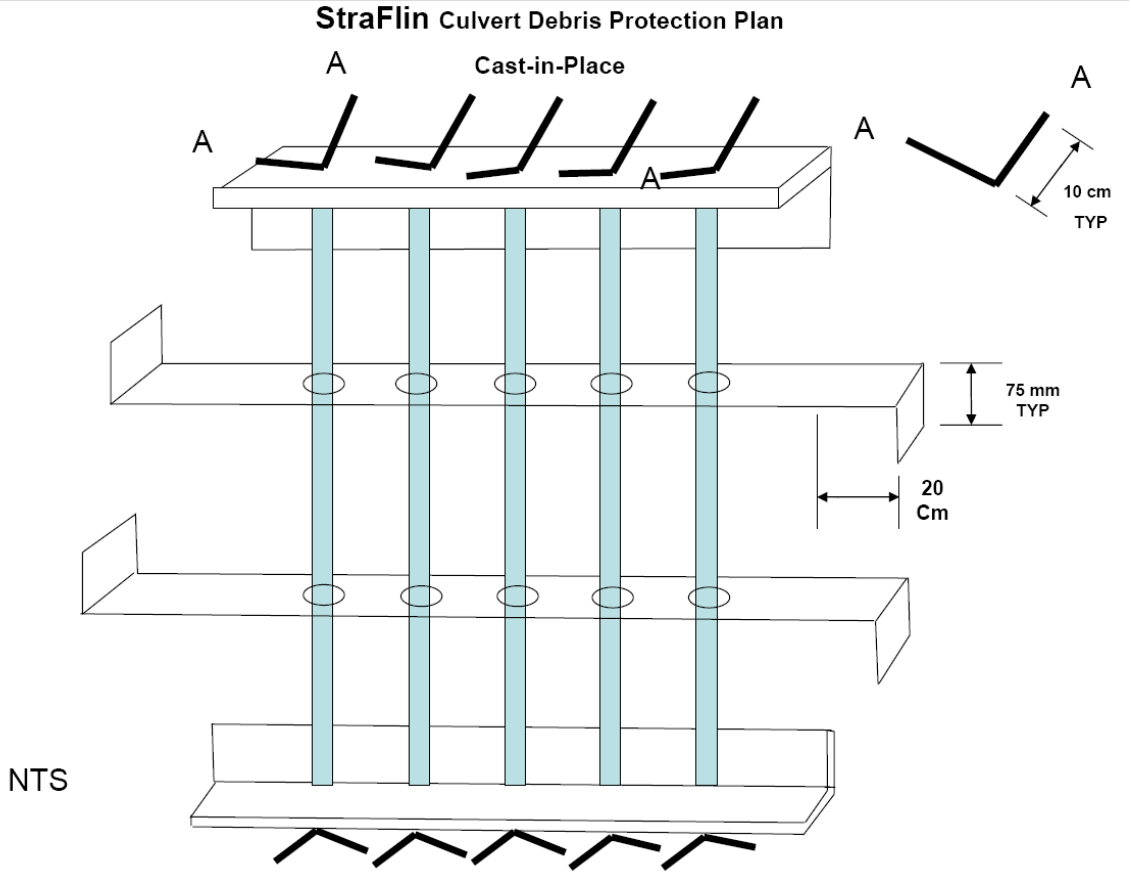
Table 4-3 Manning's n Values

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	2 2/3 by 1/2-inch corrugations	0.027-0.022
Metal Pipes and	6 by 1-inch corrugations	0.025-0.022
Boxes, Annular	5 by 1-inch corrugations	0.026-0.025
Corrugations	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	2 2/3 by 1/2-inch corrugated	
Metal Pipes,	24-inch plate width	0.024-0.012
Helical Corrugations,		
Full Circular Flow		
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)

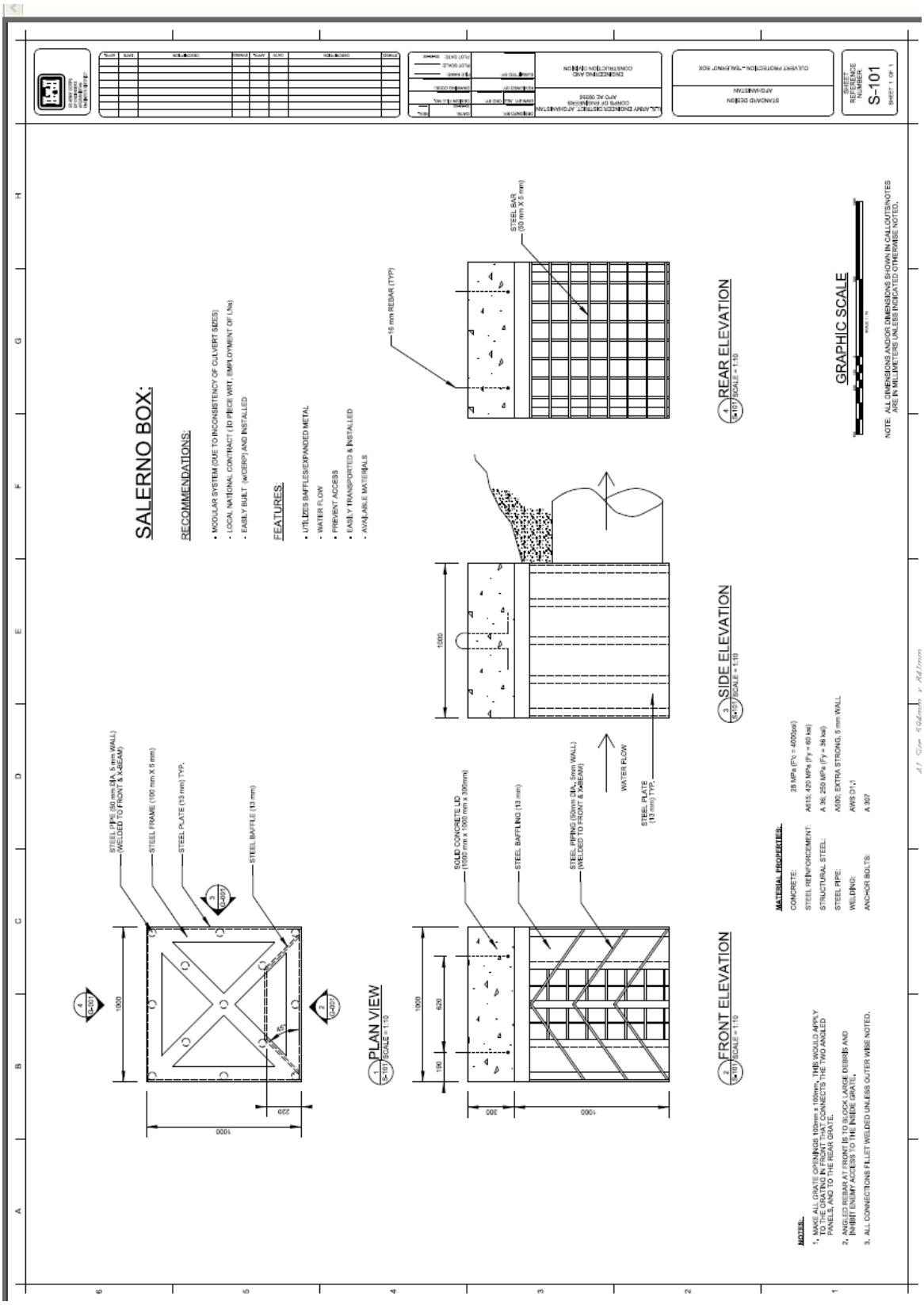
Designed by:

William Stratton

CPT Darrell Flinn

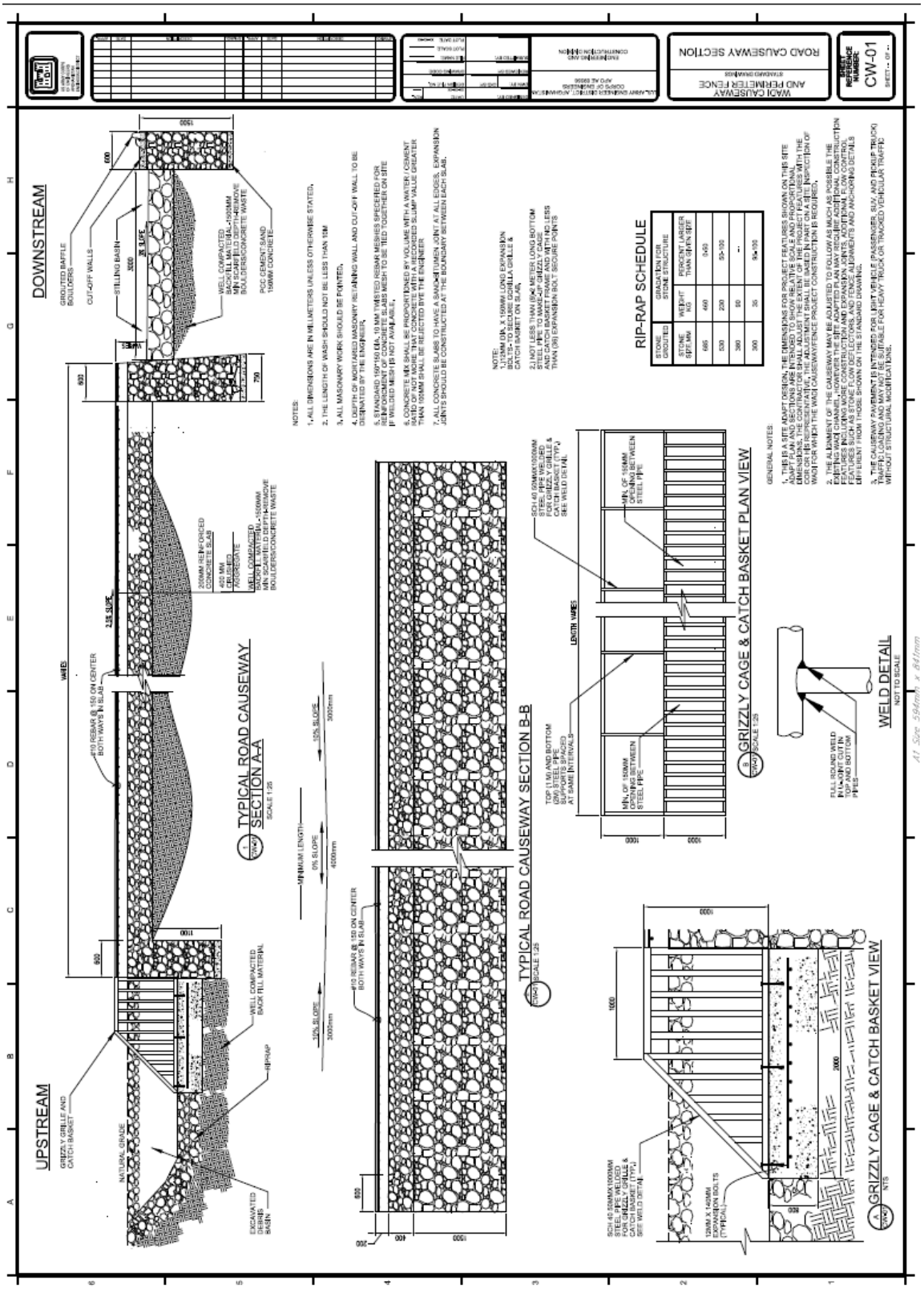
Mehtarlam PRT

Site Adapt SALERNO BOX

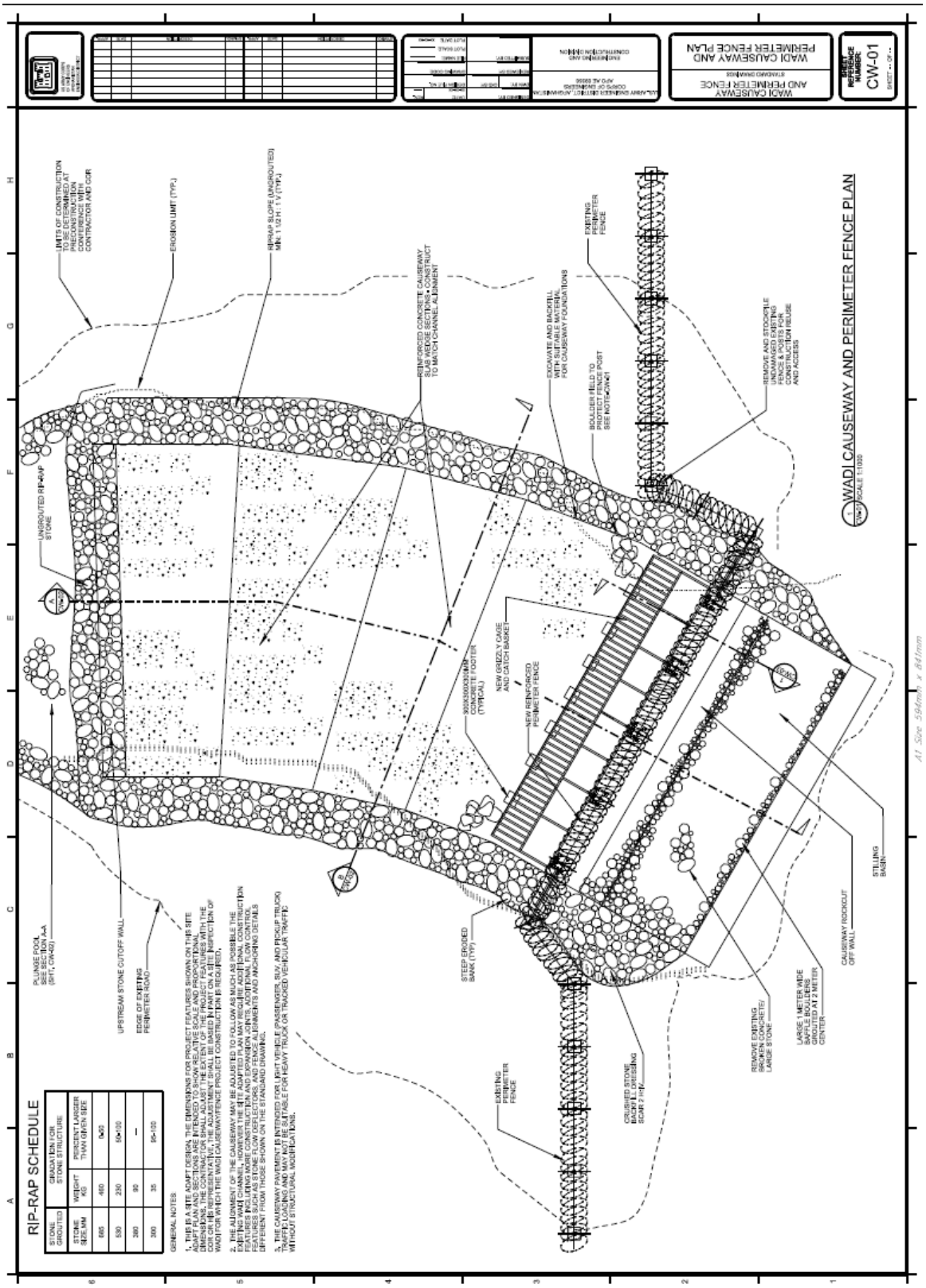


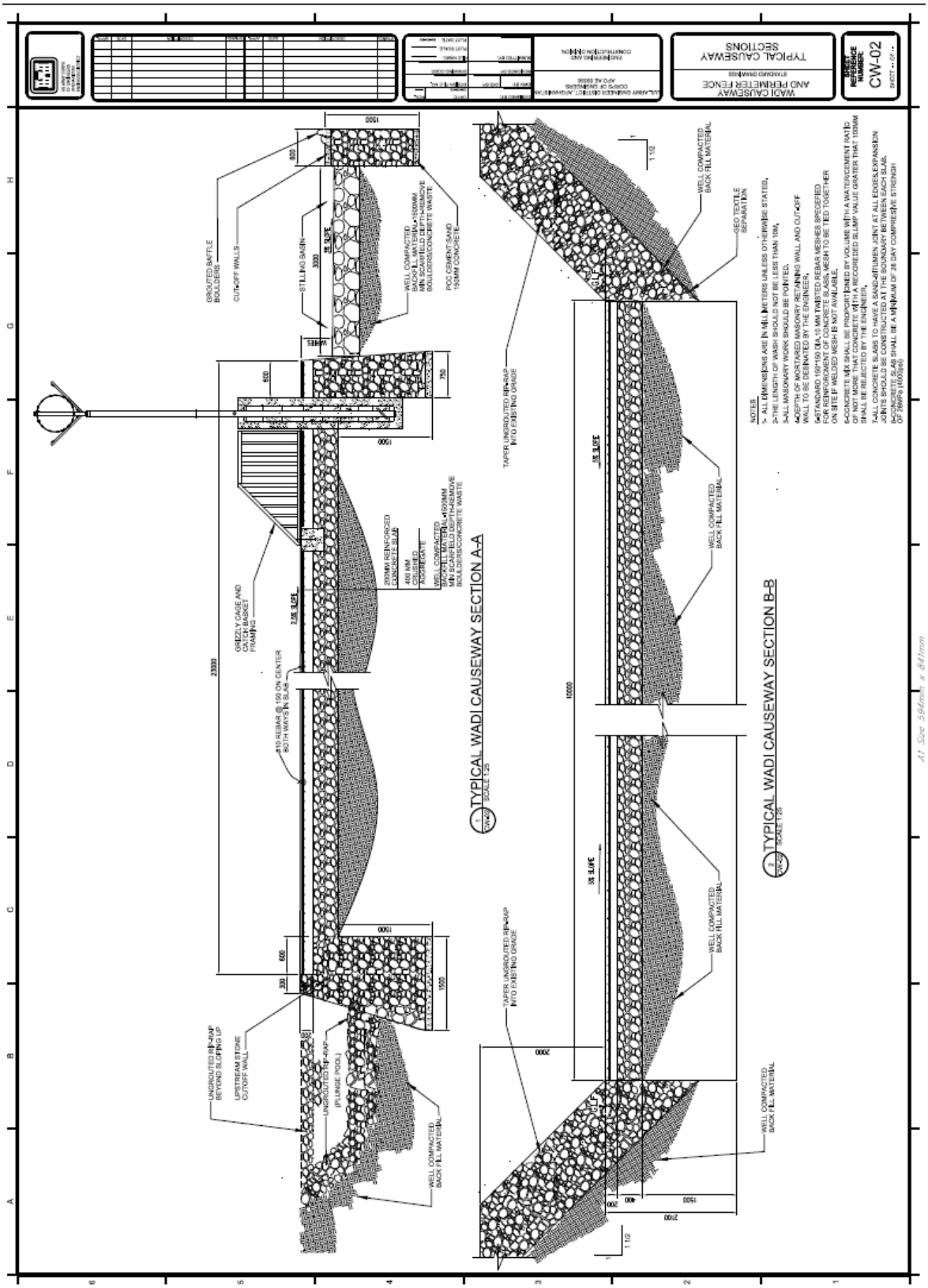
Appendix D – Causeway Debris Protection Devices

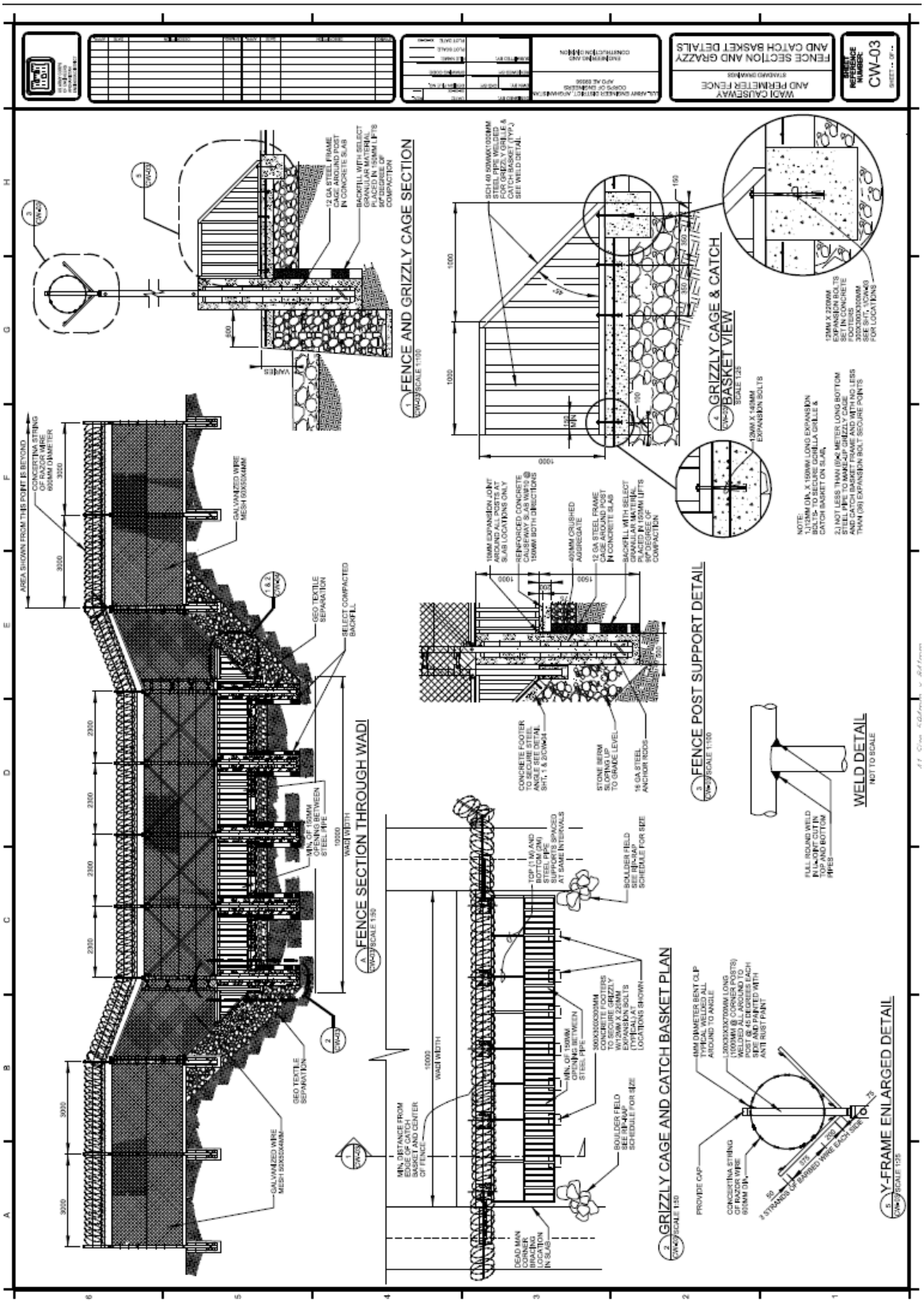
Site Adapt Roadway Causeway with Grizzly Cage

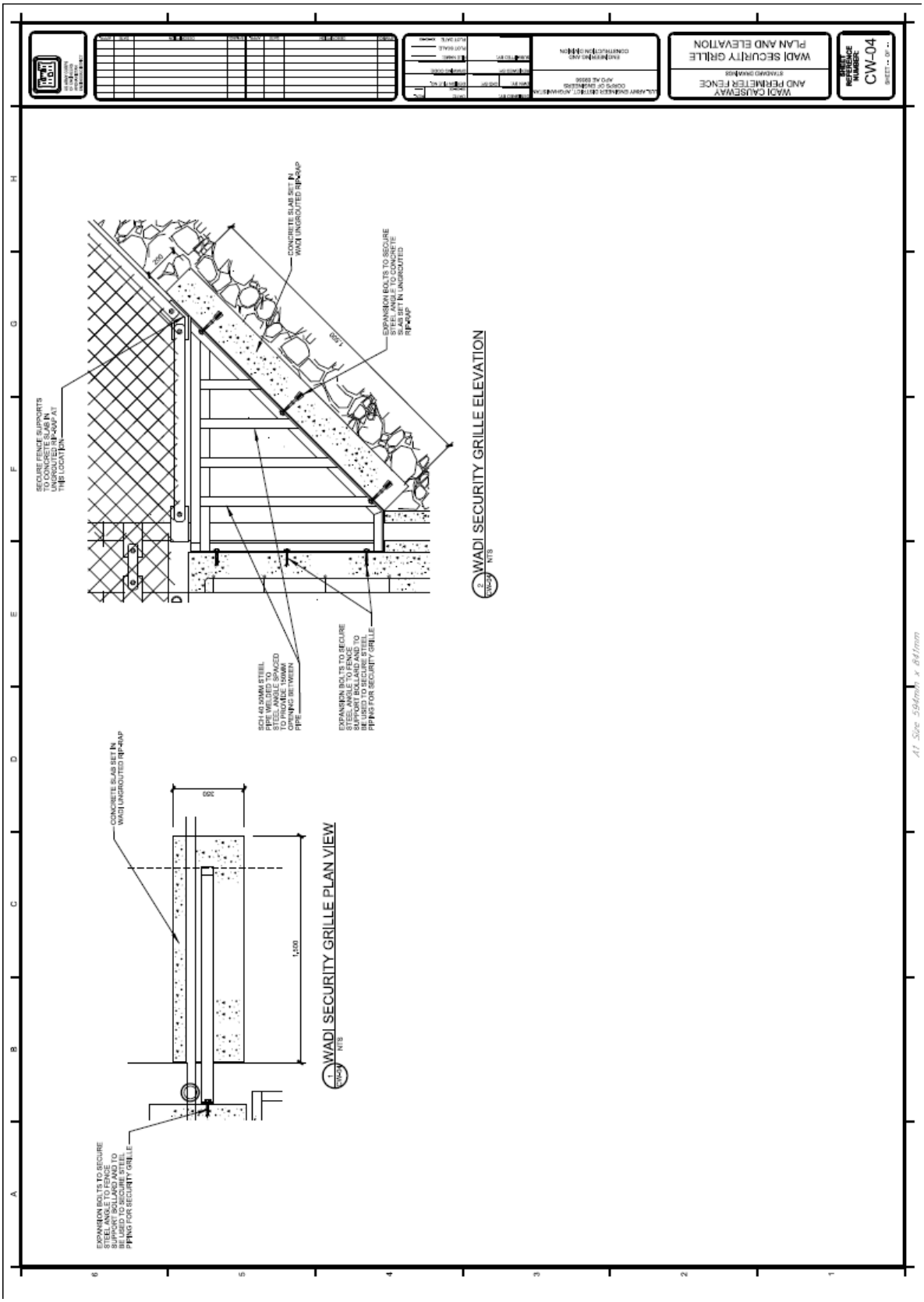

WMDI CAUSEWAY AND PERIMETER FENCE
ROAD CAUSEWAY SECTION
CW-01
DOWNSTREAM
UPSTREAM
TYPICAL ROAD CAUSEWAY SECTION A-A
TYPICAL ROAD CAUSEWAY SECTION B-B
GRIZZLY CAGE & CATCH BASKET PLAN VIEW
GRIZZLY CAGE & CATCH BASKET VIEW
WELD DETAIL
RIP-RAP SCHEDULE
NOTES:
GENERAL NOTES:
A1 Size 594mm x 841mm

Site Adapt Perimeter Fence Wadi Causeway with Grizzly Cage




WADI CAUSEWAY AND PERIMETER FENCE
STANDARD DRAWINGS
TYPICAL CAUSEWAY SECTIONS
CW-02
GROUTED BAFFLE BOULDERS
CUTOFF WALLS
STILLING BASIN
WELL COMPACTED BACKFILL MATERIAL
MIN SCARIFIED DEPTH=REMOVE BOULDERS/CONCRETE WASTE
PCC CEMENT SAND 150MM CONCRETE
GREIZY CAGE AND CATCH BASKET FRAMING
200MM REINFORCED CONCRETE SLAB
400 MM CRUSHED AGGREGATE
#10 REBAR @ 150 ON CENTER BOTH WAYS IN SLAB
TYPICAL WADI CAUSEWAY SECTION A-A
TYPICAL WADI CAUSEWAY SECTION B-B
TAPER UNGROUTED RIPRAP INTO GABIONS GRADE
WELL COMPACTED BACK FILL MATERIAL
GEO TEXTILE SEPARATION
UNGROUTED RIPRAP BEYOND SLOPING UP
UPSTREAM STONE CUTOFF WALL
UNGROUTED RIPRAP (PLUNGE POOL)
NOTES
1- ALL DIMENSIONS ARE IN MILLIMETERS UNLESS OTHERWISE STATED,
2-THE LENGTH OF WASH SHOULD NOT BE LESS THAN 10M,
3-ALL MASONARY WORK SHOULD BE POINTED.
4-DEPTH OF MORTARED MASONRY RETAINING WALL AND CUTOFF WALL TO BE DESINATED BY THE ENGINEER,
5-STANDARD 150*150 DIA.-10 MM TWISTED REBAR MESHES SPECIFIED FOR REINFORCEMENT OF CONCRETE SLABS, MESH TO BE TIED TOGETHER ON SITE IF REQUIRED MESH IS NOT AVAILABLE.
6-CONCRETE MIX SHALL BE PROPORTIONED BY VOLUME WITH A MINIMUM OF 300 KG OF CEMENT PER CUBIC METER OF CONCRETE. THE CONCRETE SLUMP VALUE GREATER THAN 100MM SHALL BE REJECTED BY THE ENGINEER,
7-ALL CONCRETE SLABS TO HAVE A SAND-BITUMEN JOINT AT ALL EDGES.EXPANSION JOINTS SHOULD BE CONSTRUCTED AT THE BOUNDARY BETWEEN EACH SLAB.
8-CONCRETE SLAB SHALL BE A MINIMUM OF 28 DAY COMPRESSIVE STRENGH OF 35MPa (4000psi)
A1 Size 594mm x 841mm


WADI CAUSEWAY
AND PERIMETER FENCE
STANDARD DRAWINGS
FENCE SECTION AND GRIZZLY
AND CATCH BASKET DETAILS
SHEET REFERENCE NUMBER:
CW-03
FENCE AND GRIZZLY CAGE SECTION
GRIZZLY CAGE & CATCH BASKET VIEW
FENCE POST SUPPORT DETAIL
WELD DETAIL
NOT TO SCALE
FENCE SECTION THROUGH WADI
GRIZZLY CAGE AND CATCH BASKET PLAN
Y-FRAME ENLARGED DETAIL
CONCERTINA STRING OF RAZOR WIRE 600MM DIAMETER
GALVANIZED WIRE MESH 60X50X4MM
GEO TEXTILE SEPARATION
SELECT COMPACTED BACKFILL
12 GA STEEL FRAME CAGE AROUND POST IN CONCRETE SLAB
BACKFILL WITH SELECT GRANULAR MATERIAL PLACED IN 300MM LIFTS 90°DEGREE OF COMPACTION
SCH 40 50MMX100MM STEEL POSTS SET FOR GRIZZLY GRILLE & CATCH BASKET (TYP.) SEE WELD DETAIL
150MM EXPANSION JOINT AROUND POSTS NOT SLAB LOCATIONS ONLY
REINFORCED CONCRETE CRUSERWAY SLAB WIP10 @ 150MM BOTH DIRECTIONS
40MM CRUSHED AGGREGATE
CONCRETE FOOTER TO SECURE STEEL ANGLE SEE DETAIL SHT. 1 & 2CW-04
STONE WITH SLOPE WALL TO GRADE LEVEL
16 GA STEEL ANCHOR RODS
BOULDER FIELD SEE RIP-RAP SCHEDULE FOR SIZE
TOP 1 1/4" AND BOTTOM (2M) STEEL PIPE SUPPORTED ANCHORED AT SAME IN INTERVALS
MIN. OF 150MM OPEN SPACE BETWEEN STEEL PIPE
300X300X200MM CONCRETE FOOTERS TO SECURE GRIZZLY WITH 4 X 20MM EXPANSION BOLTS (TYPICAL) & LOCATIONS SHOWN
DEAD MAN CORNER BRACING LOCATION IN SLAB
MIN. DISTANCE FROM BOTTOM OF GRIZZLY BASKET AND CENTER OF FENCE
WADI WIDTH
4MM DIAMETER BENT CLIP TYPICAL WELDED ALL AROUND TO ANGLE
PROVIDE CAP
CONCERTINA STRING OF RAZOR WIRE 600MM DIA.
3 STRANDS OF BARBED WIRE EACH SIDE


WADI CALDERWAY AND PERIMETER FENCE
WADI SECURITY GRILLE PLAN AND ELEVATION
CW-04
SECURE FENCE SUPPORTS TO CONCRETE SLAB IN UNGROUTED RIP-RAP AT THIS LOCATION
CONCRETE SLAB SET IN WADI UNGROUTED RIP-RAP
EXPANSION BOLTS TO SECURE STEEL ANGLE TO CONCRETE SLAB SET IN UNGROUTED RIP-RAP
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
WADI SECURITY GRILLE ELEVATION
NTS
CONCRETE SLAB SET IN WADI UNGROUTED RIP-RAP
EXPANSION BOLTS TO SECURE STEEL ANGLE TO FENCE SUPPORT BOLLARD AND TO BE USED TO SECURE STEEL PIPING FOR SECURITY GRILLE
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

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AED DESIGN REQUIREMENTS
FOR
GEOTECHNCIAL INVESTIGATIONS
VARIOUS LOCATIONS,
AFGHANISTAN

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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 <u>at the project site</u> (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

Field Boring Log	**Project:**	**Boring #:**	**Page of**
County:	**Site:**	**Location:**	
Drilled by:	**Start Date:**	**Completion Date:**	**Total Depth:**
Drilling Method & Equipment:		**Logged by:**	

DEPTH (m)	**SPT Blow Count**			**SAMPLE**			**Litho logy Strip**	**Visual Description/Other Field Test**
				% Recovery	**Type**	**#**		

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	

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 1586	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/m³	SPT Blow Counts	Compresssion 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/m³	CBR	Compresssion index, C_c	Recompression 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) $\alpha_p = \tan^{-1}\left(\dfrac{-\Delta z}{\Delta x}\right) = \tan^{-1}\left(\dfrac{0.1}{0.8}\right) = 7.1°$

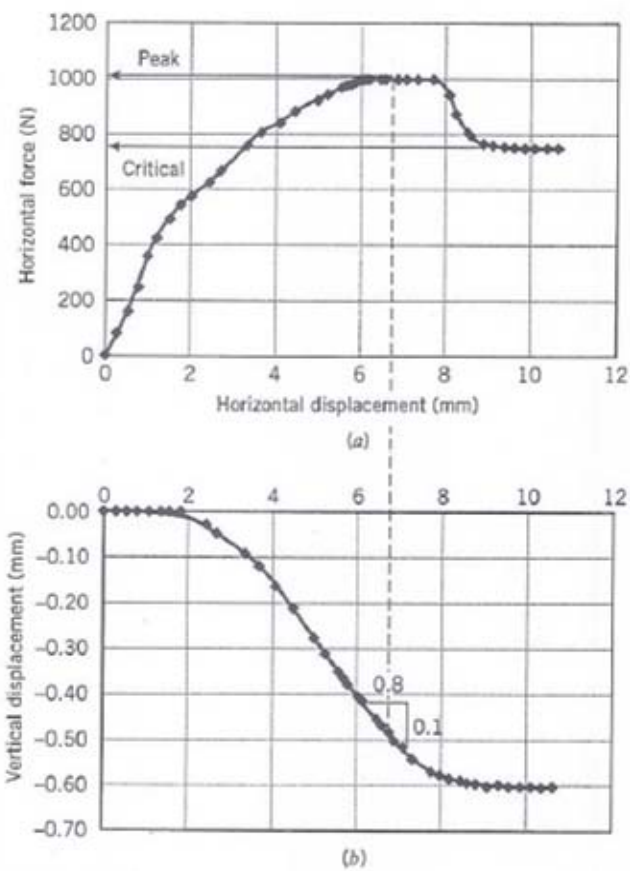

FIGURE E5.5

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.

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

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

Step 4: Determine α_p.

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

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

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

Note that as the normal force increases α_p decreases.

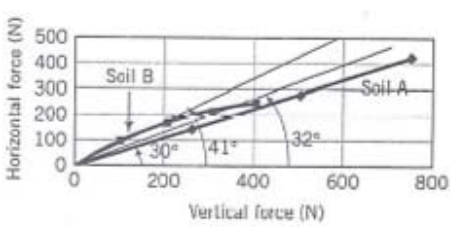

FIGURE E5.3

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:
Grease Trap Design

Various Locations,
Afghanistan

September 2009

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FOR
GREASE TRAP DESIGN
VARIOUS LOCATIONS,
AFGHANISTAN

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AED Design Requriements
Grease Trap Design

1. General

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

2. Grease Traps

Food-service operations typically use grease traps to trap fats, oils and grease (FOG) and to prevent excessive discharge of FOG into the wastewater collection and treatment system. If grease traps are not properly designed and maintained, slug loads of grease will interfere with the performance of both the collection and treatment system. Grease traps are generally located outside the food-service establishment in an underground tank with ground-level access. The traps use the physical principal that fats, oils and grease are lighter than water and will rise to the top of a water surface when the mixture is allowed to stand for a period of time in quiet conditions. A submerged pipe at the outlet of the tank allows water to exit the tank while retaining the FOG materials. .

3. Configuration

Grease traps usually consist of an underground, watertight, concrete tank with baffled inlet and outlet piping. The outlet pipe has a tee that allows the internal discharge to be located within 0.3 m of the tank bottom. The size of the grease trap depends on the anticipated flow rate, water temperature, and grease concentration. In general, grease traps range from a minimum capacity of 2.8 m^3 to a maximum capacity of 4.7 m^3. Where a capacity of more than 4.7 m^3 is required, two or more grease traps may be placed in a series. Access to the tank is typically through one or two manhole rings and covers. Grease traps should be located outside food-service buildings in an accessible location for inspection and maintenance. The traps are installed in the waste line between the sink drains, floor drains and kitchen fixtures and the wastewater collection system.

4. Discharge to Grease Traps

Grease traps do not perform effectively if they receive discharges with elevated temperatures or high solids concentrations. Grease traps should not receive discharges from garbage grinders or produce-preparation sinks. Discharges from mechanical dishwashers are also not recommended. However, the pre-flush / pre-scraping sinks that serve mechanical dishwashers may be connected to the grease trap, provided no garbage grinders are used at these sinks. Kitchen food wastes and kitchen cleaning wastes are discharged from the building in a separate line which flows to the grease trap.

Grease traps do not receive sanitary waste from the facility. Sanitary wastes flow directly to the wastewater collection system through separate piping.

5. Design Factors

The design factors to consider when sizing a grease trap are as follows:

Flow rate of discharge to grease trap.

Retention time in the grease trap.

FOG concentration of the influent waste.

Emptying frequency.

6. Grease Trap Design

There are several methods of determining the size of a grease trap. The most common method is provided by the U.S. Environmental Protection Agency (EPA) and is as follows:

Trap size = (# seats)*(flow/meal)*(storage factor)*(hours open/2)

Where:

Seats = number of available seats in dining facility (not number of meals served)
Flow/meal = 0.02 m^3
Storage Factor = 1.7
Hours open = Preparation time +serving time cleaning time for EACH meal. For example, if a facility serves 3 meals per day and each meal requires 1 hour to prepare a meal, 1.5 hour to serve each meal, and 1 hour to clean the facility, then the total hours open would be 3 x (1.5 hour serving) = 4.5 hours.

A grease sized for a dining facility with 50 seats with the serving times listed above would be:

50 x 0.02 x 1.7 x (4.5 / 2) = 4.8 cubic meters

The liquid depth in the trap should be between 0.76 m and 1.82 meters and the length to width ratio should be a minimum of 3 to 1.

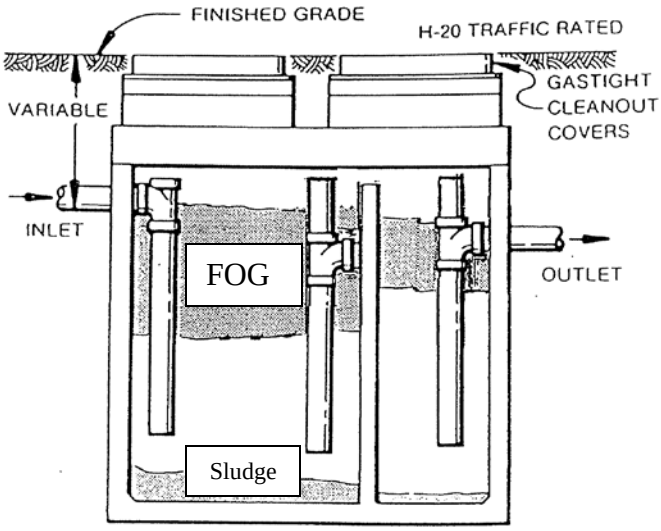
7. As-Builts

Upon completion of the installation, 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.

8. Reference

EPA, 625/1-80-012 (Design Manual: Onsite Wastewater Treatment and Disposal Systems)

Grease Trap / Grease Interceptor Design Layout



TYPICAL CROSS-SECTION — GREASE INTERCEPTOR

Notes:

1. Piping inside grease trap shall be 100mm diameter, PVC with solvent welded joints
2. Bottom of inlet and outlet piping shall be 300mm above floor of tank.
3. bottom of interior baffle / connection piping shall be 500mm above floor of tank.
4. Grease trap shall be pumped out regularly to prevent solids buildup and clogging.
5. Volume of inlet chamber shall be 3 times volume of outlet chamber.
6. Length-to-width ratio of tank configuration shall be 3:1.
7. Tank depth shall be between 0.76 – 1.82 m.
8. Minimum volume = 2.8 m^3
9. Maximum volume for a single grease trap = 4.7 m^3.

US Army Corps
of Engineers
Afghanistan Engineer District

AED Design Requirements:
Superelevation Road Design

Various Locations,
Afghanistan

MARCH 2009

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FOR
SUPERELEVATION ROAD DESIGN
VARIOUS LOCATIONS,
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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	V_d = 30 km/h	V_d = 40 km/h	V_d = 50 km/h	V_d = 60 km/h	V_d = 70 km/h	V_d = 80 km/h	V_d = 90 km/h	V_d = 100 km/h	V_d = 110 km/h	V_d = 120 km/h	V_d = 130 km/h
	R (m)	R (m)	R (m)	R (m)	R (m)	R (m)	R (m)	R (m)	R (m)	R (m)	R (m)	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						Roadway width = 6.6 m						Roadway width = 6.0 m					
	Design Speed (km/h)						Design Speed (km/h)						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.

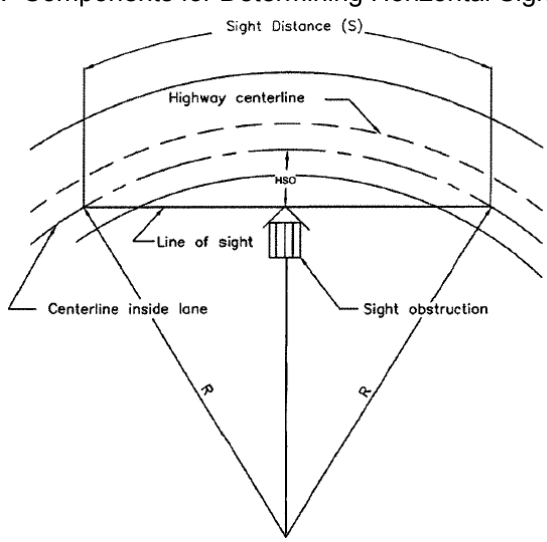
6. Stopping Sight Distance

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				
Design speed (km/h)	Brake reaction distance (m)	Braking distance on level (m)	Stopping sight distance	
			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

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AED DESIGN REQUIREMENTS
FOR
HYDROLOGY STUDY
VARIOUS LOCATIONS, AFGHANISTAN

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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

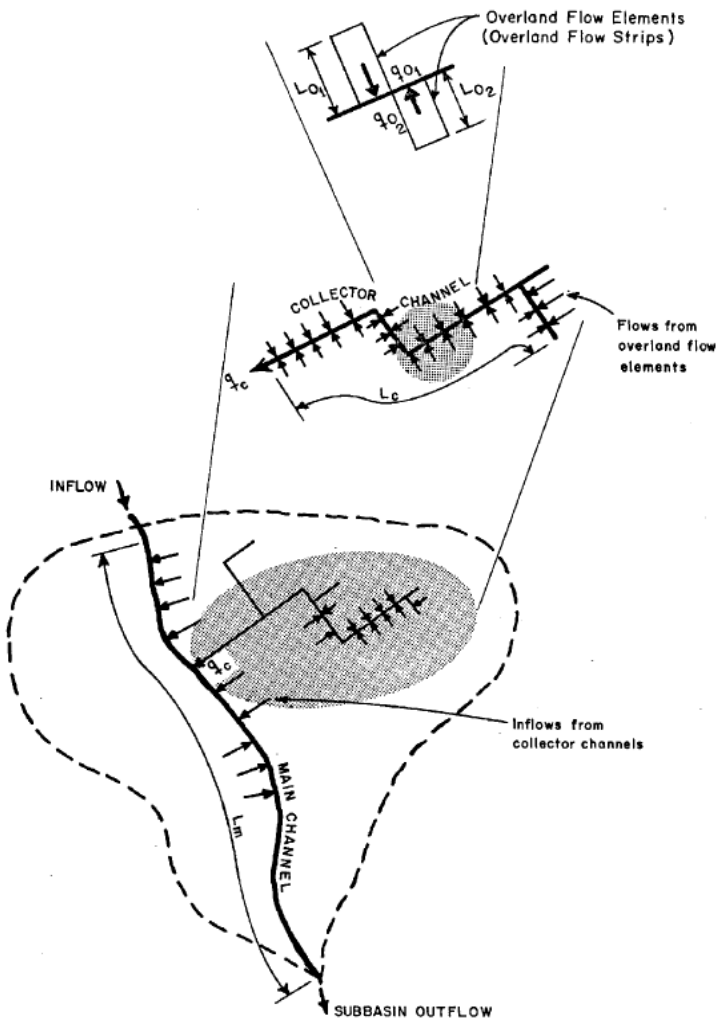
In regions where no I-D-F curve is available, the rainfall intensity may be calculated by the following formula:

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

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

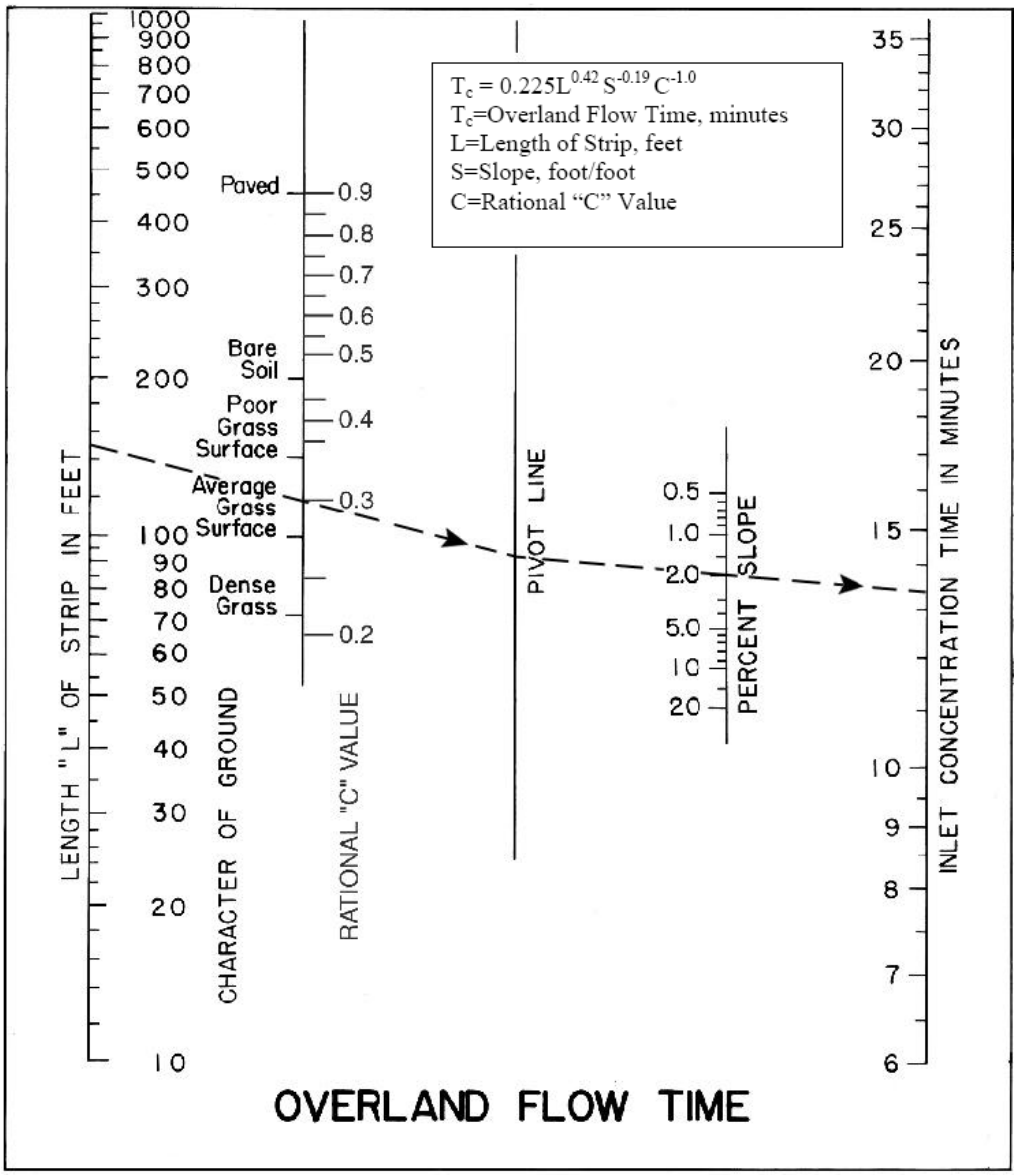
c) Time of Concentration. Time of concentration is the time for runoff to travel from the most hydraulically distant point in the watershed to the point of interest within the watershed. The time of concentration is the sum of the overland flow time, the shallow concentrated flow time and the channel 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

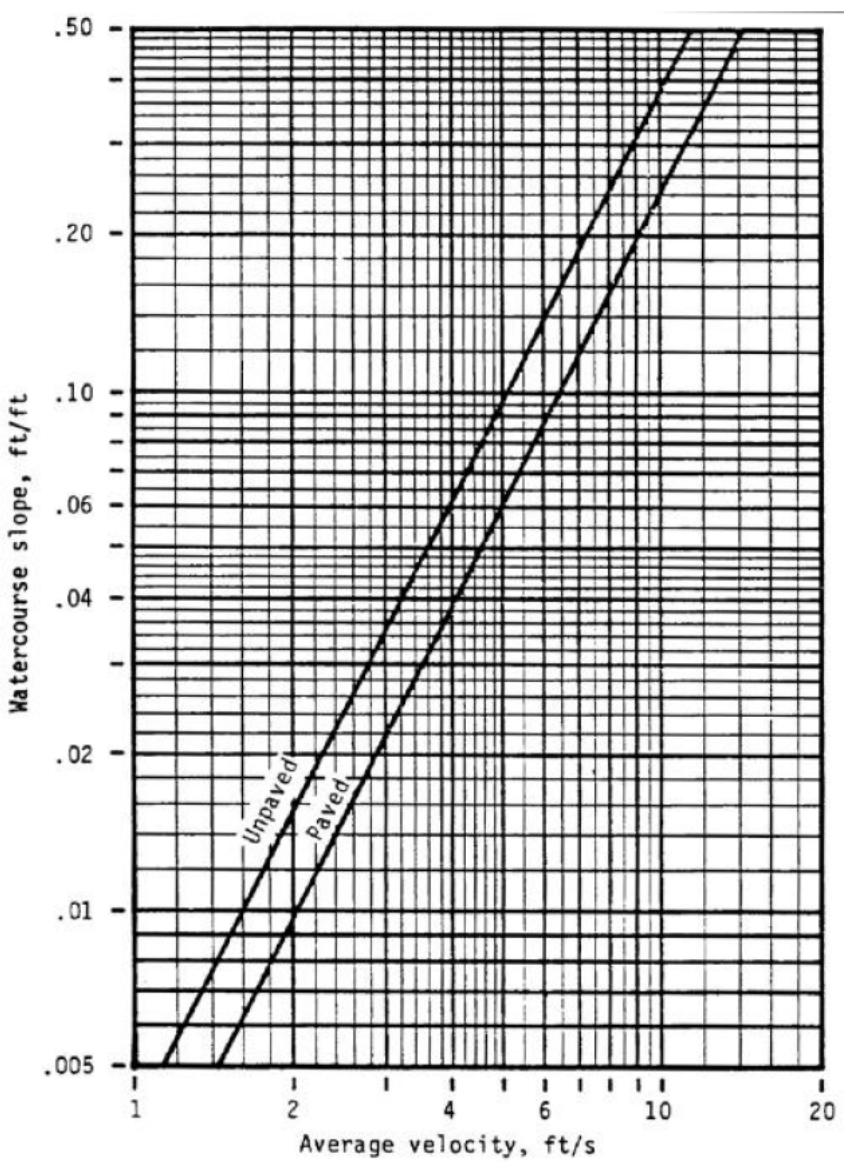


The overland flow time of concentration is determined by drawing a straight line through the flow length of the overland flow and the surface type or rational runoff coefficient and extending this line to the pivot line in the center of the nomograph. A line is then drawn from the intersecting point 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.

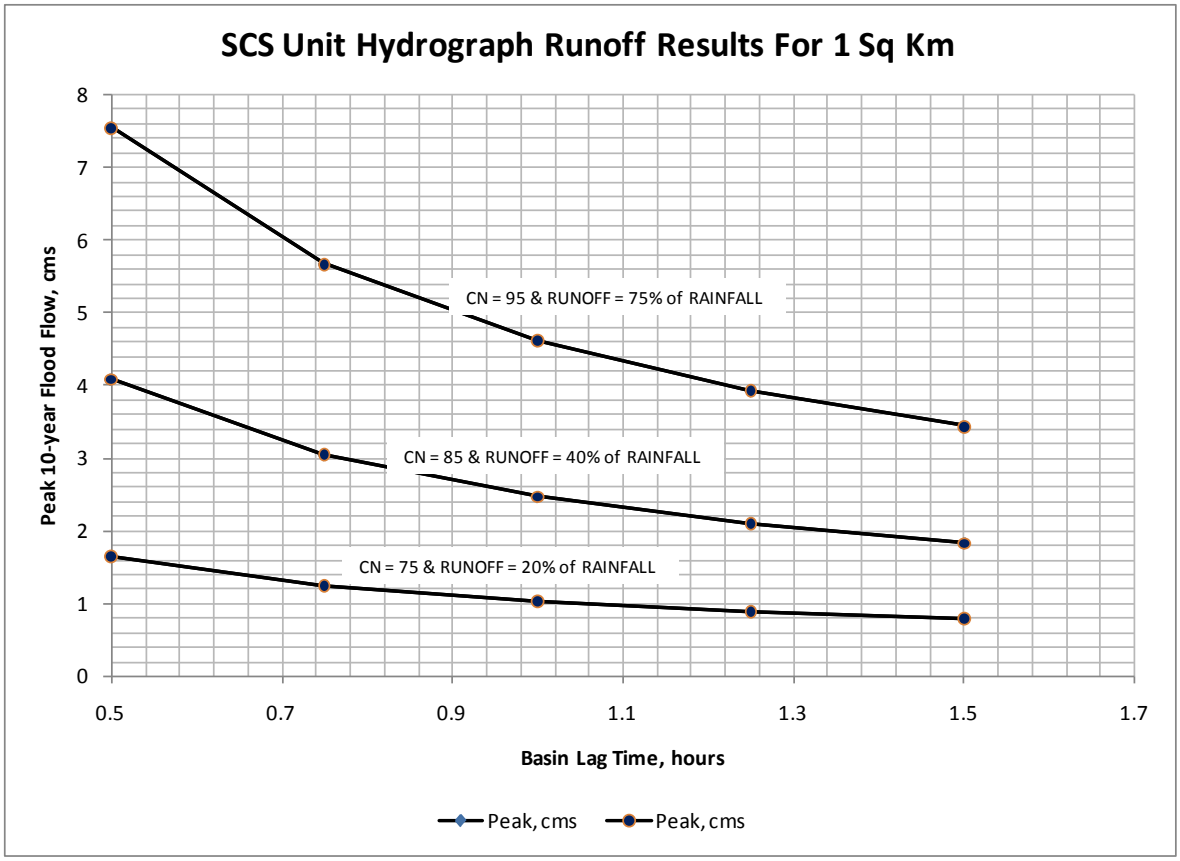
4.2. Soil Conservation Service (SCS) Unit Hydrograph Method

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

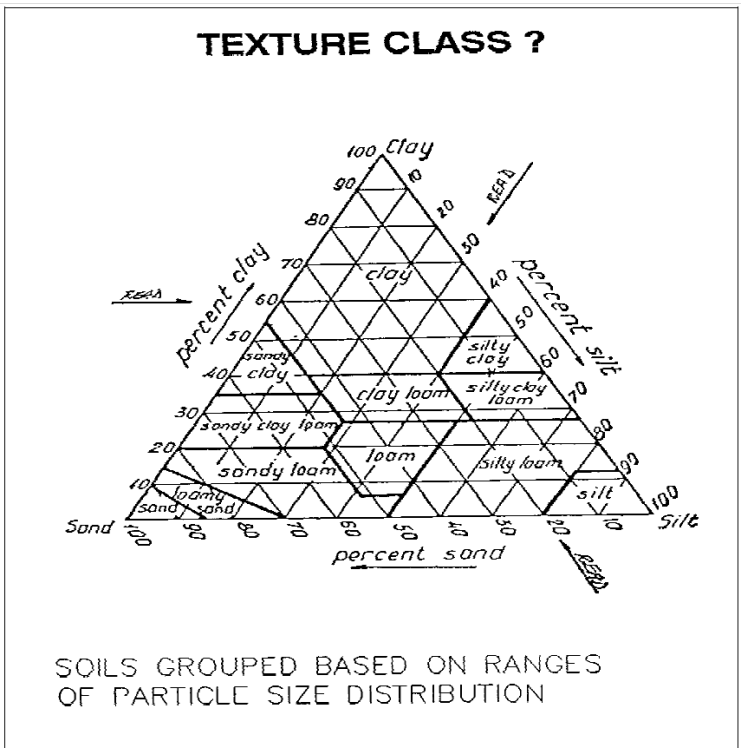
groups and land use are published in several sources.

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 5-5
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

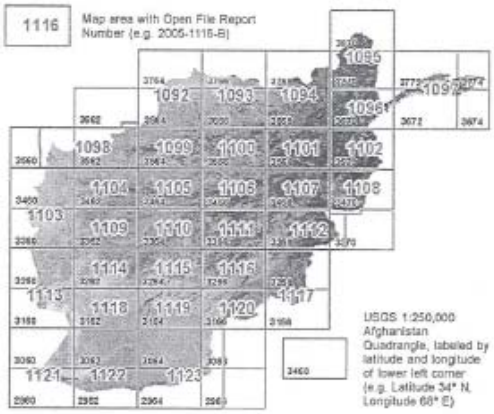
1. UFC 3 230 17FA Drainage for Areas Other than Airfields. Department of Defense, January 2004.
2. U.S. Army Corps of Engineers. Engineering and Design Flood hydrograph Analysis, EM 110-2-1417, August 1994. Found at http://140.194.76.129/publications/eng-manuals/em1110-2-1417/toc.htm
3. U.S. Army Corps 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.

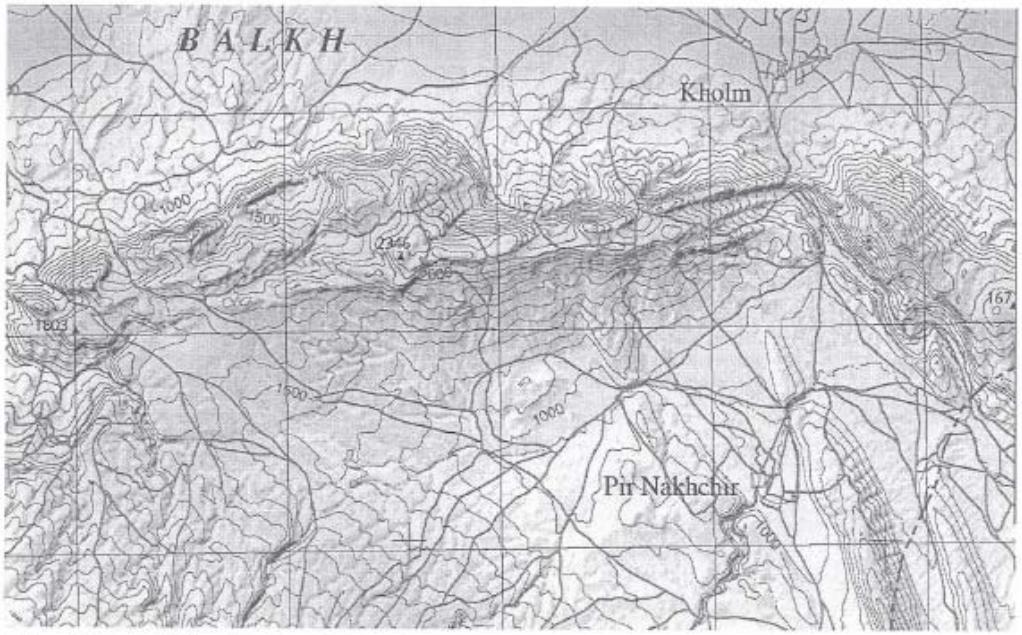
Topographic Map Index Information for

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.

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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, Maymayh (211), Jumarj-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-Chagmaqtin (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), Khanja-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-Band (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 (319) 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.

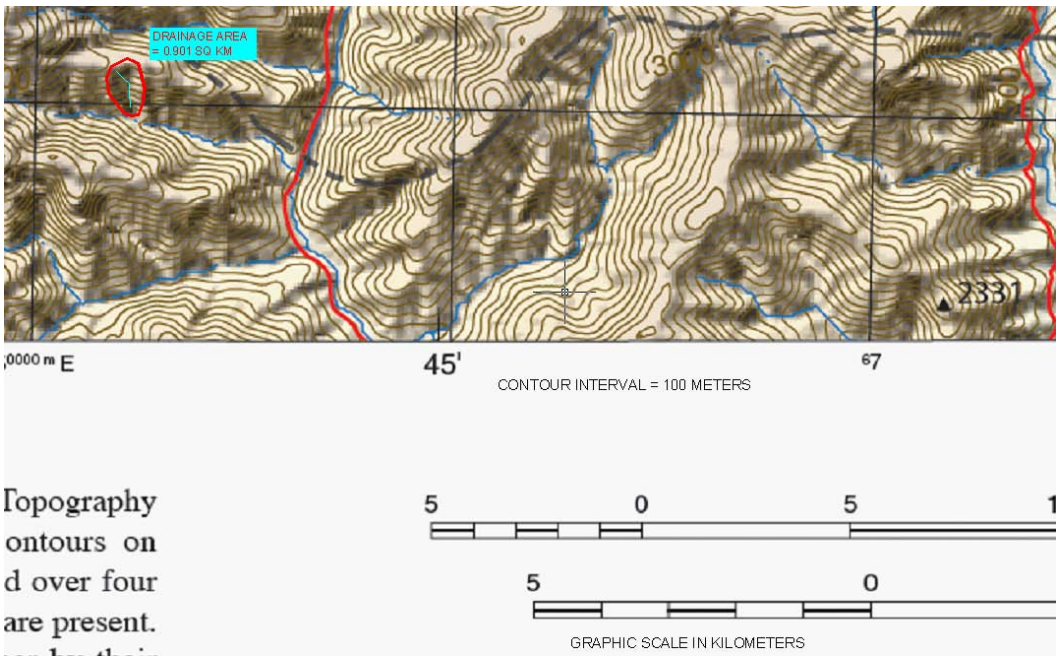
Topographic Map of Quadrangles 3062 and 2962, Charburjak (609), Khanneshin (610), Gawdezereh (615), and Galachah (616) Quadrangles, Afghanistan. U.S. Geological Survey Open File Report 2005-1122-B, compiled by Robert G. Bohannon.

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.

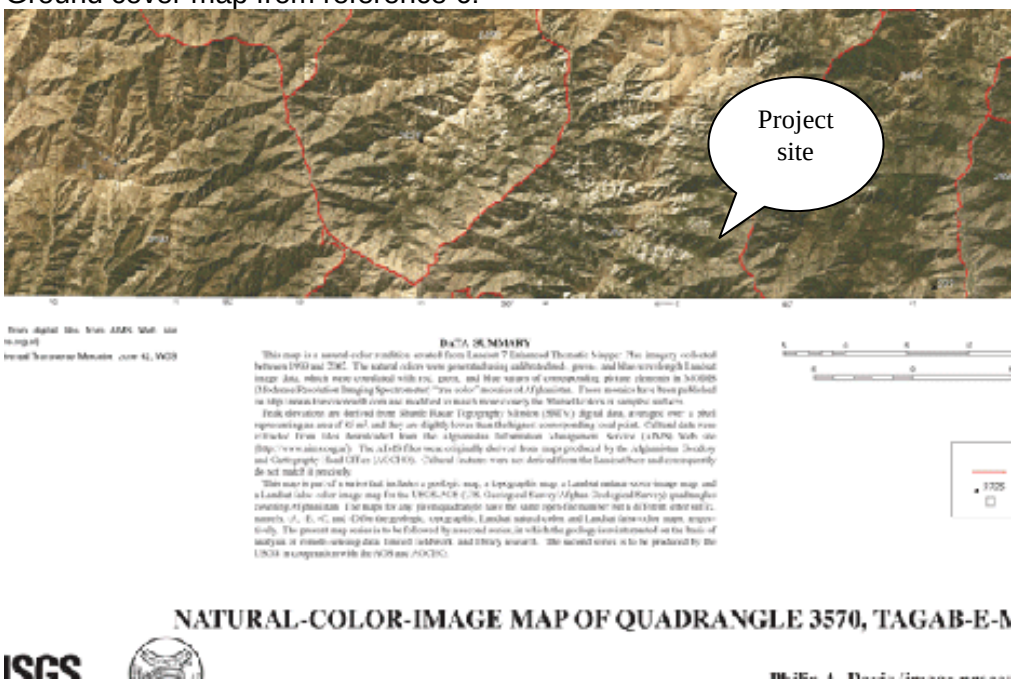
Appendix A Example of Rational Method

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:

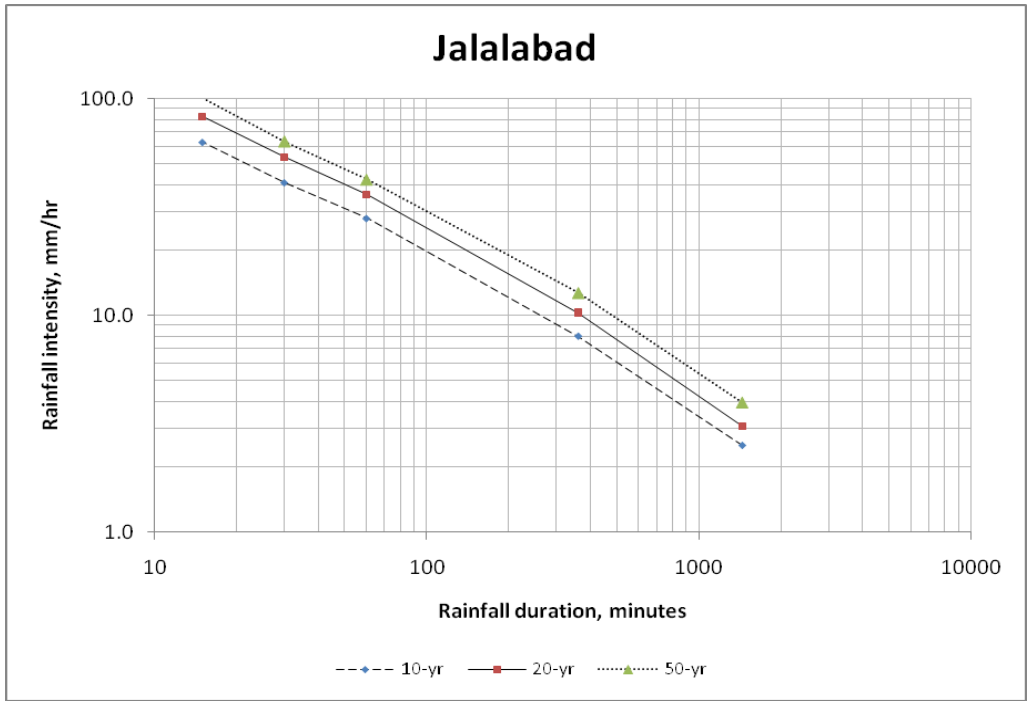
Drainage area map determined using AutoCAD



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 = $\frac{\Delta \text{ Elevation} = 984}{\text{Ls} = 1{,}273}$ 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 = $\frac{\Delta \text{ Elevation} = 1{,}312}{\text{Lc} = 1{,}804}$ Based on map contour over longest flow distance; Based on map measurement of channel length

Sc = 0.73

Tc = $\frac{1{,}804}{16.441*(0.73^{0.5603})}$ = 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

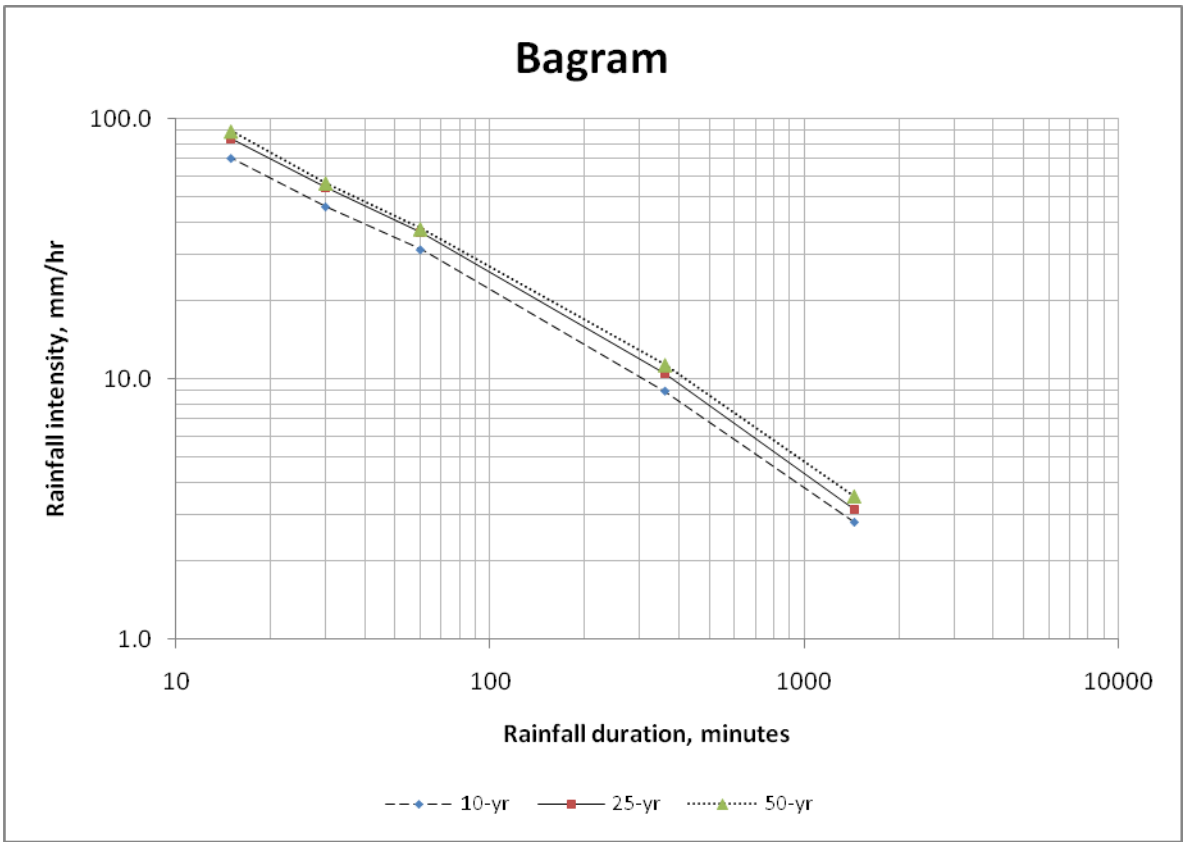
5 Ten year storm rainfall intensity = Based on Area Rainfall Intensity Hydrograph

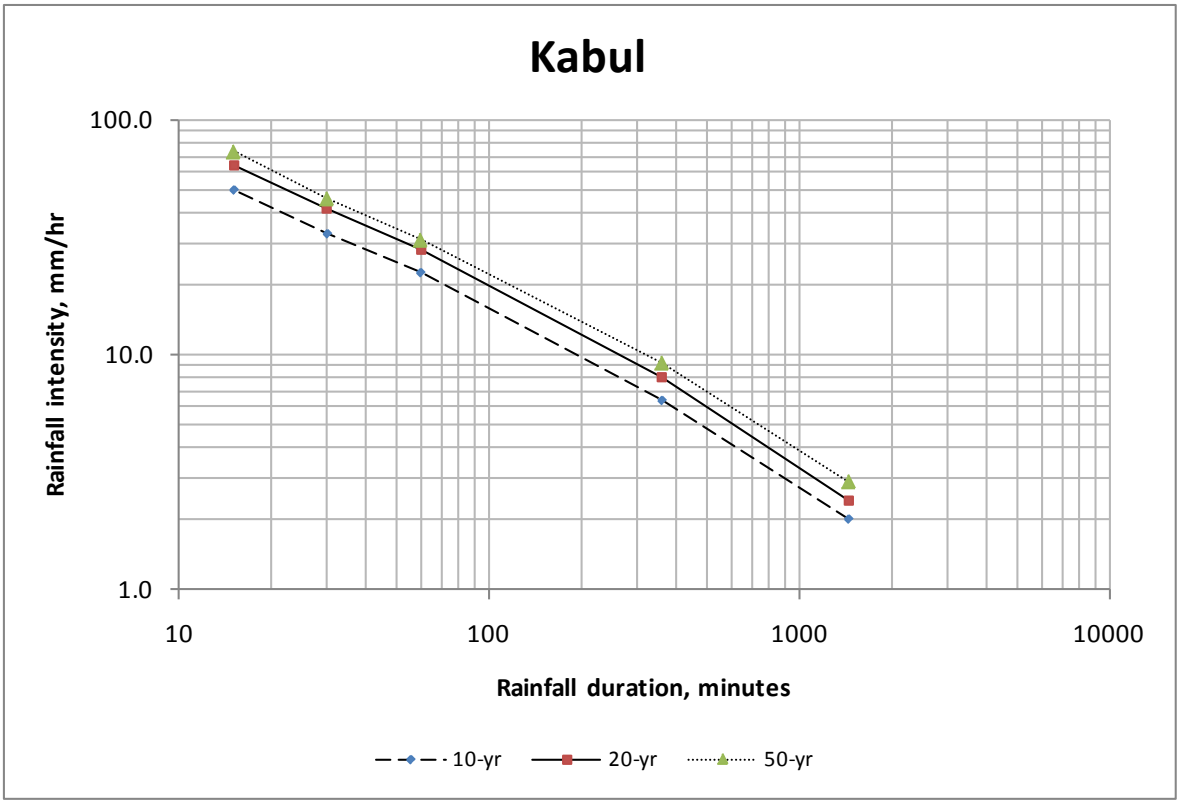
I = 62.0 mm/hr

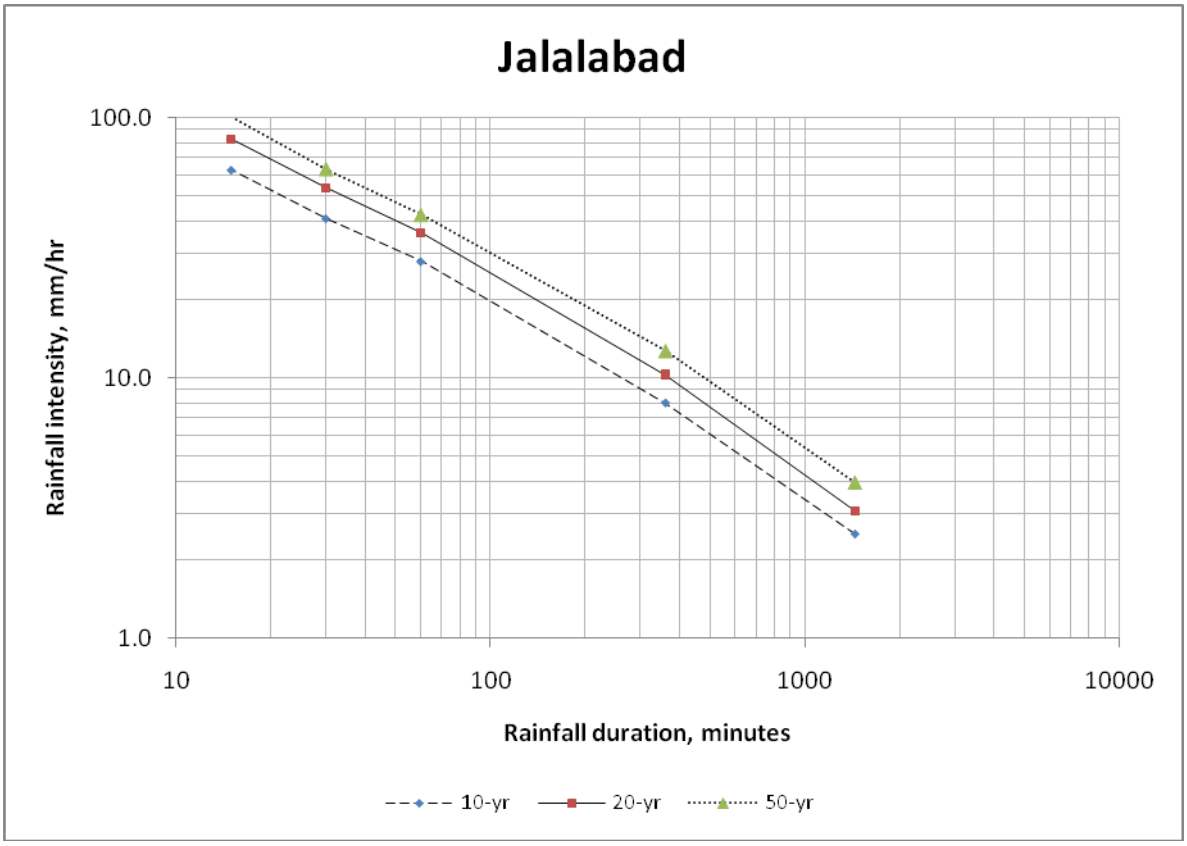
6 Q = Flow at time of concentration (ratinoal method) = Q = 0.278*C*A*I

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

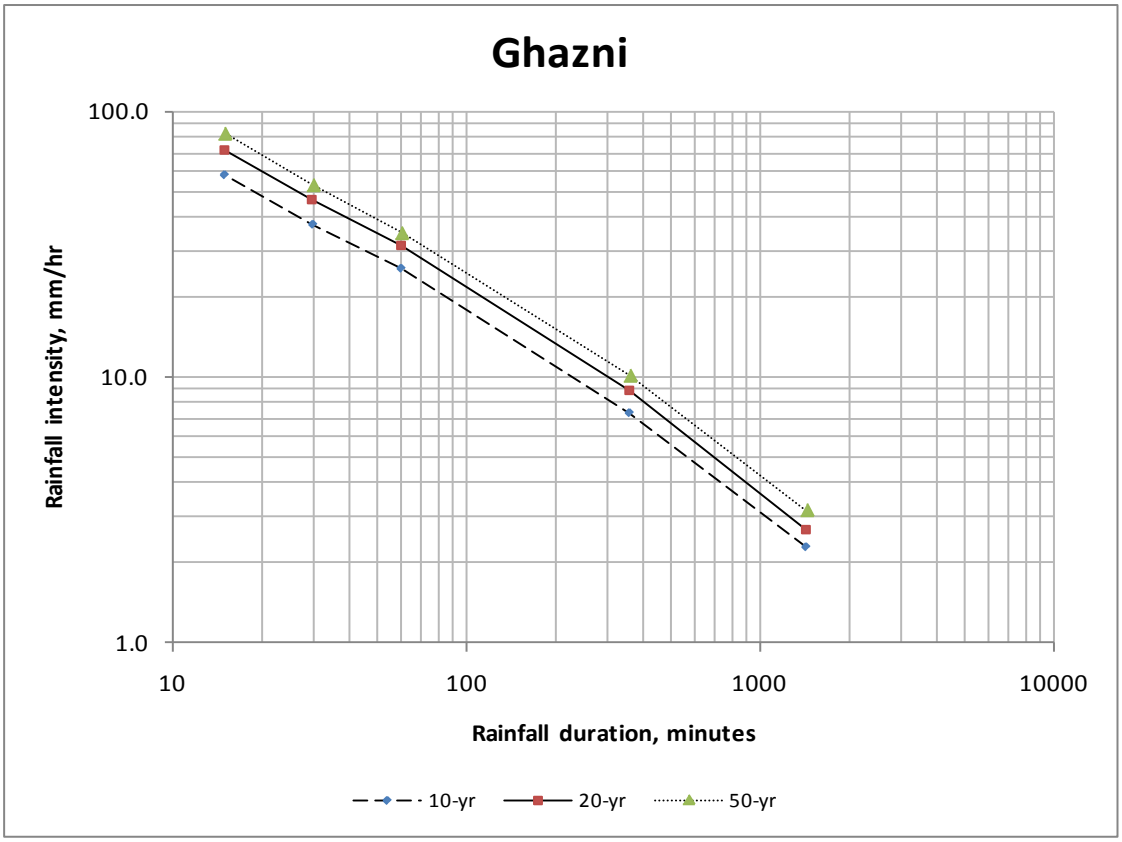
Appendix B Intensity-Duration-Frequency Curves for Selected Project Regions

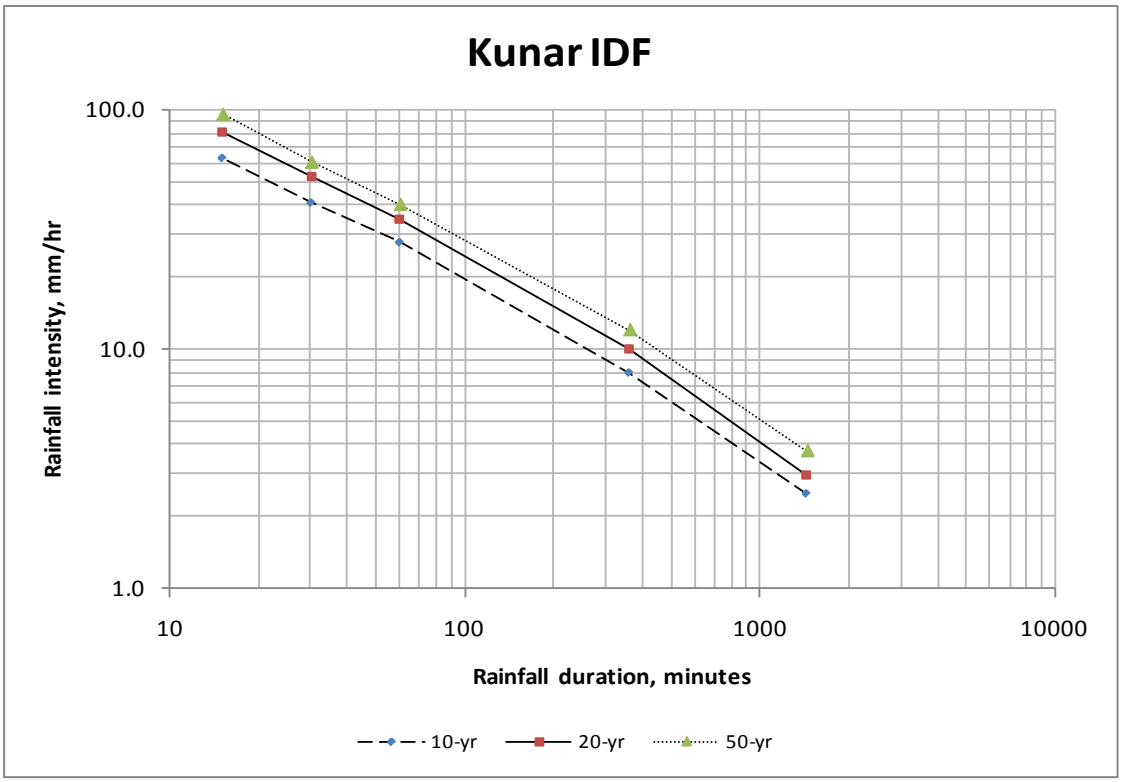


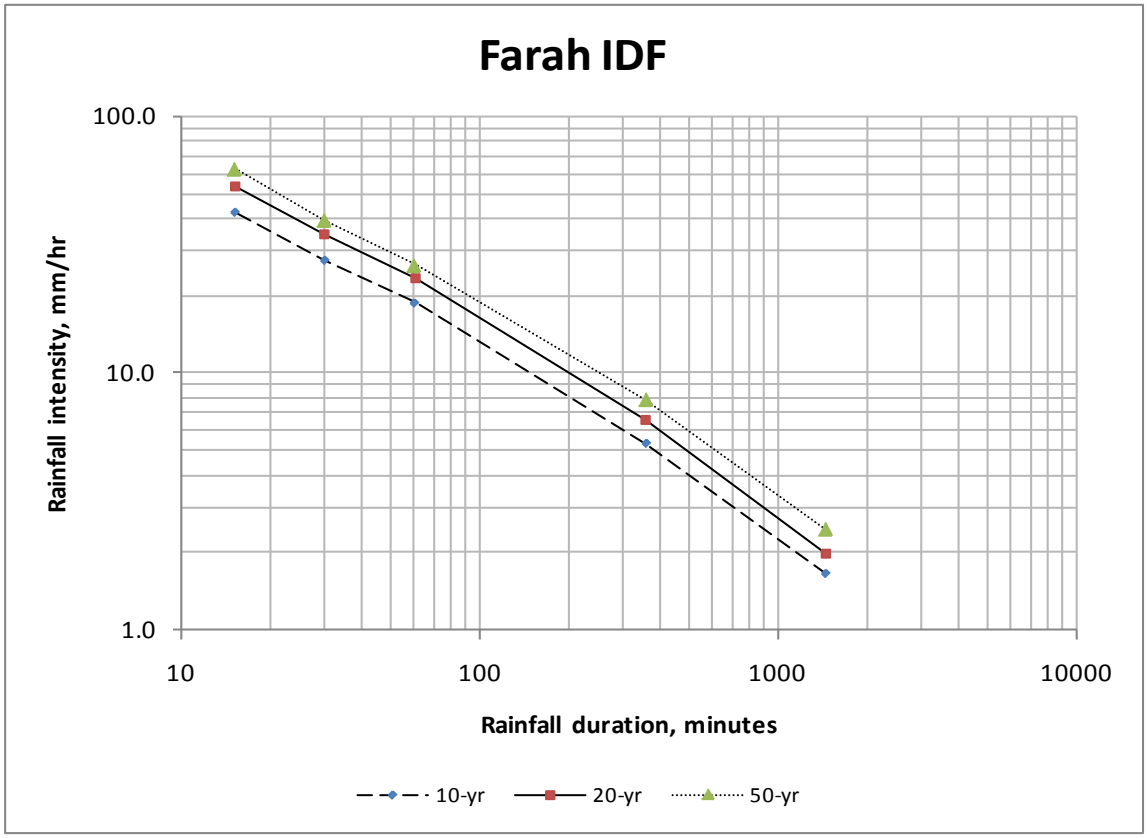


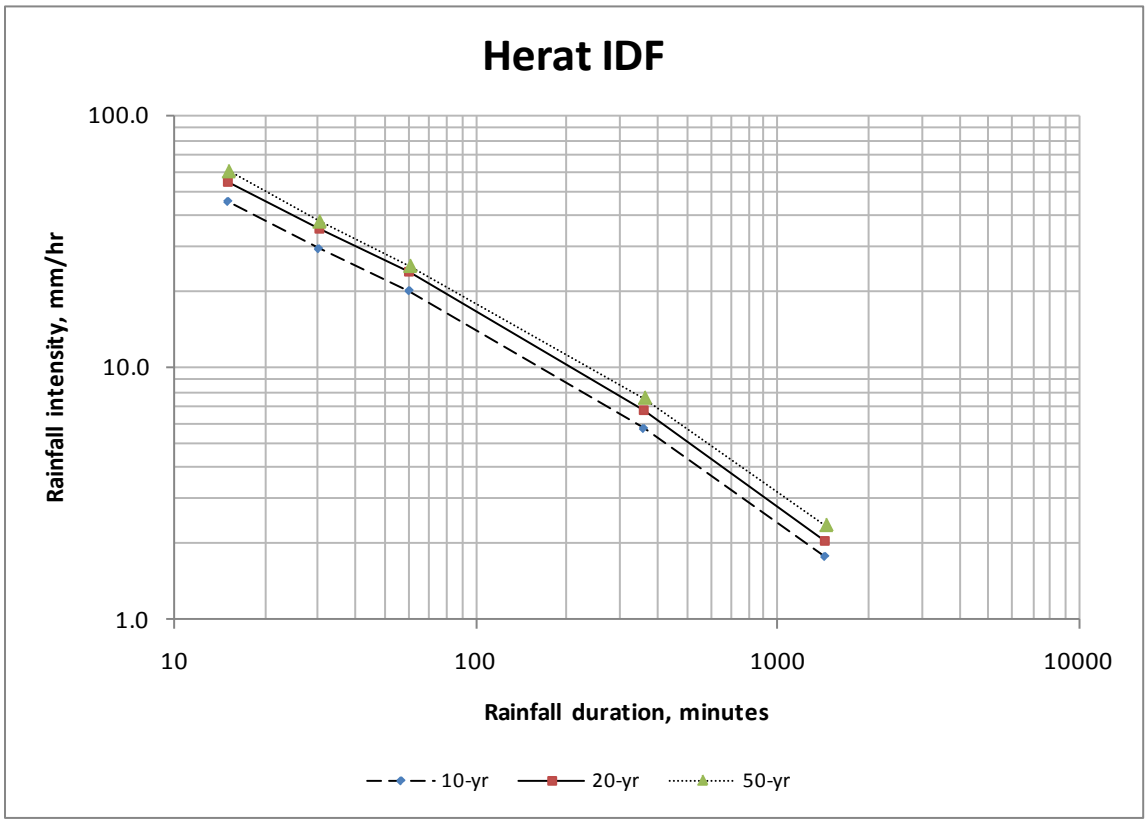

Jalalabad
100.0
10.0
1.0
Rainfall intensity, mm/hr
10
100
1000
10000
Rainfall duration, minutes
10-yr
20-yr
50-yr

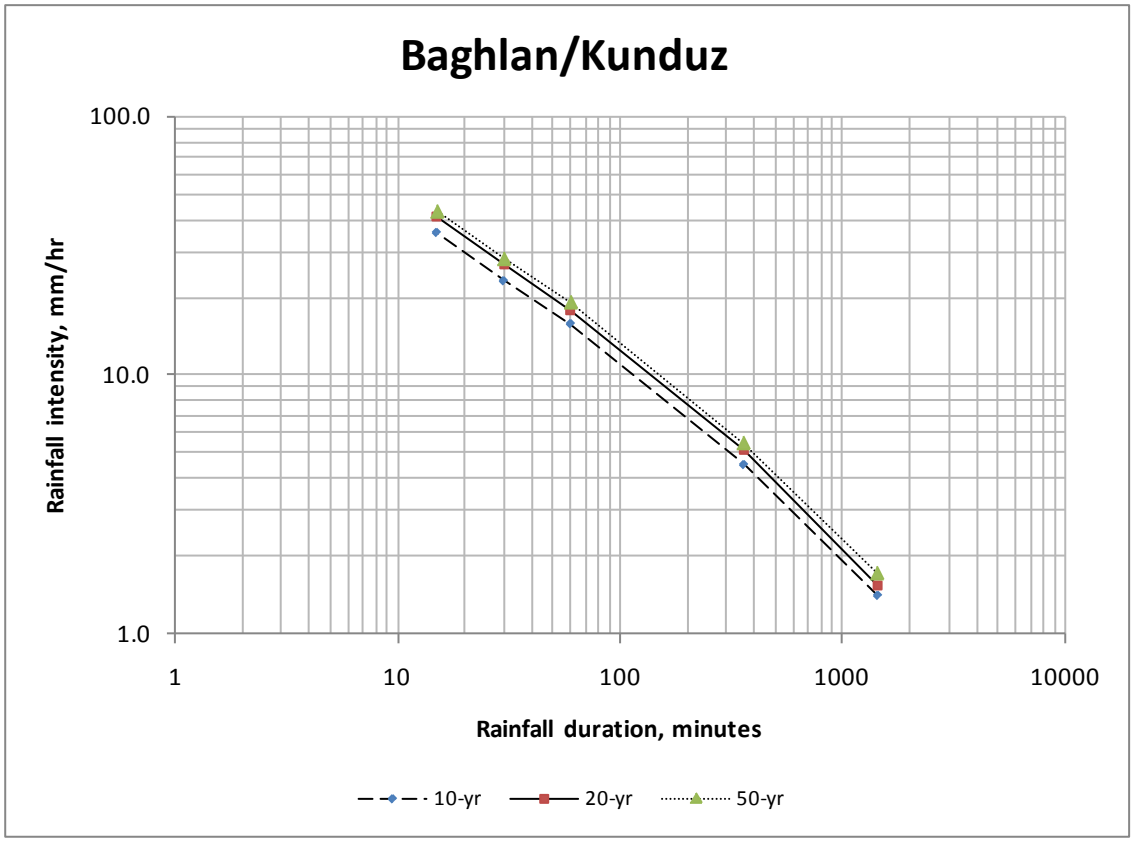
Gardez
100.0
10.0
1.0
Rainfall intensity, mm/hr
10
100
1000
10000
Rainfall duration, minutes
10-yr
20-yr
50-yr


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


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


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


Herat IDF
100.0
10.0
1.0
Rainfall intensity, mm/hr
10
100
1000
10000
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

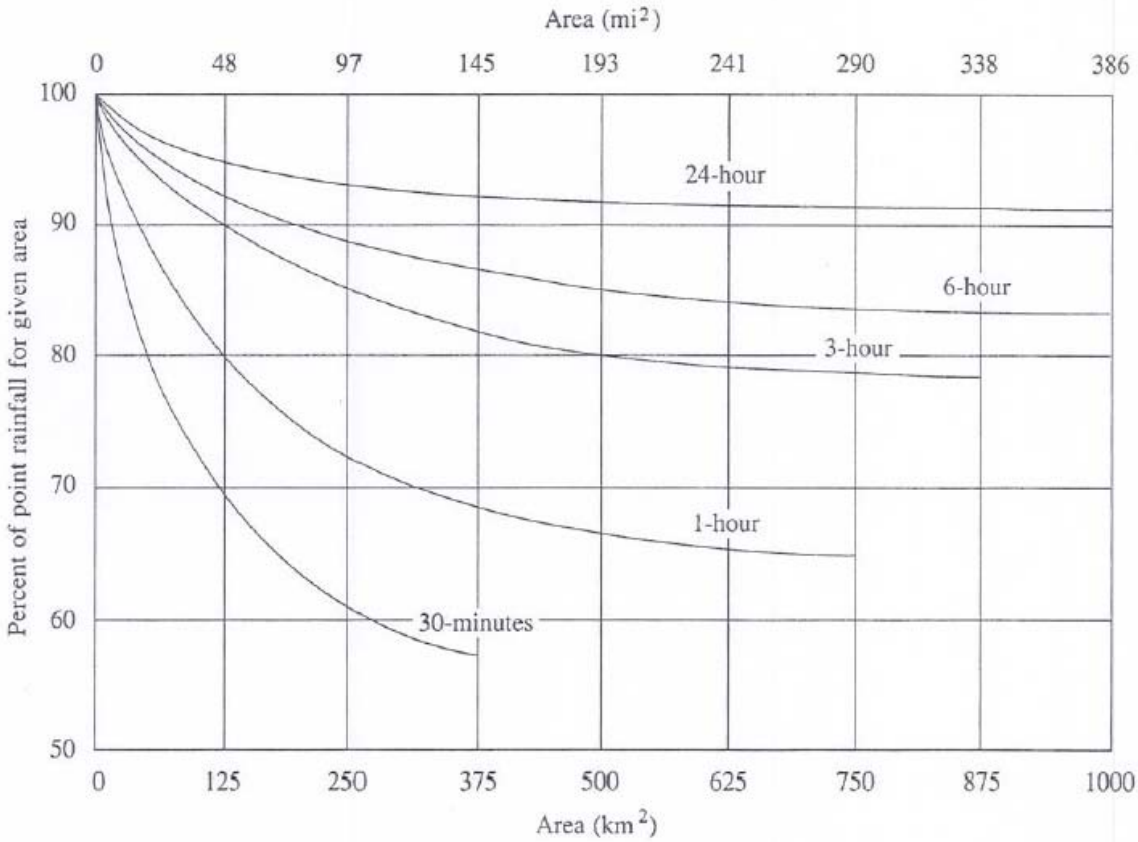


FIGURE 14.1.3
Depth-area curves for reducing point rainfall to obtain areal average values. (*Source*: World Meteorological Organization, 1983; originally published in Technical Paper 29, U. S. Weather Bureau, 1958.)

US Army Corps
of Engineers
Afghanistan Engineer District

AED Design Requirements:
Hydropneumatic Tanks

Various Locations,
Afghanistan

SEPTEMBER 2009

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AED DESIGN REQUIREMENTS
FOR
HYDROPNEUMATIC TANKS
VARIOUS LOCATIONS,
AFGHANISTAN

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1. General

The purpose of this document is to provide requirements to Contractors for any project requiring hydro-pneumatic tank design and construction.

2. Hydropneumatic Tanks

There are several different functions that a hydropneumatic tank can perform. In a booster pump application, it can provide water to the system during periods of a no flow shutdown of the booster pump or it can provide water to replace leak loads. In a well water application, it can provide the desired volume of water required between the pump shut down pressure and the pump turn on pressure. In a sprinkler or irrigation pump application, the tank may provide a cushion to maintain necessary pressure so the jockey pump will not short cycle.

3. Type of Tanks

a) Bladder Tanks. Bladder tanks are required in a closed loop heating or chilled water HVAC system to absorb the expanding fluid and limit the pressure within a heating or cooling system. A properly sized expansion or compression tank will accommodate the expansion of the system fluid during the heating or cooling cycle without allowing the system to exceed the critical pressure limits of the system. The expansion or compression tank uses compressed air to maintain system pressures by accepting and expelling the changing volume of water as it heats and cools. Some tank designs incorporate a diaphragm or bladder to isolate the expanded water from the pressure controlling air cushion. As water is expanded, it is contained in the bladder preventing tank corrosion and water logging potentials. The pressure controlling air cushion is pre-charged at the factory and can be adjusted in the field to meet critical system requirements. This design and operation of this style of expansion tank allows the designer/specifying engineer to reduce tank sizes up to 80%. Bladder tanks have a totally replaceable bladder. This tank must be equipped with large service openings and also allow the bladder to be replaced without having to remove the tank from its mounting.

b) FXA Bladder Tanks (ASME). The FXA Series tanks are manufactured to ASME specifications. The FXA's are industries broadest line of bladder style tanks. These vessels are designed using a heavy-duty replaceable butyl bladder, and are offered in sizes ranging from 10 to 4,000 gallons, with working pressures of 125 psi (standard), 200 and 250 psi.

c) Diaphragm Tanks. The diaphragm tank has been developed to allow the systems air cushion to be separated from the systems water. No water logging of the tank can occur as the air is held between the tank wall and the outside of a bladder placed inside the tank, while the system water is contained inside the bladder. This changes the system to an air elimination system, as any air extracted from the system water is passed out of the system into the atmosphere. The diaphragm cannot be replaced if the tank fails. The advantage of this style is the initial cost. On smaller tanks the cost of replacing the bladder may exceed the cost of replacing the tank.

4. Hydropneumatic Tank Sizing

A tank can be viewed as a bag of air. The goal in sizing a tank is to make the bag of air large enough so that when the desired amount of water is put in the tank the air is not compressed to a pressure greater than system design. The first step in sizing a hydropneumatic tank is to determine the amount of water that the tank will be required to supply during any given cycle, drawdown. Drawdown is determined by the following equation.

$$D = t * (Q * 60) / 4$$

Where:

D = Drawdown (liters)
T = cycle time (min.)
Q = tank flow rate (liters/sec.)

Once the drawdown has been determined the minimum and maximum pressure of the system should be determined. The minimum pressure is the pressure where the system pumps will turn on and the maximum pressure is the shut off pressure of the pumps.

The acceptance volume is the volume of water a tank is designed to hold based on the minimum and maximum pressures. In a bladder tank, a bladder smaller than the tank size is used, the acceptance volume would be what the bladder is designed to hold. Air is contained between the tank shell and the bladder,therefore it is not possible to fill the entire tank with water. Some bladder tanks have a stated "full acceptance volume". This means that if there is a loss of air in the tank, the bladder would be able to accept full tank volume without permanent damage. The acceptance volume is determined by multiplying the tank volume by an acceptance factor. The acceptance factor is determined by the following equation.

$$F = 1 - [(P_f + \text{atmospheric pressure}) / (P_o + \text{atmospheric pressure})]$$

Where:

F = acceptance factor
P_f = minimum operating pressure (gauge pressure)
P_o = maximum operating pressure (gauge pressure)
Atm = atmospheric pressure=101 kpa

Alternatively, the tank size may be determined by dividing the drawdown by the acceptance factor.

$$V=D / F$$

Where:

V = tank volume
D = drawdown
F = acceptance factor

5. As-Builts

Upon completion of installing the hydropneumatic tank, 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.

Example 1. Hydropneumatic Tank Sizing

Select a ASME replaceable bladder type hydro-pneumatic tank for municipal well water and booster application. The tank shall provide 12.5 liters per second to prevent booster pump short cycling. The allowable cycle of the booster pump is 10 minutes and the minimum and maximum operating pressures are 30 psi (207 kpa) and 60 psi (414 kpa) .

Known:

Flow (Q)=12.5 lps
Cycle (t)=10 min
P_f=207 kpa
P_o=414 kpa
Atm = 101 kpa

Find drawdown.

D=t(Q*60)/4

D=10 min((12.5 l/s)(60sec/min))/4 = 1,875 liters

Determine acceptance factor.

F=1-[(P_f+atmospheric pressure)/(P_o+atmospheric pressure)]

F=1-[(207+101)/(414+101)=0.402

Determine tank volume.

V=D/F

V=1,875 liters/0.402=4,664 liters

Use one 4,800 liter hydropneumatic tank or two 2,400 liter hydropneumatic tanks.

US Army Corps
of Engineers
Afghanistan Engineer District

AED Design Requirements:
Jockey Pumps

Various Locations,
Afghanistan

MARCH 2009

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AED DESIGN REQUIREMENTS
FOR
JOCKEY PUMPS
VARIOUS LOCATIONS,
AFGHANISTAN

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1. General

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

2. Jockey Pumps

Jockey pumps are variable speed pumps used in conjunction with the main water distribution system pumps. A typical pumping system may include two pumps, each capable of handling average daily flow and a third larger pump that will be used during peak flow. The system will operate, during normal times with one of the average daily flow pumps operating. As the demand increases, the second average daily flow pump will activate to increase system pressure and flow. Under extreme demand situations the two average daily flow pumps will be in operation and the third peak flow pump will activate to maintain the necessary pressure and flow within the system. Generally, during the overnight hours the system demand will be much lower than what is provided by one of the average daily flow pumps. During these times the jockey pumps will maintain pressure when the larger main pumps are not necessary due to a reduction in the flow demands on the system. Typically, a hydro-pneumatic tank will be provided with the jockey pump to assist in flow and pressure maintenance.

3. Pump Selection

When sizing a jockey pump set it is important to ensure that the performance of the pump can meet the highest possible demand both in terms of both flow rate and pressure and yet not be oversized. The following should be considered when sizing the pump:

The consumption pattern to be met by the pump set, i.e. how much does the consumption vary and how suddenly does the consumption vary.

The distribution of consumption over time.

Main pump operating characteristics.

The system size including pump performance and the number of pumps.

Hydro-pneumatic tank selection.

The dry-running protection to be selected.

The consumption pattern may be shown as a 24-hour profile or as a duty-time profile as shown below.

Figure 1. 24-Hour Profile

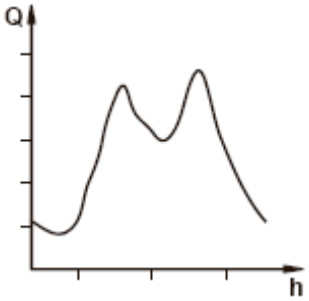
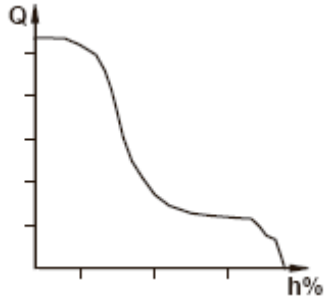


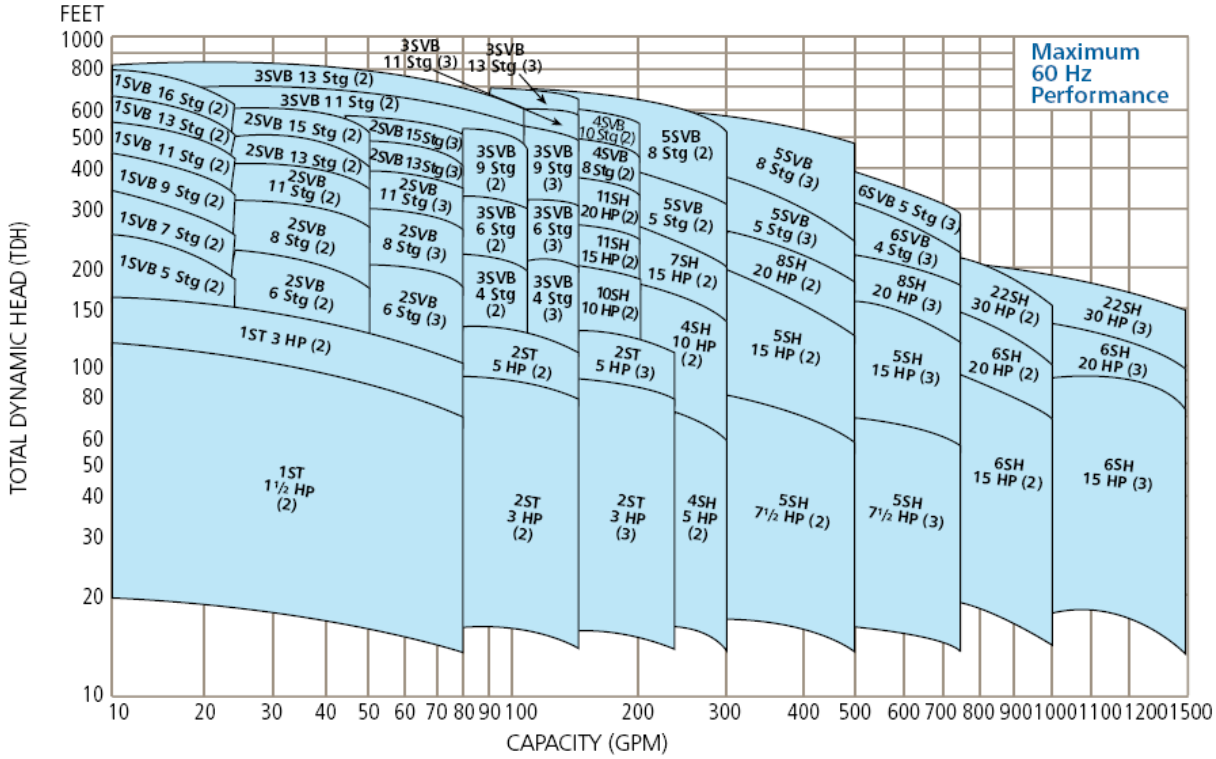
Figure 2. Duty-Time Profile



The 24-hour profile shows the instantaneous average consumption during any period of a 24-hour time period. The duty-time profile shows the average consumption based on the percent of the day. The designer should be able to determine this information from the records for an existing system or should be able to predict this information for a new installation based on anticipated uses and expected operational hours. The jockey pump system should be either a duplex (2 pump) or a triplex (3 pump) system. For both a duplex and a triplex system, each pump should produce 50% of the total flow.

The system head curve for the distribution system can be determined as outlined in the Booster Pump section of this design manual. The designer should include all head losses associated with the jockey pump system. Using the system head curve the designer should determine the total head that needs to be maintained and the maximum flow rate for the jockey pumps. This information is then plotted along the pump curve to select the proper pumps. The best results are obtained when the maximum pressure is within 10% of the best pump efficiency. An example of a variable speed pump curve is shown below.

Figure 3. Variable Speed Pump Curve



4. Tank Selection

For smooth operation and shut-down at zero flow, it is recommended that a pressure tank be sized and included with the jockey pump system. Tank capacity should be between 10 and 25 percent of the maximum flow rate of one pump. If a duplex pump system were designed to so that each pump would provide a flow rate of 50 m^3/hr. then the tank capacity should be between 5 and 12.5 m^3. After installation the tank should be pre-charged to a pressure that is approximately 0.7 to 0.85 kg/cm^2 below the system pressure. There should not be any water in the system when the tank is pre-charge to ensure accuracy. When selecting a tank, the designer should make sure that the tank can support or exceed the maximum system pressure and is compatible with the distribution system.

5. Tank Location

The tank should be located in the discharge piping between the pump check valves and the system shut-off valve. The tank connection to the system should be with a full pipe size connection and reducers or “quick disconnect” couplings to the tank which will impede the flow from the tank during system operations should not be used. There should be no valves or other devices between the tank and the transducers.

6. As-Builts

Upon completion of installing the jockey pump 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.

US Army Corps
of Engineers
Afghanistan Engineer District

AED Design Requirements:
Oil/Water Separators

Various Locations,
Afghanistan

January 2010

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FOR
OIL/WATER SEPARATORS
VARIOUS LOCATIONS,
AFGHANISTAN

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1. General

The purpose of this document is to provide requirements to Contractors for any project requiring the design and construction of oil/water separators.

2. Oil/Water Separators

Oil/water separators are devices commonly used to separate oily waste products from wastewater streams. They are typically installed in industrial and maintenance areas such as aircraft and vehicle maintenance and washing and other activities involving petroleum, oil and lubricants (POL) to separate oils at low concentrations from wastewater.

3. Types of Oil/Water Separators

There are three predominant types of oil/water separators: Gravity separators (conventional gravity separators and parallel plate gravity separators) and flotation separators. The overwhelming majority of oil/water separators used are conventional gravity separators. Gravity separators use a process that relies on the different densities of oil, water, and solids for successful operation. The wastewater is fed to a vessel sized to provide a quiescent zone of sufficient retention time to allow the oil to float to the top and the solids to settle to the bottom. Gravity oil/ water separators come in two configurations: conventional separators such as those designed in accordance with guidelines established by the American Petroleum Institute (API) and parallel plate separators. The primary types of oil/water separators described in this section are intended for the removal of free and emulsified oils and greases, usually as a pretreatment method before discharge to the sanitary sewer or septic system.

4. Conventional Separators (API)

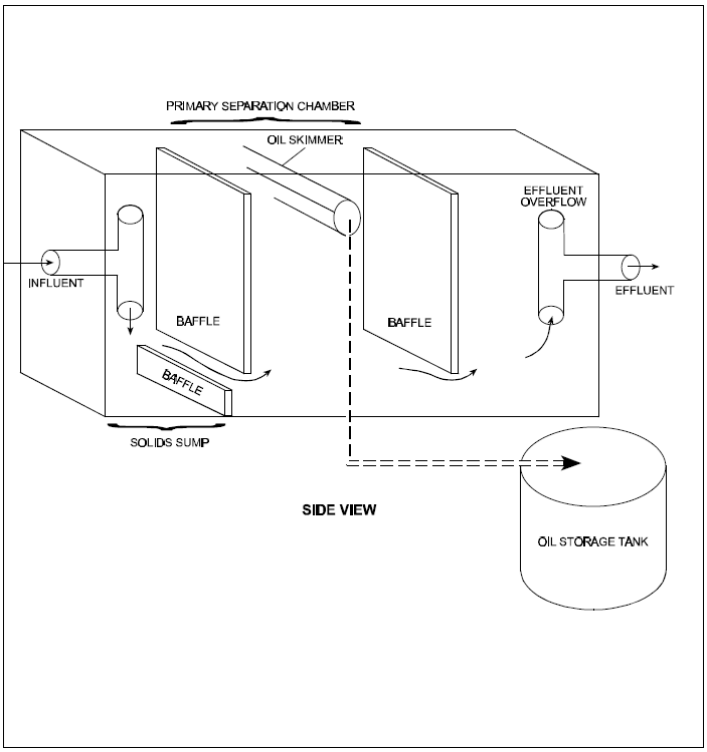
A typical conventional separator system is shown in Figure 1. The separator itself has three chambers separated by baffles: an influent chamber, the main separator chamber, and an effluent chamber.

The influent chamber removes free oil that has already separated from the oil/water mixture during conveyance to the unit. Two baffles separate the influent chamber from the larger, main settling chamber. The upper baffle is placed at the top of the water level and extends three quarters of the way to the bottom. It prevents the floating oil and scum from entering the main chamber, and allows it to be skimmed off through an overflow pipe. The lower baffle extends from the bottom and directs the wastewater to the top of the main chamber and to prevent short-circuiting.

In the main separator chamber, the oily wastewater flows from one end to the other under quiescent conditions. The wastewater velocity is kept very low, typically less than 0.9 m/min to prevent turbulent mixing. For flat-bottom chambers, removal of settled solids is typically accomplished by taking the chamber out of service; the chamber is drained and accumulated solids are removed either manually or by a vacuum truck. If the floor is sloped, the solids can be removed from the hopper or V-bottom trough by pumping or gravity discharge while the unit is still in service. An oil-skimming device should be provided at the end of the separation chamber. The rotatable, slotted-pipe skimmer is the most common type. Other oil-skimming devices include belt skimmers and floating skimmers. The oil collected by the skimmer is discharged to a used oil holding tank.

The effluent chamber is also separated from the main chamber by upper and lower baffles. Wastewater flows under and over the baffles into the effluent chamber. From the effluent chamber, the treated water can be discharged to the sewer system or to additional treatment if necessary.

Figure 1. Conventional Gravity Separator



5. Parallel Plate Separators

A typical parallel-plate separator system is shown in Figure 2. Parallel-plate separators function on the same principles as conventional gravity separators, but they require less space. Incorporating an array of closely spaced parallel plates within the separator chamber increases the surface settling area, allowing the overall size of the unit to be reduced. Flow through a parallel-plate unit can be two to three times that of an equivalently sized conventional separator. The oil is removed by passing the wastewater at laminar velocity through the pack of closely spaced plates, which are constructed at various inclines ranging from 45 to 60 degrees. The oil droplets rise vertically until they contact the underside of the plates, where the oil coalesces and continues to gradually move upward along the underside of the inclined plates. Eventually, the floating oil is collected at the surface of the tank. The plates aid in separation in the following ways:

Prevent shout-circuiting of the oily waste.

Increase the effective settling area.

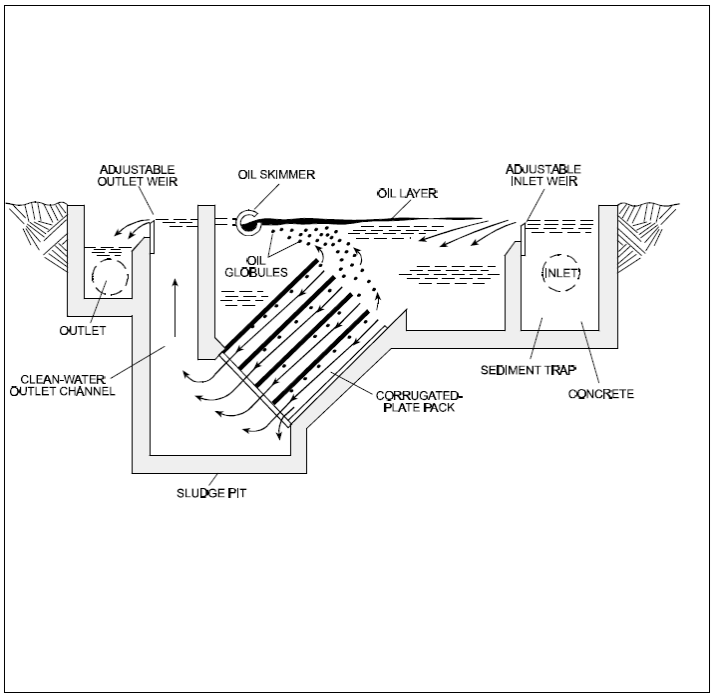
Enhance contact/agglomeration of oil particles.

The plates may be made of an oleophilic (oil attracting) material, such as polyethylene, fiberglass, or nylon to promote coalescence of oil droplets. For this reason, the units are sometimes referred to as coalescing plate separators. Parallel or coalescing plate separators are usually recommended only for light oil loadings when a higher level of oil removal is required, when the wastewater stream contains minimal solids concentrations, and when the facility is committed to the additional maintenance procedures required keeping the coalescing pack free of debris. The plates may also be constructed in a corrugated configuration with alternate troughs and ridges, such as in the Corrugated Plate Interceptor (CPI). It should be noted that polypropylene is not recommended for

use in coalescing plates because it has been found to have excessively strong oleophilic properties that prevent oil droplets from detaching and floating to the surface.

In parallel-plate separators, solids settle to the bottom and are collected in a sludge well. From the well, sludge is pumped or withdrawn by gravity. If sludge transfer is by gravity displacement, an automatic valve is usually provided.

Figure 2. Downflow Parallel-Plate Separator



6. Oil Collection

A tank should be provided for collection of used oil collected by the skimmer. The tank should be designed as an integral part of the oil/water separator system and should also be designed so that confined space entry is not required for operation and maintenance.

7. Design Considerations for Oil/Water Separator Systems

Design of conventional and parallel-plate oil/water separator systems requires proper characterization of the wastewater, establishment of the design flow and required effluent quality, sizing of the separator, and proper flow attenuation/flow equalization of the influent. Sludge disposal and used oil disposal also needs to be properly considered. The designer should work to identify the user's needs and the capability of any vendor-supplied equipment. These aspects of system design are discussed below.

a) Wastewater Characterization. If possible, the wastewater to be treated should be analyzed for total oil and grease, including determination of the free, emulsified, and dissolved oil fractions. As indicated previously, although oil/water separators are designed to remove free oil, they also remove solids. Therefore, the solids content of the wastewater is important in overall system design. Analyses should include TSS, volatile suspended solids (VSS), and settleable solids. These analyses will help determine the amount and frequency of settled materials that will need to be removed from the bottom of the oil/water separator and the advisability of providing grit removal upstream of the oil/water separator.

Other wastewater characteristics important in the design of oil/water separators are the specific gravities of the oil and water phases and the absolute viscosity of the wastewater, both at the minimum design temperature. Wastewater temperature and pH have a major impact on the efficiency of the separator, with poorer separation occurring at lower temperatures and high pH values. Separators are not designed to remove pollutants such as phenols, solvents, and heavy metals. These pollutants should be addressed by the use of pollution-prevention techniques. If the wastewater characteristics cannot be determined through testing, the facility should refer to the general wastewater characteristics for similar facilities as provided in Section 4 of MIL-HDBK-1005/17.

b) Site Considerations. The oil/water separator should be designed to be readily accessible for maintenance and inspection, including access for a vacuum truck or other oil- and grit-removal equipment. Visual inspection and the ability to probe for solids levels are critical to good operation. Aboveground units are the easiest to access for maintenance and inspection; however, they may require pumping of the wastewater, which may cause emulsification of oils.

Below grade systems have the advantage of providing greater temperature consistency, especially in cold temperature climates. However, below grade separators with only manhole access are difficult to maintain and should be avoided. When below grade installations are necessary, they should preferably be located within an accessible vault for ease of operation and maintenance. If buried, units should have ample access points or open tops for inspection and cleaning, and should be installed with a double liner and leak-detection system.

c) Establishing the Design Flow. The efficiency of separation also decreases when flow exceeds the design capacity of the separator. Therefore, the design flow should be based on the maximum flow rate to be treated, including the addition of any future oily wastewaters and stormwater runoff. In determining peak flow rates, variations between shifts and daily and seasonal variations should be considered. Flow rates should be measured where the wastewater generating process already exists, or accurately estimated where it does not.

In some cases, establishing production-based wastewater generation rates may be useful for projecting future flows. For example, the maximum flow expected from an aircraft washing facility may be estimated from the expected washwater per aircraft multiplied by the maximum number of aircraft to be washed in a given period. If unit wastewater generation rates from another facility are used, differing conditions should be accounted for, such as differences in the type and size of aircraft and washing procedures.

Where high flows of short duration are to be handled, alternatives to constructing a larger separator include:

Providing flow equalization at an upstream grit basin.

Providing flow equalization within the oil/water separator itself by operating the unit in batch treatment mode (emptying after each batch).

Providing curbing and covering of outdoor washracks to exclude stormwater.

Some installations have installed a manually controlled valve on washrack drains to allow stormwater to be diverted to a storm drain during rainfall periods, but such valves are discouraged because they are prone to be left open and allow washwater to be discharged.

d) Pretreatment for Grit Removal. A separate basin to remove grit should be provided upstream of the oil/water separator for wastewaters with a high suspended solids concentration. The design should be based on gravity separation of grit with a minimum detention time of 5 minutes at the maximum rate of flow. Larger basins may be used if the grit removal basin is also designed as a flow equalization basin. In either case, access should be provided for a truck with suction equipment to periodically remove grit. In large systems, the use of a ramp at one end of the basin may be considered if the material to be removed is primarily sand and readily dewaterable. Mechanical grit removal equipment should not be provided because it requires extensive maintenance.

8. Design of Oil Water Separator Systems

a. Parallel-Plate Separators

Parallel-plate oil/water separators are furnished as pre-engineered, factory-assembled units. As such, designs vary by manufacturer, and vendor experience must be used in unit sizing and selection. In general, however, the parameters and procedures used for the design of parallel-plate separators are the same as for conventional separators.

The perpendicular distance between plates should range from 2 to 4 cm, and the angle of plate inclination from the horizontal should typically range between 45 and 60 degrees. Wider plate spacing and steeper plate inclination generally reduces maintenance and improves performance but adds to cost. Manufacturers' claims of settling area should be checked because some manufacturers incorrectly count both sides of the plates when calculating the effective surface area for their products. Settling area and detention time are key variables that may be used to compare competing equipment.

Accessible placement of the oil/water separator is particularly important for parallel-plate separators, which may require frequent cleaning. Removal for cleaning with high-pressure cleaning equipment is the procedure of choice. If cleaning in place is used, a hose connection and proper provisions to minimize worker health and safety risks should be provided. If high solids concentrations are present, installation of a grit-removal basin upstream of the oil/water separator is highly recommended.
Design Criteria for Conventional Separators

b. Conventional Separators

Key parameters that should be considered in the design of conventional oil/water separators include flow rate, minimum wastewater temperature, oil-fraction specific gravity, oil concentration, relative fractions of free and emulsified oils, quantity of solids to be removed, effluent oil concentration limits to be met, and detention time. Design criteria should conform to the following:

a) Separator Depth: greater than 1.0 meter.
b) Horizontal Velocity: Less than 0.91 meters/minute or, if known, less than 15 times the rise rate of oil droplets, whichever is less.
c) Width-to-Depth ratio: Between 2:1 and 3:1
d) Length-to-Width ratio: between 3:1 and 5:1.
e) Detention Time Required at Average Flow: 30 minutes
f) Detention Time Required at Anticipated Peak Flow: 15 minutes.

Regardless of the calculated separator dimensions, the **minimum size separator** which may be installed in an AED project shall be 1 meter deep x 2 meters wide x 3 meters long. Larger separators shall be installed when the calculations indicate larger separators are needed.

Where continuous service is required, a backup separator should be provided.

The recommended design for oil water separators within AED is a triple basin design. The separator shall have three collection chambers and shall be constructed of concrete and located so that water flows in and out by gravity. The structure may be placed at any depth, but shall have 3 access ports which are accessible at grade, to allow removal of solids and separated oils from each chamber. See diagram below.

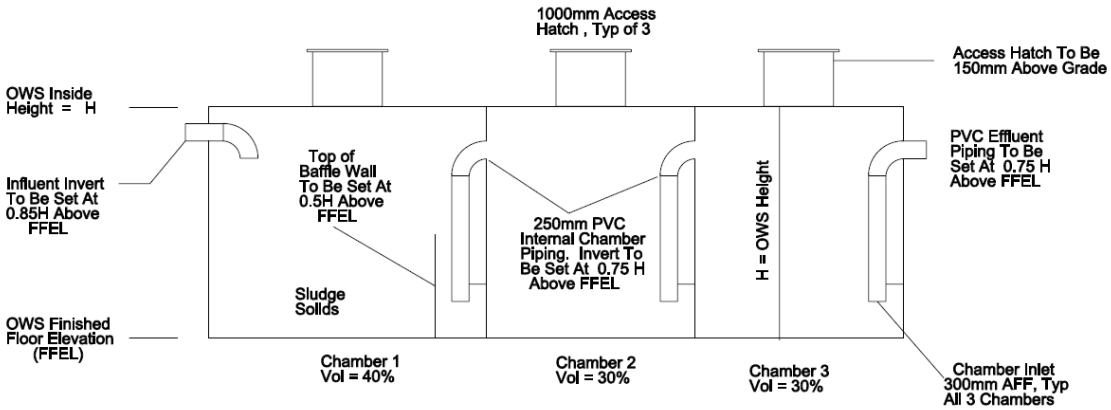


Figure 3. Oil Water Separator Layout

The first chamber traps floatables, oils, sand and other sediments. The second chamber traps oils which were entrained in with the water, and the third chamber is a polishing chamber. A majority of the solids will settle in the first chamber which should be periodically cleaned. The oils will also preferentially remain in the first chamber, but carryover will occur into the remaining chambers during high flow surges.

10. Design Example

Design an oil water separator for a vehicle maintenance facility with 3 hose bibs which are used for wash down. Each hose bib is rated at 0.5 L/s. Normally 1 maintenance bay will be rinsed at a time, but all three may be at the same time during a spill cleanup.

Average Flow = 1 * 0.5 L/s;
Required Volume for Average Flow = 30 min * 0.5 L/s * 60 sec/min = 900 L = 0.9 m3

Peak Design Flow = 3 * 0.5 L/s = 1.5 L/s
Required Volume for Peak Flow = 15 min * 1.5 L/s * 60 sec/min = 1350L = 1.35 m3

The required separator must have a minimum volume of 1.35 m3.

Calculated Separator Dimensions:

Horizontal Velocity = 0.91 m/min, therefore

Depth x Width = $\frac{0.09 \text{ m3/min}}{0.91 \text{ m/min}}$ = 0.1 m2

If assume Width:Depth = 2:1, then W = 2D and therefore 0.1 m2 = D * 2D.

Depth = 0.22m and Width = 0.44 m

If assume Length:Width = 3:1, then L = 3W, and Length = 1.34 m

Calculated Separator Dimensions: D = 0.22 m, W = 0.44 m and L = 1.34 m

Calculated Separator Volume: 0.13 m3

A separator with dimensions of D = 0.25 m, W = 0.5 m and L = 1.4 m will meet the separator dimensional design requirements, but this separator does not meet the minimum AED separator requirements. Therefore, the separator required for this application would be D = 1 m, W = 2 m, L = 3 m which would have a volume of 6 m3, and a detention time of 200 minutes at the average flow when using 1 hose bib, and 67 minutes at the peak flow when using 3 hose bibs.

11. Central Vehicle Wash Facilities (CVWF)

Facilities for the collection, treatment and reuse of runoff and recycled wash waters from vehicle wash facilities shall be designed in accordance with UFC 4-214-03. Separators designed using this document typically provide sedimentation, filtration, chlorination and oil water separation.

12. Sludge Removal and Disposal

Reliable sludge removal from the separation chamber is a frequent problem with both commercially available units and custom-designed separators. Currently, the most satisfactory method involves suction removal by installation personnel using equipment normally used for cleaning catch basins. This equipment is commonly referred to as a “vacuum" or "vac-all” truck.

13. Guidance Documents

The following documents provide additional guidance in operating and maintaining oil/water separators.

MIL-HDBK-1005/17	Nondomestic Wastewater Control and Pretreatment Design Criteria.
MIL-HDBK-1138	Wastewater Treatment System Operation and Maintenance Augmenting Handbook.
UFC 4-214-03	Central Vehicle Wash Facilities, 16 January 2004.

14. As-Builts

Upon completion of installing the oil/water separator, 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:
Package Waste Water Treatment Plants and Lagoons (Provisional)

Various Locations,
Afghanistan

September 2009 Version 1.2

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FOR
PACKAGE PLANT
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1. General

The purpose of this document is to provide wastewater treatment plant (WWTP) design requirements to Contractors for projects at medium-sized installations. It is a summary of requirements that should be provided in design submittals. References contained herein provide more specific design procedures and examples. Medium-sized installations are considered those requiring the design and construction of package plant for wastewater treatment for average daily waste water flow rates of between 60 cubic meter/day (15,000 gpd) and 378 cubic meter/day (100,000 gpd). Projects at small-sized installation with projected wastewater flows smaller than this range should be evaluated for onsite treatment systems such as septic/leach field disposal or self-contained lagoon systems. Septic and leach field systems are discussed in the USACE-AED Design Requirements Sanitary Sewer and Septic Tank System, 2009.

The contractor's design and construction shall comply with technical requirements contained herein. WWTP design involves several engineering disciplines including civil, mechanical, electrical and structural engineering. The designer shall have a minimum of 5 years experience with the design and construction of pre-engineered package wastewater treatment plants of the same magnitude and complexity as required in this project. The designer shall be responsible for providing (as demonstrated in shop submittals, Section 8) project experience in all the specialized disciplines and possess knowledge of the construction of package wastewater treatment plants as well as construction and industry standards for wastewater treatment plants.

2. Package Waste Water Plant

Treatment systems handling less than 378 cubic meters per day (100,000 gpd) are generally considered small package WWTP systems. For some packaged treatment systems, the principles of design are no different than larger plants with customized design but the choice of equipment will usually differ from that used in large plants. In addition, the level of redundancy for process components, and the types of containment systems may be dramatically different because package systems are pre-engineered and components may be constructed offsite. Packaged WWTPs must use larger safety factors for flow variation, altitude and temperature effects relative to total wastewater flows, and the lack of daily maintenance. Package WWTPs inherently have less operational flexibility; however, they are nonetheless capable of performing effectively and economically. Complete WWTPs can be obtained from various manufacturers. Some prefabricated plants may be capable of being relocated, depending on size and original construction. Most of these units are factory fabricated and shipped as complete units, ready for connections to piping and power. Small physical-chemical units have been developed as “add on" units to existing facilities to provide additional treatment efficiency. Shop submittals of package WWTP systems are a contract requirement.

Package plants are appropriate for forward operating bases (FOB) or small permanent installations such as ANP or ANA garrisons. These systems are designed to generally treat between 57,000 L/d (15,000 gpd) and 378,500 L/d (100,000 gpd), however larger capacities can be considered on a case by case basis. For MILCON projects beyond this flow range, the treatment processes used in the package WWTP may require HQDA (CEEC-EB) approval for Army projects and HQ USAF/LEEE approval for Air Force projects, with complete documentation and information to support the requested waiver. The wastewater treatment plant should be designed to achieve effluent quality standards as set forth in reference 1, unless superseded by subsequent host nation environmental guidance. In addition, the plant must be capable of being operated by few (no more than three) operating personnel, capable of being by-passed during operational failures, need a minimum of energy to provide treatment such that the plant can operate using standby power generation for a period of 48-hours. If the package plant design is supplementing the capacity of existing plants, criteria contained herein regarding flows and wastewater characteristics may be modified to conform to existing plant performance data provided the plant has been in operation long enough to have

established that it is meeting effluent quality standards.

3. Site Selection

The major factors in the selection of suitable sites for treatment facilities include the following:

- Topography
- Availability of a suitable effluent discharge point
- Access to the plant for construction, operation, and maintenance
- Availability and acceptable location of sludge drying and disposal facilities
- Minimum separation distance between the wastewater plant and living quarters, dining facilities, and water wells of 30 meters
- Site locations shall be selected such that the WWTP is fully operational and functional during a 25-year flood.

The siting criteria for the package plant should consider protection requirements for drinking water sources. Sufficient space must be allocated not only for suitable arrangement of the initial units and associated plant piping but also to accommodate future expansion. The site will be selected so that an all-weather road is available or can be provided for access to the plant.

4. Treatment Requirements

Package WWTP for USACE-AED projects will generally be designed to provide secondary treatment consistent with the effluent quality requirements in reference 1. However other levels of treatment may be required depending upon the characteristics of the waste water. *Preliminary* treatment is defined as any physical or chemical process at the wastewater treatment plant that precedes primary treatment. Its function is mainly to protect subsequent treatment units and to minimize operational problems. *Primary* treatment is defined as physical or, at times, chemical treatment for the removal of settleable and floatable materials. *Secondary* wastewater treatment is defined as processes which use biological and, at times, chemical treatment to accomplish substantial removal of dissolved organics and colloidal materials that degrade receiving water and impair its use for other beneficial purposes. Disinfection of waste water to inactivate harmful microorganisms is considered a process that can be used in conjunction with both primary and secondary treatment processes. *Advanced* wastewater treatment is defined as that required to achieve pollutant reductions by methods other than those used in secondary and primary treatment (sedimentation, activated sludge, trickling filter, etc.). Before treatment plant design is begun, treatment requirements will be determined on the basis of meeting stream and effluent requirements.

Pretreatment of pollutants which may interfere with operation of a sewage treatment plant or pass through such a plant untreated may be required. Additionally, in many cases, pretreatment of industrial or maintenance facilities wastewater such as laundry waste water and water desalination treatment plant brine and back wash will be necessary to prevent adverse effects on the sewage treatment plant processes. Some types of industrial waste may be admitted to wastewater treatment plants, e.g., cooling tower discharges, boiler blowdown, vehicle washrack wastewater, swimming pool filter discharges, and aircraft wash wastes using biodegradable detergents. Flow of industrial wastewater may be reduced through process modification or wastewater recirculation. Adverse impacts on the treatment plant can be mitigated by reducing the concentration of those compounds such as bleach or disinfectants causing the problem. Table 1 is a listing of compounds which inhibit

biological treatment processes. In some cases, the adverse impact may be caused by short-lived occurrences of either wastewater containing high concentrations of compounds or a wastewater flow rate much higher than the average daily flow. This situation, which is commonly called “slugs,” may, in some cases, be managed by including an equalization basin upstream of the treatment plant.

Advanced treatment employs a number of different unit operations, including ponds, post-aeration, microstraining, filtration, carbon adsorption, membrane solids separation, and specific treatment processes such as phosphorus and nitrogen removal. Advanced wastewater treatment is capable of very high effectiveness and is used when necessary to meet strict effluent standards. Organics and suspended solids removal of over 90 percent is obtainable using various combinations of conventional and advanced wastewater treatment processes. Phosphorus levels of less than 1 milligram per liter and total nitrogen levels of 5.0 milligrams per liter or less can also be reached through advanced treatment.

Table 1. Materials Which Inhibit Biological Treatment Processes

Pollutant	Inhibiting or Toxic Concentration[1], mg/L		
	Aerobic Processes	Anaerobic Digestion	Nitrification
Copper	1.0	1.0	0.5
Zinc	5.0	5.0	0.5
Chromium (Hexavalent)	2.0	5.0	2.0
Chromium (Trivalent)	2.0	2,000[2]	*
Total Chromium	5.0	5.0	*
Nickel	1.0	2.0	0.5
Lead	0.1	*	0.5
Boron	1.0	*	*
Cadmium	*	0.02[2]	*
Silver	0.03	*	*
Vanadium	10	*	*
Sulfides (S^-)	*	100[2]	*
Sulfates (SO_4^-)	*	500	*
Ammonia	*	1,500[2]	*
Sodium (Ma^+)	*	3,500	*
Potassium (K^+)	*	2,500	*
Calcium (Ca^{++})	*	2,500	*
Magnesium (Mg^{++})	*	1,000	*
Acrylonitrite	*	5.0[2]	*
Benzene	*	50	*
Carbon Tetrachloride	*	10[2]	*
Chloroform	18.0	0.1[2]	*
Methylene Chloride	*	1.0	*
Pentachlorophenol	*	0.4	*
1, 1, 1 Trichloroethan	*	1.0[2]	*
Trichlorofluormethane	*	0.7	*
Trichlorofluorethane	*	5.0[2]	*
Cyanide (HCN)	*	1.0	2.0
Total Oil (Petroleum origin)[3]	50	50	50

* Insufficient data available to determine effect.

[1] Raw wastewater concentration unless otherwise indicated.

[2] Digester influent concentration only; lower values may be required for protection of other treatment processes as noted above under aerobic and nitrification processes.

[3] Petroleum–based oil concentration measured by API Method 733–58 for determining volatile and non–volatile oily materials. (The inhibitory level does not apply to animal or vegetable oil.)

5. Basic Design Considerations

The required treatment processes are determined by the influent characteristics, the effluent requirements, and the treatment processes that produce an acceptable effluent. Influent characteristics are best determined by laboratory testing of samples from an existing waste stream or from a similar waste stream; but they may be predicted on the basis of standard waste streams as stated in the contract technical requirements. Treatment capacity is based on the design population, effective multiplied by a Capacity Factor) which is a projected population obtained by analysis, per capita loading rates and other industrial or operational waste water loads. The effective population and per capita loading rates are given for each project in the contract technical requirements. The design population is determined by adding the total resident and 1/3 the non-resident populations and multiplying by the appropriate capacity factor taken from table 2 which allows for variations in the using population.

Table 2. Capacity Factors

Effective Population	Capacity Factor
under 5,000	1.50
5,000	1.50
10,000	1.25
20,000	1.15
30,000	1.10
40,000	1.05
50,000	1.00

The rates of sewage flow vary widely throughout the day. The design of process elements in a sewage treatment plant is based on the average daily flow. Transmission elements, such as conduits, siphons and distributor mechanisms, will be designed on the basis of an expected peak flow rate of two times the average rate or the peak hourly rate for a specific process or conveyance component given in Reference 3. For example secondary clarifiers will be designed for a peak hourly flow rate (i.e., 1.75 times the average daily rate). Consideration of the minimum rate of flow is necessary in the design of certain elements, such as grit chambers, measuring devices and dosing equipment; for this purpose, 40 percent of the average flow rate will be used. The average daily wastewater flow to be used in the design of new treatment plants will be computed by multiplying the design population by the per capita rates of flow. Unless already provided in the contract technical requirements, per capita wastewater flows can be determined from Table 3, and then adjusting for such factors as industrial wastewater flow and infiltration. Where shift personnel are engaged, the flow will be computed for the shift when most of the people are working. A check on sewage volumes could be to compare water consumption to the sewage estimate (neglecting infiltration, which will be considered subsequently). About 60 to 80 percent of the consumed water will reappear as sewage, the other 20-40 percent being lost to irrigation, wash-down, and points of use not connected to the sewer. If existing flow data are not available the wastewater flow fraction of water use shall be considered 80 percent.

Table 3. Per Capita Sewage Flows (1)

Installation	Per capita, liters/day (gal/day) (2)
U.S. bases	152 (40)
ANA/ANP and other host nation	125 (33)

Notes:
(1) Wastewater per capita rates in contract technical requirements shall take precedence over these values
(2) AED per capita sewage flows are defined as 80% of the per capita water allowance. For example the ANP and ANA per capita water allowance is 155 lpcd (41 gpcd). 80% of this number is reflected in Table 3 above as the per capita sewage flow.

If an existing wastewater stream exists at the project site, the wastewater at existing facilities will be analyzed to determine the characteristics and constituents as required. Analytical methods will be as given in Reference 2. For treatment facilities at new installations which will not generate any unusual waste, the characteristics shall shown in Table 4 unless specifically stated otherwise in the contract technical requirements or as determine by influent sampling and testing using Standard Methods (Reference 2).

Table 4. Assumed Wastewater Influent Characteristics Flows

Parameter	Concentration, mg/l unless noted
pH	7 to 7.5 (standard units)
temperature	20 to 22 degrees celsius
BOD_5	400[1]
Total suspended solids	400
Total nitrogen	80
Ammonia nitrogen	15
Oils and grease	100
Phosphorous	15
Chloride	100
Hardness	200

Notes: BOD_5 shall be the greater number based on 400 mg/l or based on 0.09 kg/capita-day (0.2 lb/capita-day)

Concentrations are presented above in milligrams per liter; which is equivalent to parts per million (ppm). These values represent an average waste and therefore should be used only where detailed analysis is not available. When the water supply analysis for the installation is known, the above analysis will be modified to reflect the normal changes to constituents in water as it arrives at the wastewater treatment plant. Industrial waste loadings can also be characterized to a large extent by normal sewage parameters. However; industrial waste contains contaminants not generally found in domestic sewage and is more variable than domestic sewage in terms of pH, biochemical oxygen demand, chemical oxygen demand, oil and grease, and suspended solids. Each industrial wastewater must be characterized individually to determine any and all effects of treatment processes.

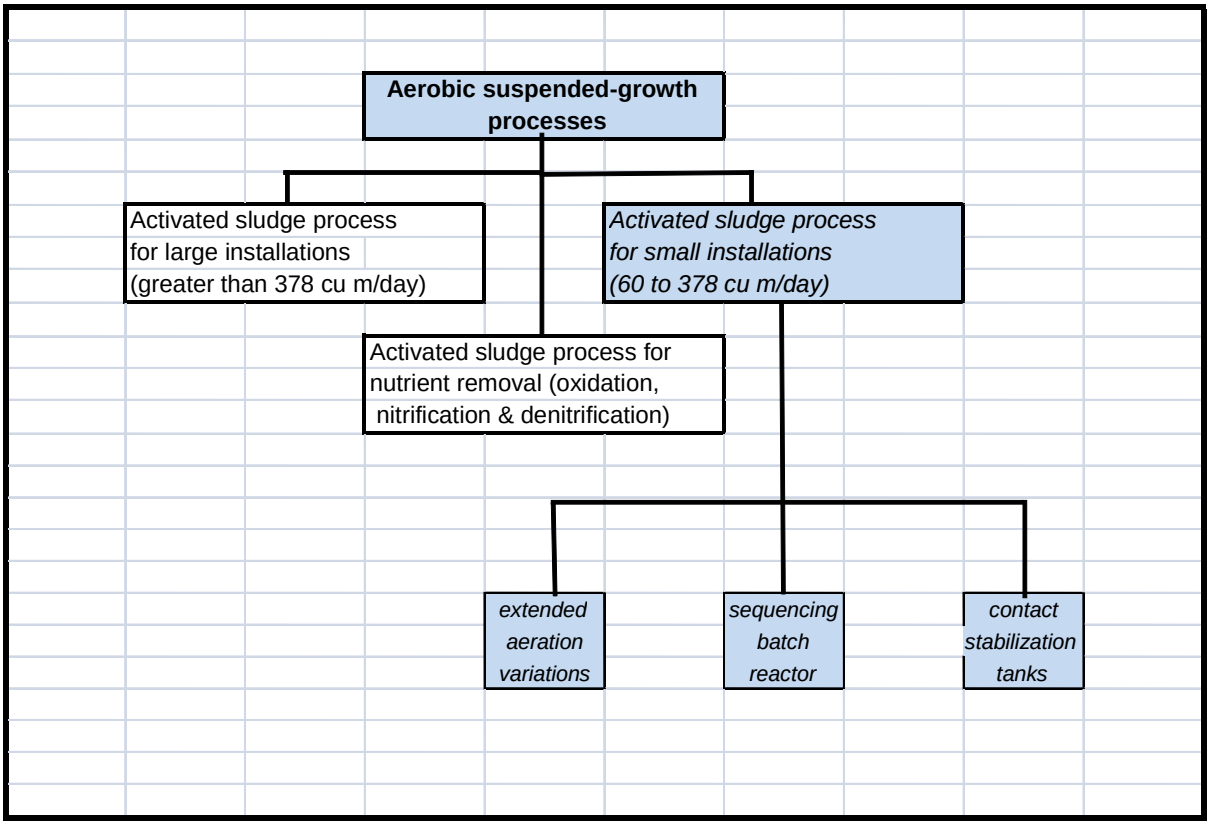
6. Selection of Treatment Process

Packaged WWTPs combine the different levels of treatment described in the previous section into a single treatment train. The secondary treatment process in the treatment train may consist of trickling filter plants, rotating biological discs, membrane bioreactors, physical-chemical technology, or an aerobic suspended growth (activated sludge) biological treatment process. Physical-chemical systems also have the flexibility to operate in an on-off mode which is not possible with biological systems. However, they are often costly to operate, require skilled attention, and produce large

amounts of sludge and are not the preferred treatment type of USACE-AED projects which often require training of Afghan nationals for the operation and maintenance in remote locations. Membrane bioreactors (MBR) which are now common in the U.S. at small package WWTP installations, use low pressure micro or ultra filtration membranes and eliminate the need for secondary clarification and filtration. The membranes are usually suspended in an aerated tank. The main advantage of MBRs is that they overcome the poor settling of sludge that can occur in convention activated sludge processes.

Pre-engineered package WWTP's used at USACE-AED projects are predominantly aerobic suspended growth (activated sludge) biological treatment processes. Although many variations of the activated sludge process exist, the three which have been most successful for USACE-AED projects are extended aeration, sequencing batch reactor and contact stabilization plants. Figure 1 lists processes applicable to USACE-AED projects including the three named previously.

Figure 1 Classification Sketch of Activated Sludge Variations



Large WWTPs for installations that anticipate average daily flow greater than 378 cubic meter/day (100,000 gpd) are better suited to complete mix or plug flow activated sludge processes that are only considered to be subsets of the activated sludge process on the left side of the above figure.

Table 5 describes the differences between the three types of activated sludge processes highlighted in Figure 1. Note variations of other activated sludge process may be selected for pre-engineered wastewater package plants but the highlighted variations have already been approved and provided successful operation at USACE-AED projects.

Table 5. Description of Activated Sludge Processes for Use at Medium-Size Installations

Process	Description
Extended aeration	A plug flow process that operates for a relatively longer aeration time at lower loading rates resulting in the biochemical degradation of the organic matter as well as d the heterotrophic bacteria in the reaction. Hydraulic residence times are typically 18 to 36 hours and biological loading rates are 64 to 190 kg BOD_5/m^3-day (5 to 15 lb BOD_5 / 10^3- ft^3-day).
Sequencing batch reactor	A fill and draw type biological reactor involving one or two complete mix reactors in which all the steps of the activated sludge process occurs. A separate secondary sedimentation tank is not required, but typically two reactors are required for sequencing. Hydraulic residence times are typically 12 to 50 hours and biological loading rates are 64 to 190 kg BOD_5/m^3-day (5 to 15 lb BOD_5 / 10^3- ft^3-day).
Contact stabilization	Two tanks or compartments are used to mix the influent with the activated sludge; during treatment, the mixed liquid is settled in a secondary settling tank and return sludge is aerated in a separate reaeration basin to stabilize organic matter. Hydraulic residence times are typically 4 to 8 hours and biological loading rates are 765 to 950 kg BOD_5/m^3-day (60 to 75 lb BOD_5 / 10^3- ft^3-day).

Design procedures for these processes are contained in references 4 and 5. The designer shall consider the affect of high temperature and elevation at the project in the calculation of aeration requirement for secondary treatment and accordingly size the air diffusers blowers or mechanical mixers with adequate size motors for the project site conditions.

A typical system consists of the following components.

- Influent manhole that receives and conveys raw wastewater flows to a lift station or headwork where there the wastewater receives comminution and hydraulic energy potential needed to transport the flow through the entire plant (except as provided by recirculation pumps)
- Headwork to provide coarse screening, primary sedimentation and flow measurement
- Flow equalization to store peak flows and recycled activated sludge
- Aeration basin with air diffusers or mechanical aerators with return activated sludge pump capability
- Secondary sedimentation including waste sludge collection and pumping to sludge drying facility
- Sludge drying beds
- Chorine contact chamber for effluent disinfection
- Effluent gravity or pumping system to convey the effluent flow to the proposed outfall location at the receiving surface water point
- Emergency bypass pipe and valve system

Package WWTP systems need to provide safety factors such as equalization basins, generous loading capacity and aeration capacity, and emergency bypass provisions because they do not have the parallel trains required in Reference 3 for permanent WWTP facilities. Parallel components may be required depending upon the size of the package WWTP and the safety factors used in design.

The basis of design for the package WWTP shall be completely documented using the submittal process. Submittal can be through the project design submittal process or using ENG FORM 4025, Transmittal of Shop Drawings, Equipment Data, Material, Samples, or Manufacturer's Certificates of Compliance, however 100% final design submittal shall include all submittal information to provide a single set of documents for future reference. Section 8 describes the submittal requirements.

The major design and operational issues that affect package plants are summarized in Table 6. The design shall provide explanations of how these issues will be mitigated by the proposed system.

Table 6. Types of Design and Operational Issues at Package WWTP

Issue	Discussion
Hydraulic shock loads	Provide equalization tank
Large fluctuations in BOD loading	Provide equalization tank and pump equipment necessary for recirculation
Small flows that are do not self clean conduits and channels	Provide adequate water service to allow washing channels and lift stations
Positive sludge return provisions that allow an extended aeration system to meet all normal requirements	Pump capacity for recirculation ratio of up to 3:1
Adequate provisions for scum or froth and grease removal from the secondary clarifier	Provide scum collection facilities and pumping equipment to return the scum and froth to the equalization tank
Denitrification in final clarifier that results in solids discharge in the effluent	Overflow rates limited to 24 to 33 m^3/m^2 – day 600 to 800 gal/ft^2 – day
Adequate removal and storage of waste sludge	Provide holding tank for sludge storage (may be combined with equalization)
Provision to adjust to large and rapid temperature changes	Size aeration facilities for expected high temperature at project site using site climatic data given in mechanical section of RFP technical requirements
Adequate air supply rate	Adjust air supply rate for altitude and temperature extremes anticipated at project site using site climatic data given in mechanical section of RFP technical requirements

The performance of most package WWTPs can be improved by sizing facilities with using conservative design parameters and providing sufficient storage for handling side stream flows.

7. Lagoon Treatment Systems

Lagoon wastewater systems are earthen basins that are engineered for BOD and total suspended solids removal similar to package WWTPs; however, they are sized such that the biological loading rate is lower than traditional aerobic suspended growth processes such as in pre-engineered package WWTP plants. In addition BOD is removed by sedimentation. These systems are capable of providing effluent quality that meets the technical requirements, however, they are affected by climate conditions and shock loading and require conservative design factors. They provide significant removal of pathogens as a result of natural die-off, sedimentation, and adsorption.

The design requirements include embankment design, impermeable layer or liner design, erosion control design both for the embankment and outlet features, and hydraulic design for pipe lines and lift stations (if needed). These design requirements are considered in other design guides and Unified Facility Criteria (UFC). The information presented in this section demonstrates the type of design information required for project submittal of WWTPs involving lagoons. The information may need to be expanded or adapted for the specific project site.

Two types of lagoons are applicable to USACE-AED projects that are described in Table 7.

Table 7. Description of Lagoon Treatment Systems for Use at Medium-Size Installations

Type of lagoon	Description
Facultative	The surface zone is aerobic as a result of wind shear mixing, but the subsurface zone may be anaerobic or even anoxic. Hydraulic residence times are typically 25 to 180 days and biological loading rates are 64 to 190 kg BOD_5/cu m-day (20 to 60 lb BOD_5 / ac-day). Hydraulic depths are typically 1.5 to 2 meters.
Aerated Partial-Mix	Surface aeration using mechanical equipment produces a surface aerobic zone that ranges in depth depending upon the amount and efficiency of oxygen transferred and the lagoon depth Hydraulic residence times are typically 5 to 20 days and biological loading rates are 64 to 190 kg BOD_5/cu m-day (5 to 15 lb BOD_5 / 10^3- ft^3-day). Hydraulic depths are typically 3 to 5 meters.

The principal advantage of aerated lagoons is that they require less land area and because they are smaller they may be more economical considering lining and covers. They are more expensive to construct and operate and maintain, and the construction of liners that do not fail and are capable of enduring the weight of maintenance vehicles requires more experience. Liner design is a specialized engineering topic and not discussed in this design guide.

The principle design concept for facultative lagoons is to provide sufficient detention time and low organic loading that the surface layer of the lagoon remains aerobic. However these lagoons are subject to wind mixing and thermal turnover at times of the year that can create odors.

Pretreatment of influent for lagoons shall be designed to minimize floatables, scum, and large objects from being transported in to the lagoons. Bar screens are used in combination with primary sedimentation to remove rags, large objects and grit.

Design procedures for facultative lagoons shall be based on area biological loading rates shown in Table 8.

Table 8. Area BOD_5 Design Loading of Facultative Lagoon Treatment Systems

Average winter temperature, ^oC	Kg/ha-day	Lb/ac-day
Greater than 15	45-90	40-80
0-15	22-45	20-40
Less than 0	11-22	10-20

Notes: Source – reference 6

Design procedures for aerated partial mix lagoons shall be based on hydraulic detention times shown calculated using the following equation:

$$t= (n/k)*((Co/Cn)^{(1/n)}-1)) \qquad \text{Eq. 1}$$

where t= hydraulic detention time, days
n= number of equal sized cells
k= BOD removal constant rate, per day
Co= influent BOD concentration to first cell in series, mg/l
Cn= effluent BOD concentration from cell n, mg/l

The BOD removal rate used in 0.276 day^{-1} based on reference 7. The rate constant is adjusted for the design temperature using the following equation:

$$k_2/k_{20}=theta^{(T_2-T_{20})} \qquad \text{Eq. 2}$$

where k_2= BOD removal rate constant at T_2
k_{20}= BOD removal rate constant at T_{20}
T_{20}= 20° C
T_2= average winter temperature ^{o} C

The lagoon area is calculated as the design flow multiplied by the hydraulic detention time divided by the hydraulic depth of the lagoons. In the example below, average winter temperatures were used in equation (2) to compute hydraulic detention times from equation (1) to size three lagoon systems, each containing "n" lagoons in sequence, in 3 locations at 4 different design flow rates. In every case the lagoon hydraulic depth is 3 meters. Results are provided in Table 9. The table shows the size for each lagoon for the number of cells shown in the left column.

Table 9. Partial-Mix Aerated Lagoon Treatment Systems

	DESIGN FLOW RATE, cubic meters/day											
	61			**120**			**340**			**680**		
	Lagoon size, hectare			Lagoon size, hectare			Lagoon size, hectare			Lagoon size, hectare		
number of cells, n	Kandahar	Kunduz	Kabul	Kandahar	Kunduz	Kabul	Kandahar	Kunduz	Kabul	Kandahar	Kunduz	Kabul
1	0.30	0.28	0.38	0.60	0.56	0.74	1.70	1.58	2.10	3.40	3.17	4.20
2	0.11	0.10	0.14	0.22	0.20	0.27	0.62	0.58	0.77	1.24	1.16	1.54
3	0.08	0.08	0.10	0.16	0.15	0.20	0.46	0.43	0.57	0.92	0.86	1.14

Notes: Basis of calculations shown in reference 6

Reference 6 provides the basis for the size calculations and can be used for sizing lagoons with various depths at other locations. Appendix A provides climate conditions for various locations in Afghanistan. Aeration system shall be designed for the maximum summer temperatures at the altitude of the project region.

Aeration requirements partial-mix lagoons can be met by floating mechanical aerators. The key to successful aeration is to meet the oxygen demand of the wastewater and to apply the aeration in the upper portion allowing the lower portion to remain quiescent to enhance sedimentation and anaerobic decomposition.

The oxygen transfer rate at standard temperature (20° C) and one atmosphere pressure can be calculated from the following equation

$$SOTR = AOTR/[(beta*Cs_{TH}-C_L)/C_{s20})]*(1.024^{\wedge(T-20)})*alpha \qquad \text{Eq. 3}$$

Where AOTR = actual oxygen transfer rate required under field conditions, kg/h
SOTR = standard oxygen transfer rate at 20° C standard conditions, kg/h
beta = salinity-surface tension correction factor, assumed = 0.95
Cs_{TH} = dissolved oxygen saturation concentration in clean water at temperature T and altitude H
C_L = operating dissolved oxygen concentration, typically 2 mg/L
C_{S20} = dissolved oxygen saturation concentration in clean water at 20° C shown in Appendix B
T = wastewater temperature, ^{o} C
H = altitude, meter
alpha = oxygen transfer correction factor for wastewater, typically 0.6 (unit-less)

Mechanical aeration shall be designed for the average high summer temperature and the project site altitude. Appendix A provides outside climate conditions and altitude information at various locations in Afghanistan. Higher temperatures and altitudes increase the mechanical energy required for oxygen transfer because of the lower oxygen solubility at higher temperature and lower oxygen content of less dense air (at higher altitudes). Table 10 presents three examples of the type of information required for aerated lagoon design submittals.

Table 10. Partial-Mix Aerated Lagoon Aeration Requirements

Field condition - project	Kandahar	Kunduz	Kabul
elevation, m	1010	432	1790
elevation, ft	3314	1417	5876
summer high, o C	41	39	34
summer high, o F	106	102	93
AOTR, kg/day	730	720	760
SOTR, kg/day	2200	2000	2260
total aerator, kw	48	43	49
CsTH, mg/l	5.7	6.2	5.6

Mechanical aerators are rated in terms of their standard oxygen transfer rate (SOTR) at standard conditions of temperature and pressure. This rate shall be estimated by using Eq.3 adjusting the AOTR for field conditions at the project site.

8. System Submittals

All of the information about the design process and plant selection should be included within the design for review by USACE-AED. Package WWTP shop submittals by the contractor shall include the following:

1. Wastewater treatment plant site plan. The plan shall identify required setbacks, buried utilities (other than sewer) and potential points of connection to the existing electrical distribution system (existing on the compound). The set back requirements for the wastewater treatment plant site shown on a drawing to identify the buildable footprint. Setback requirements shall be obtained from applicable technical criteria listed in Section 01015.
2. Prepare a summary of basis of design report to include the following
   Design average daily loading for the wastewater treatment plant based on the design

equivalent population (See Section 01010) to be used as the basis for wastewater loading. Provide the design peak hourly flow rates for the lift station and head works of the plant. The anticipated waste loading in terms of biochemical oxygen demand (BOD_5) and total suspended solids (TSS) loading shall be shown for the plant based on wastewater characteristics in the Section 01015.

3. Prepare a treatment train diagram that includes process modules: lift station pumping and flow metering, headwork processes (e.g. grit, trash and scum removal), equalization chamber, primary treatment processes (e.g. sedimentation and aeration), secondary treatment process including aeration and secondary clarification, sludge and scum recirculation systems, effluent chlorination and outfall system, sludge holding/digestion and disposal. All process component tankage shall be above ground steel tank. The biological treatment process shall be activated sludge with extended aeration. The general dimensions of each module shall be specified based on the design capacity requirements of each module shall be shown. The lift station pumps, wet well, hatches, and pump controls, inlet bar screen, distribution weirs and pipe sizes, pump capacity and power requirements, the aeration equipment (diffusers, piping, and blower), and froth control equipment requirements shall be stated. Actual oxygen transfer rates used in the sizing of aeration equipment based on site elevation and high summer temperature range shall be shown. Return ratio ranges for sludge recirculation shall be estimated. Provide pipe and pump capacities on sludge recycling and wasting and calculated sludge storage and disposal volumes required. Show all emergency bypass piping and valve locations.
4. Provide catalogue information for the proposed lift station and treatment plant equipment. Include information on manufactured steel tanks. Information shall include material standards for field welding of tanks, piping, valves, pumps, pump control equipment, bolts, gaskets, electrical enclosures, central systems control panels, motors and generators, aerators, air piping, blowers, bar screens, handrails, access ladders and service walkway gratings. Provide an access ladder anti-climb gate a minimum of 4 feet above the ladder base to secure the plan from unauthorized entry. Include product coating and lining, and cathodic protection specifications for applicable equipment.
5. Electrical feeds to the waste water treatment plant shall be sized to limit the voltage drop to a total of 5% from the separately derived electrical system to the piece of equipment that is being powered. All breakers, cables and equipment shall be installed and sized in accordance with the latest edition of the National Electrical Code. A layout plan of all the electrical equipment that includes location of the motor control centers, pumps, panels and grounding shall be submitted. The contractors shall also submit panel schedules, motor control center (MCC) layout (with MCC equipment sizes), cable sizes, and electrical equipment sizes and ratings. The contractor shall provide all calculations to show how the equipment was sized.
6. An evaluation of the sludge disposal options and recommendation of the preferred disposal option. The proposed sludge disposal equipment/process requirements shall be shown on a plan drawing.
7. Complete the checklist shown in Table 11 summarizing design parameters, and reference to the drawings where the features are shown. Clearly show dimensions and numbers of each component.

Table 11. Summary of WWTP Design Parameters to be Included in Submittal

Parameter	Design Value	Units	Source or Equation[1] (with supporting calculations shown)	Construction Drawing and Catalogue Reference Number
Average daily flow (ADF)		m^3/d		
Peak hour flow (PHF)		m^3/hr		
Equalization storage volume [7]		m^3		
Biological loading (BOD_5) [2]		kg BOD_5 /day		
Primary Reactor Volumetric organic loading rate, $L_{organic}$		kg/ BOD_5 /10^3 m^3 –day [3]		
Mixed Liquor Suspended Solids (MLSS)		mg/L		
Food:Microorganism Mass (FM) ratio		kg BOD_5 /MLVSS-day		
Site conditions; high and low temperature range; altitude (see section 01015)				
Mechanical aerator or differ system actual oxygen transfer rate (AOTR) under field conditions temperature and atmospheric pressure (altitude) at project site		kg O_2 /h		
Standard oxygen transfer rate (SOTR) under standard pressure and temperature conditions per manufacturer[4]		kg O_2 /h		
Number and power requirements of each aerator or diffuser blower system for each treatment unit (indicate standby units)		kW/number		
Estimate of annual energy required based on ADF		kWh		
Secondary clarifier overflow rate		m^3/ m^2 -day		
Disinfection chamber volumetric hydraulic residence time		minutes [5]		
Sludge storage volume		m^3		
Sludge drying bed loading rate (if applicable)		m^2/person-month [6]		
Effluent channel peak flow capacity		m^3/sec		
Panel schedules, motor control center (MCC) layout (with MCC equipment sizes), cable sizes, and electrical equipment sizes and ratings		varies		

Notes:
1. Documentation for sources shall cite name and date of standard, data collection, or if an equation, all parameter symbols shall be defined used in the equation
2. if process provides nitrogenous oxygen demand removal, include this loading
3. if other basis for reactor sizing is used such as mean cell residence time or F/M ratio provide, appropriate units
4. provide documentation from equipment manufacturer of aerator or diffuser system
5. minimum 30 minute contact time
6. only open beds are permitted
7. Equalization storage shall be adequate for flow variation over 24-hour period as shown in Appendix C

The lagoon system submittals shall include the additional following information in addition to Table 11 data:

1. Liner or impervious layer design for the pond including plan and elevation drawings showing liner/layer thickness, anchoring for synthetic line, side slope, liner/layer bedding preparation, pond cover layer protection, access route for equipment entry into pond for maintenance, synthetic layer specifications and details of pipe penetrations through liner/layer.
2. Mechanical aeration equipment design and specifications.
3. Size of conveyance channel or pipes and invert elevations at each pond.

All submittals shall include the WWTP proposed start-up testing and training program in including operation and maintenance manuals. When the wastewater system construction nears completion and all units are operative, the contractor shall commence a commissioning and startup procedure for the treatment system. The treatment system includes all lift stations, force main, gravity sewers, treatment plant units and associated equipment, sludge holding and digestion, septage dump pad, and laboratory building. The contractor will operate the treatment facility for a trial period of two months performing all daily and weekly operation and maintenance (O&M) tasks recommended by the equipment manufacturer. The contractor shall utilize services of qualified operators; including the use of at least two Afghan Nationals that the contractor shall train. During the routine O&M, the contractor shall perform all sampling and testing necessary to ensure proper daily operations in achieving the required effluent standards. The contractor shall maintain a log that includes records of daily O&M activities, e.g. repairs, inflow measurement, aeration cycles, effluent cycling, waste and return sludge pumping, and sludge drying. The contractor shall also maintain and operate the sludge disposal operation during the trial period.

Project submittal shall include markup UFGS specification 44 41 13 Prefabricated Biochemical Wastewater Treatment Plant. The specification paragraph 1.2.1 “Extended Aeration Plant Design” shall be changed to Activated Sludge Plant Design Type [extended aeration], [contact stabilization], [sequencing batch reactor]”. Strike out the inapplicable type. Appendix D contains a list of the specific strike-out topics to be edited by the designer.

9. As-Builts

Upon completion of installing the package wastewater treatment plant, 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.

10. References

1. Overseas Environmental Baseline Guidance Document, Department of Defense, 2007
2. Standard Methods for the Examination of Water and Wastewater, American Public Health Association (APHA) publication, current edition.
3. UFC 3-240-09FA Domestic Wastewater Treatment, 2004
4. Small and Decentralized Wastewater Management Systems, Crites and Tchobanoglous, Metcalf & Eddy, McGraw- Hill, 1998
5. Wastewater Engineering Treatment and Reuse, Metcalf & Eddy, McGraw- Hill, 2003
6. US EPA. Municipal Wastewater Stabilization Ponds – Design Manual, EPA-625/1-83-015, 1983
7. Report of the Wastewater Committee of the Great Lakes – Upper Mississippi River Board of State and Provincial Public Health and Environmental Managers. Ten State Standards for Wastewater Facilities. 2004

Appendix A - Outside Design Climate Conditions

Bagram Area			**Darualaman Area**	
Latitude:	35.0 deg North		**Latitude:**	34.42 deg North
Longitude:	69.0 deg East		**Longitude:**	69.11 deg East
Elevation:	1490 M (4888 Ft)		**Elevation:**	1737 M (5700 Ft)
Summer Temp:	DB: 35° C (95° F)		**Summer Temp:**	DB: 34° C (93° F)
	WB: 18.6 ^{o}C (66 ^{o}F)			WB: 15.6 ^{o}C (60 ^{o}F)
Winter Temp:	-12.8 ^{o}C (9 ^{o}F)		**Winter Temp:**	-8 ^{o}C (18 ^{o}F)
Daily Range:	18.3 ^{o}C (33 ^{o}F)		**Daily Range:**	8 ^{o}C (15 ^{o}F)
Farah Area			**Gardez Area**	
Latitude:	32.22 deg North		**Latitude:**	33.6 deg North
Longitude:	62.11 deg East		**Longitude:**	69.22 deg East
Elevation:	700 M (2297 Ft)		**Elevation:**	1737 M (7710 Ft)
Summer Temp:	DB: 41.1^{o} C (106° F)		**Summer Temp:**	DB: 29° C (84° F)
	WB: 22.5 ^{o}C (72.5 ^{o}F)			WB: 12.2 ^{o}C (54 ^{o}F)
Winter Temp:	1.6 ^{o}C (35 ^{o}F)		**Winter Temp:**	-10 ^{o}C (14 ^{o}F)
Daily Range:	unknown		**Daily Range:**	unknown
Ghanzni/Khair Kot Area			**Herat Area**	
Latitude:	33 deg North		**Latitude:**	34.22deg North
Longitude:	68 deg East		**Longitude:**	62.22 deg East
Elevation:	2183 M (7162 Ft)		**Elevation:**	964 M (3163 Ft)
Summer Temp:	DB: 30.5^{o} C (87° F)		**Summer Temp:**	DB: 38° C (100° F)
	WB: 15.6 ^{o}C (60 ^{o}F)			WB: 20 ^{o}C (68 ^{o}F)
Winter Temp:	-7.2 ^{o}C (19 ^{o}F)		**Winter Temp:**	-6 ^{o}C (21 ^{o}F)
Daily Range:	unknown		**Daily Range:**	9 ^{o}C (17 ^{o}F)
Jalalabad Area			**Kabul Area**	
Latitude:	34 deg North		**Latitude:**	34.31deg North
Longitude:	70 deg East		**Longitude:**	69.12 deg East
Elevation:	580 M (1903 Ft)		**Elevation:**	1800 M (5900 Ft)
Summer Temp:	DB: 39.6^{o} C (103° F)		**Summer Temp:**	DB: 19° C (68° F)
	WB: 25.6 ^{o}C (78 ^{o}F)			
Winter Temp:	4.6 ^{o}C (40 ^{o}F)		**Winter Temp:**	-5 ^{o}C (42 ^{o}F)
Daily Range:	unknown		**Daily Range:**	11 ^{o}C (20 ^{o}F)

Appendix B - Solubility of Oxygen in Fresh Water

Temperature		Saturation DO		Temperature		Saturation DO	
°C	°F	mg/l	$1.035^{(T-20)}$	°C	°F	mg/l	$1.035^{(T-20)}$
0	32	14.6	.503	21	69.8	9.0	1.035
1	33.8	14.2	.520	22	71.6	8.8	1.071
2	35.6	13.8	.538	23	73.4	8.7	1.109
3	37.4	13.5	.557	24	75.2	8.5	1.148
4	39.2	13.1	.577	25	77.0	8.4	1.188
5	41.0	12.8	.597	26	78.8	8.2	1.229
6	42.8	12.5	.618	27	80.6	8.2	1.272
7	44.6	12.2	.639	28	82.4	7.9	1.317
8	46.4	11.9	.662	29	84.2	7.8	1.363
9	48.2	11.6	.685	30	86.0	7.6	1.411
10	50.0	11.3	.709	31	87.8	7.5	1.460
11	51.8	11.1	.734	32	89.6	7.4	1.511
12	53.6	10.8	.759	33	91.4	7.3	1.564
13	55.4	10.6	.786	34	93.2	7.2	1.619
14	57.1	10.4	.814	35	95.0	7.1	1.675
15	59.0	10.2	.842	36	96.8	7.0	1.739
16	60.8	10.0	.871	37	98.6	6.9	1.795
17	62.6	9.7	.902	38	100.4	6.8	1.858
18	64.4	9.5	.934	39	102.2	6.7	1.923
19	66.2	9.4	.966	40	104.0	6.6	1.990
20	68.0	9.2	1.000				

Temperature effect on $E_t = E_{20} \cdot (T-20)$

[1] To be used for calculating oxygen transfer capability.

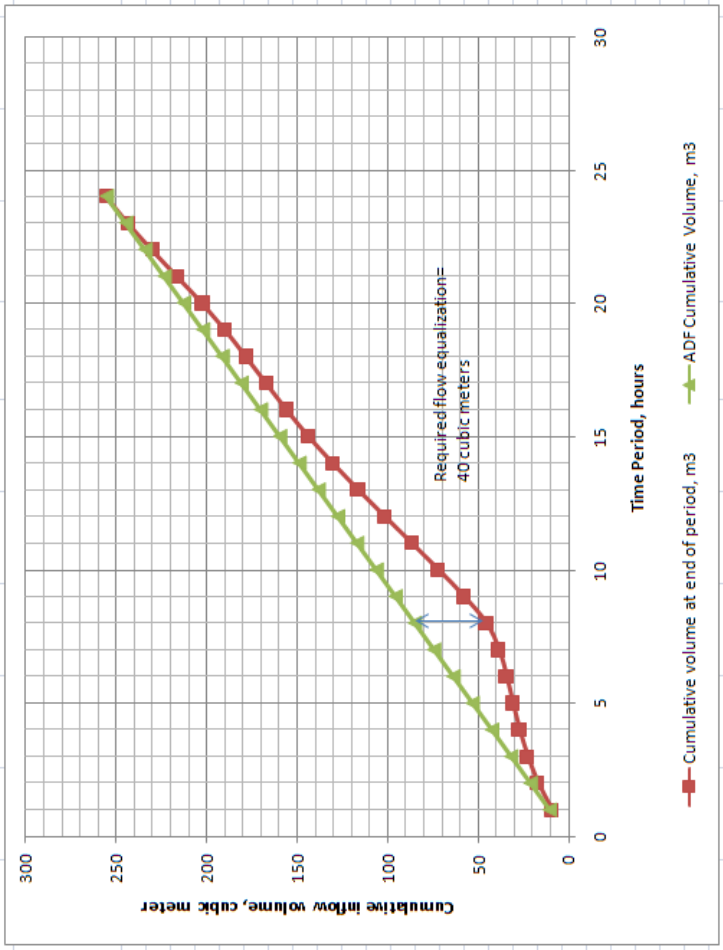
[2] To be used for trickling filter design utilizing Howland Formula.

Appendix C – Determination of Equalization Storage for Package WWTP

EQUALIZATION BASIN STORAGE VOLUME

Hydraulic loading 255,588 L/day
255.6 cu m/day
0.003 cum m/sec

Fraction of ADF	Interval	Time Period	Average flow rate in time period, m^3/sec	Cumulative volume at end of period, m^3	ADF Cumulative Volume, m^3
0.895	1	M-1	0.003	10	11
0.716	2	1-2	0.002	17	21
0.537	3	2-3	0.002	23	32
0.423	4	3-4	0.001	27	43
0.342	5	4-5	0.001	31	53
0.325	6	5-6	0.001	34	64
0.391	7	6-7	0.001	39	75
0.667	8	7-8	0.002	46	85
1.155	9	8-9	0.003	58	96
1.334	10	9-10	0.004	72	106
1.383	11	10-11	0.004	87	117
1.399	12	11-N	0.004	102	128
1.383	13	N-1	0.004	117	138
1.318	14	1-2	0.004	131	149
1.253	15	2-3	0.004	144	160
1.139	16	3-4	0.003	156	170
1.058	17	4-5	0.003	167	181
1.058	18	5-6	0.003	179	192
1.074	19	6-7	0.003	190	202
1.188	20	7-8	0.004	203	213
1.302	21	8-9	0.004	217	224
1.302	22	9-10	0.004	230	234
1.237	23	10-11	0.004	244	245
1.123	24	11-M	0.003	256	256
Average daily flow from design above			**0.003**		



Appendix D – Summary of Markup Pages for UFGS 44 41 13 – Prefabricated Biochemical Wastewater Treatment Plant

Topic	Page	Edit Options	Comments	References
Paragraph 1.2.1 Extended Aeration	4	"Extended Aeration Plant Design" shall be changed to Activated Sludge Plant"		
Paragraph 1.2.1 Activated Sludge Plant	4	Insert Select Design Type: [extended aeration], [contact stabilization], [sequencing batch reactor] – strike others		USACE –AED Package Wastewater Treatment Plant Design Requirements, 2009
Paragraph 1.2.1 Activated Sludge Plant	5	Insert design and peak flow rates for plant (L/day) in brackets	Select peak flow as 2.5 times average day flow	UFC 3 240-09fa
Paragraph 1.2.1 Activated Sludge Plant	5	Insert 400 mg/L for BOD_5 and 400 mg/L for TSS; Insert 92.5 % for removal		Effluent standards shall meet requirements in "Overseas Environmental Baseline Guidance Document", Department of Defense, 2007
Paragraph 1.2.1 Activated Sludge Plant	5	Strike last sentence in paragraph concerning seismic design		
Paragraph 1.2.1.1 Aeration Zone	5	Insert minimum wastewater depth and freeboard; insert influent pipe diameter		
Paragraph 1.2.1.2 Sludge Settling Zone	5-6	Insert surface loading rate at average daily flow rate and surface loading rate at peak hourly flow rate; edit scum baffle height		
Paragraph 1.2.1.3 Sludge Holding Zone	6	Insert sludge holding capacity		
Paragraph 1.2.1.4 Chlorination Zone		Insert chlorination zone effluent pipe diameter		
Paragraph 1.2.2	6-8	Delete in entirety		
Paragraph 1.3 Submittals	8	Insert USACE-AED-EC in bracket after "G"		
Paragraph 1.6 Extra Materials	10	Insert one month in bracket		
Paragraph 2.4 Sewage Shredder	13	Indicate zone; add batch reactor plant		
Paragraph 2.4.3 Sewage Shredder Motor and mounting	14	Insert motor voltage, phase and change 60 to 50 Hz		
Paragraph 2.5.1 Blower Capacity	14	Inset blower capacity SCFM and pressure kPA		
Paragraph 2.5.2 Blower Drive	15	Insert blower, rpm, motor voltage and phase; revise 60 to 50 Hz		
Paragraph 2.5.5 Blower Drive	15	Insert pressure relief valve set-point pressure		

Notes: 1. Other than deletions/edits shown in this table the standard specification 44 41 13 shall be retained in its entirety

Appendix D – Summary of Markup Pages for UFGS 44 41 13 – Prefabricated Biochemical Wastewater Treatment Plant (Continued)

Topic	Page	Edit Options	Comments	References
Paragraph 2.6.1 Air Diffusion Performance and Design requirements	16	Insert 3 cubic meters of air per 100 cubic meters of tank volume		
Paragraph 2.6.2 Air Diffusion Piping and Valves	16	Select piping material and state number of separate diffusers per tank		
Paragraph 2.7.1 Sludge and Scum Piping and Valves	17	Select zone for sludge recirculation; for batch reactor indicate sludge storage		
Paragraph 2.7.2 Sludge and Scum Air lift Pumps	17	Insert sludge draw or recirculation percent; add frequency		
Paragraph 2.9.1 Gas Chlorinator	19-20	Delete paragraph and all subparagraphs	Use calcium hypochlorite solution chlorinator, paragraph 2.10	
17Paragraph 2.10.2 Calcium Hypochlorite Chlorination System Metering Pump	21	Select type of metering pump, motor enclosure, motor voltage, phase, and change 60 to 50 Hz		
Paragraph 2.11 Sewage Pumps	22	Select final effluent as required		
Paragraph 2.11.1 Sewage Pumps	22	Select motor enclosure		
Paragraph 2.12 Flow meter and Control	22-23	Delete all but one applicable paragraph leaving one paragraph as flow metering method		
Paragraph 3.5.2 Welding	25-26	Delete paragraph and subparagraphs		
Paragraph 8 Field training	27	Insert hours of training	As per contract technical requirements or minimum per manufacturer's recommendation; whichever greater	

Notes: 1. Other than deletions/edits shown in this table the standard specification 44 41 13 shall be retained in its entirety

US Army Corps
of Engineers
Afghanistan Engineer District

AED Design Requirements: Sanitary Sewer & Septic System

Various Locations,
Afghanistan

June 2010
Version 1.6

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

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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:

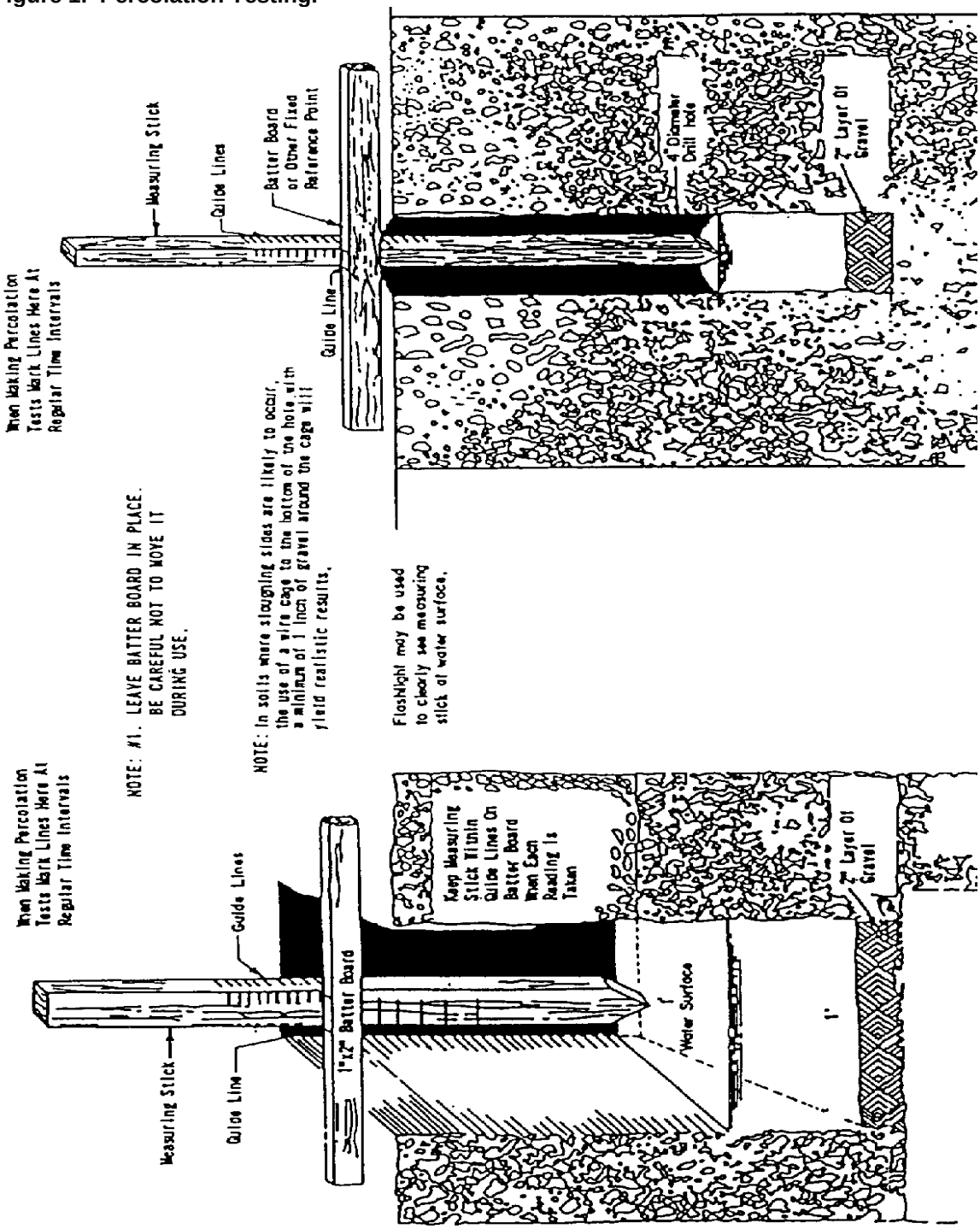
Example 1: Calculating Percolation Rates - In 30 minutes, the measured drop in the water level is 15 mm.

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.

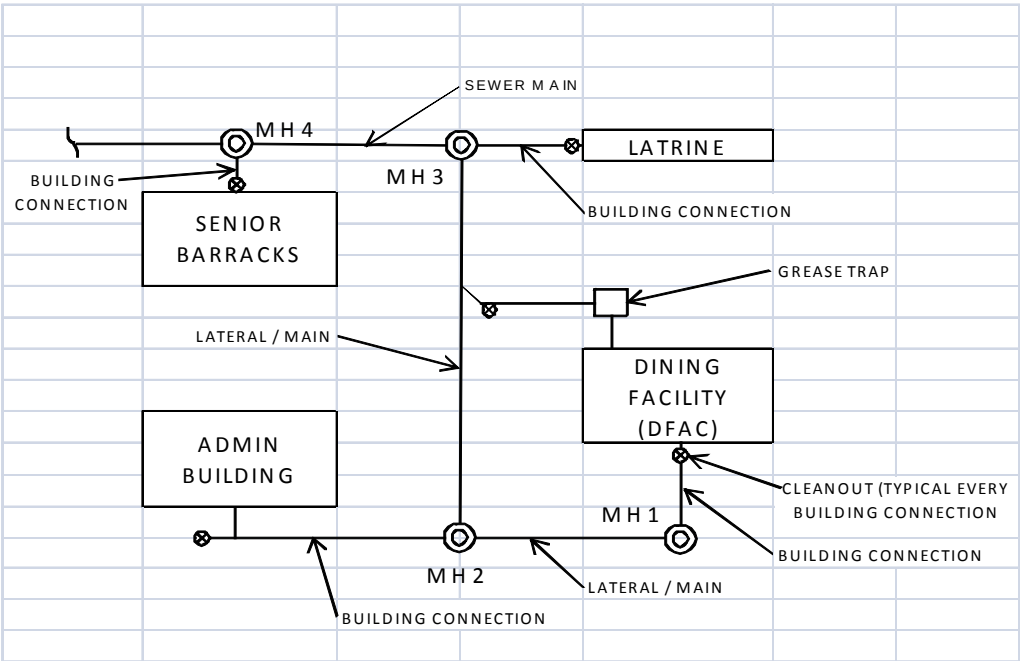


Figure 2. Schematic Pipe Definitions

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. Maximum depth for leach field percolation pipe lines shall be one (1) meter to allow for air exchange with the surface.

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.

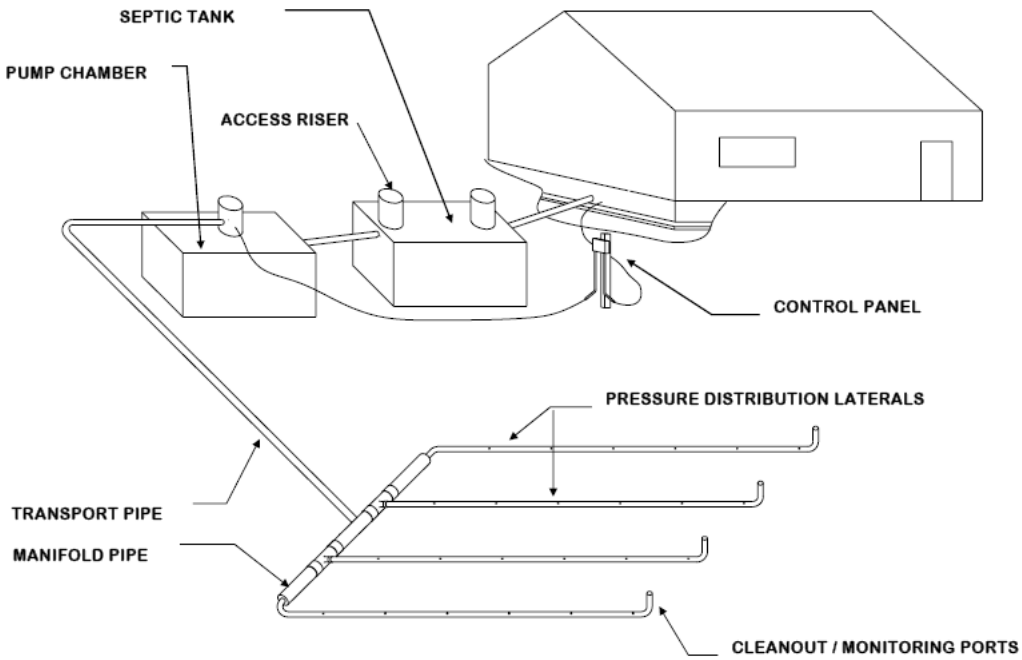
7) Manifold Pipe - The manifold pipe is located between the transport pipe and the laterals in the leach field. This pipe is sized so that there is no more than a 15% variation in the rates of discharge between the first and last orifices in the network. The PVC pipe shall conform to ASTM D2241 PVC Pressure-Rated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

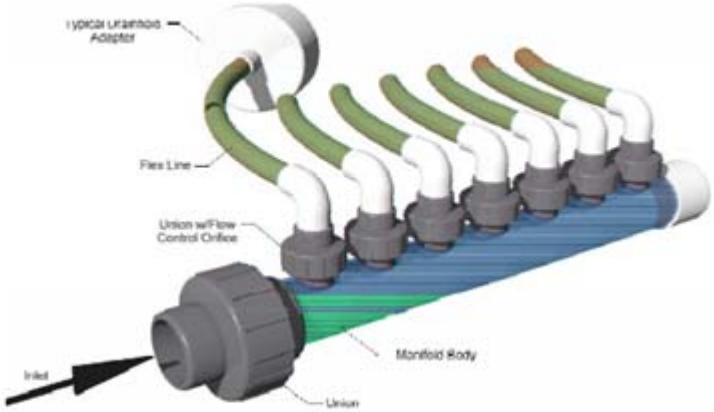
   Clean outs - Clean outs shall be placed at the ends of the laterals, a minimum of 0.5 meter above ground. They should have treaded caps at their ends to allow for inspection and cleaning of the laterals. In cold climates they should be insulated to prevent the laterals from freezing.

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

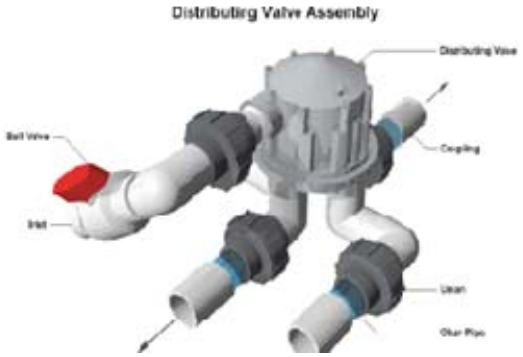
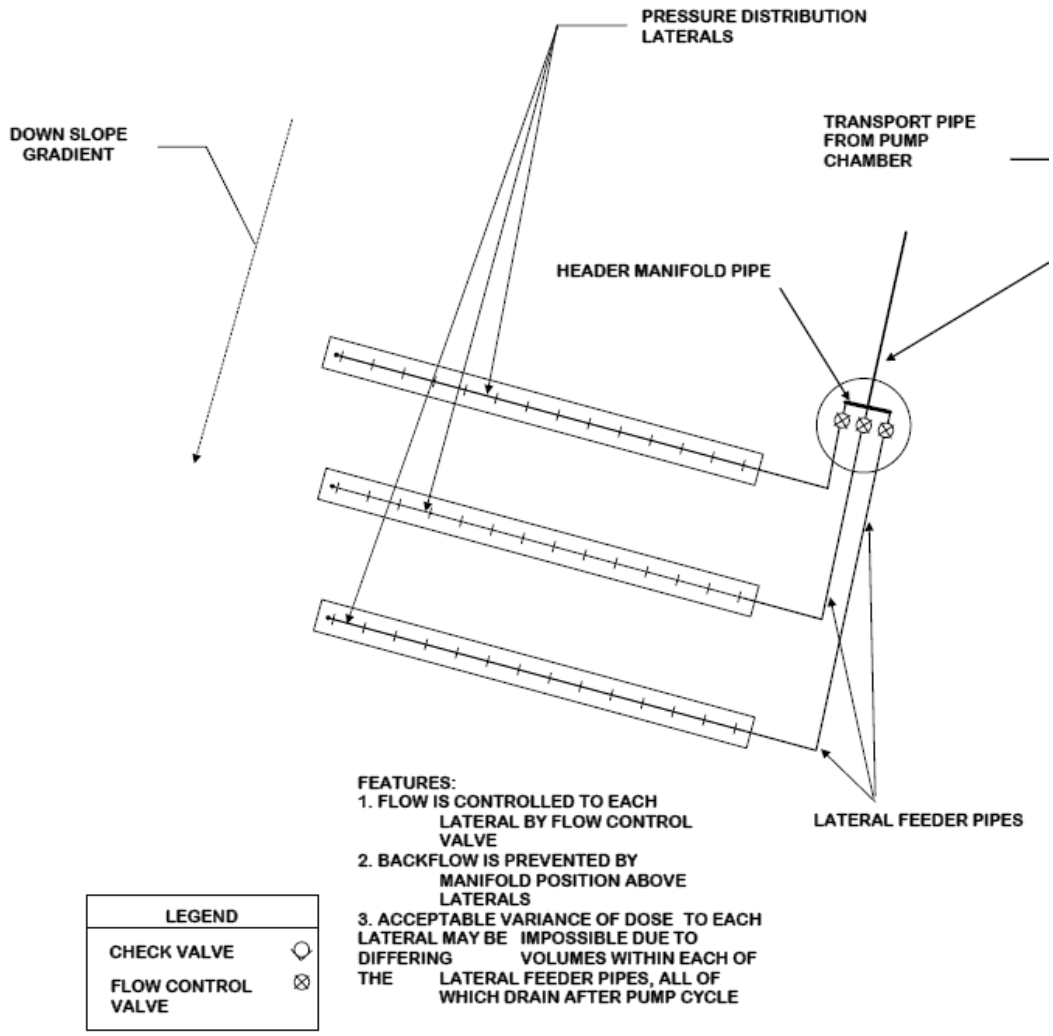


Figure 5. Typical Flow Distribution Valve

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.

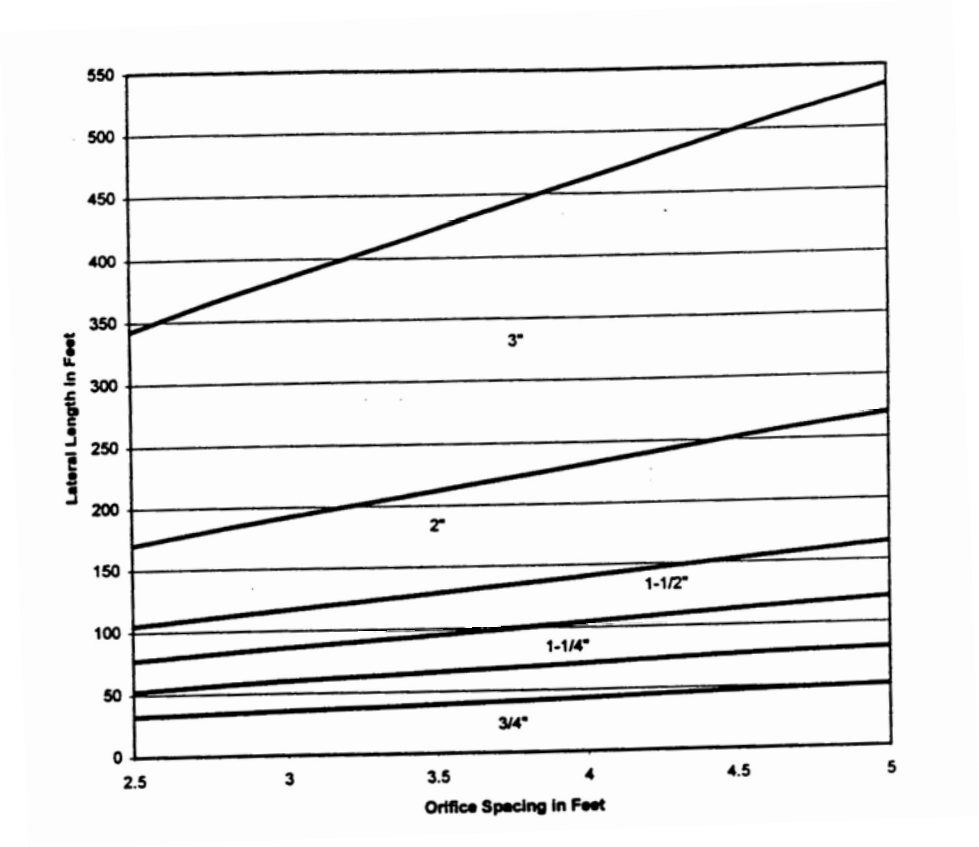
Spacing = (area/orifice x number of laterals / (absorption area width).

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.

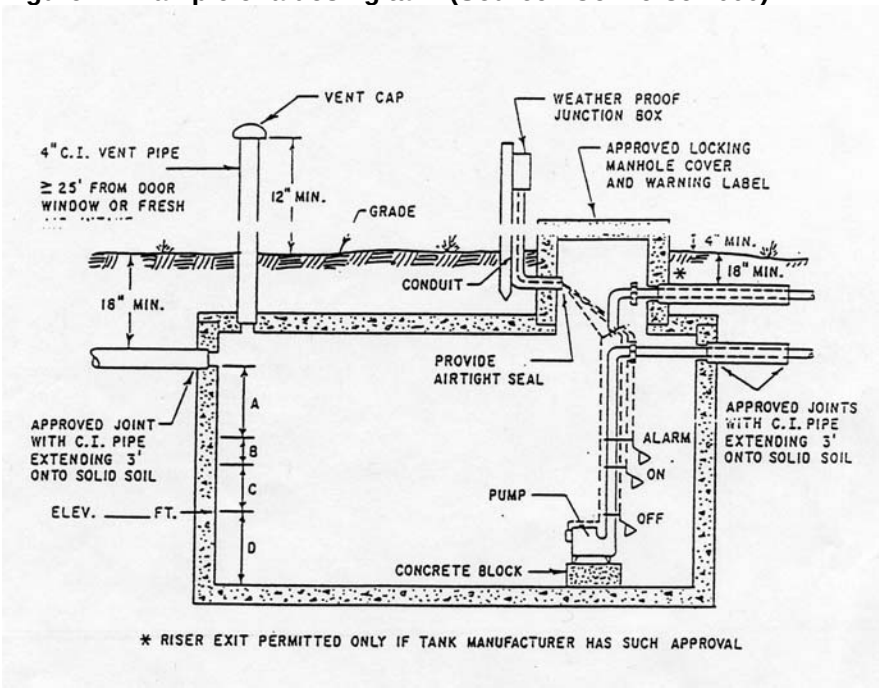
Step 4. **Size the dose chamber.**

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.

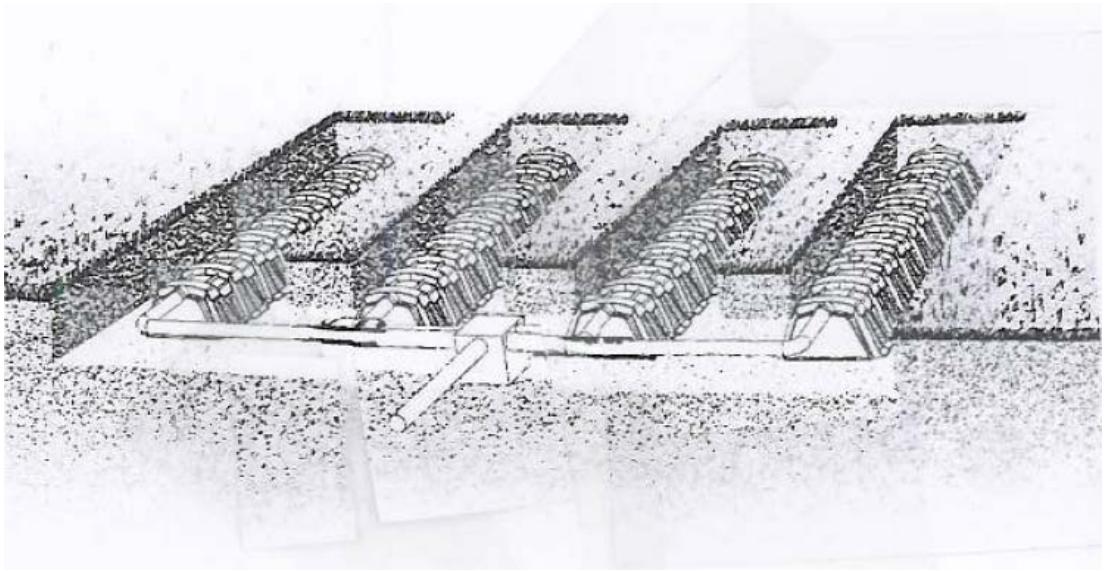


Figure 8. Example of Leaching Chamber Trench Installation

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.

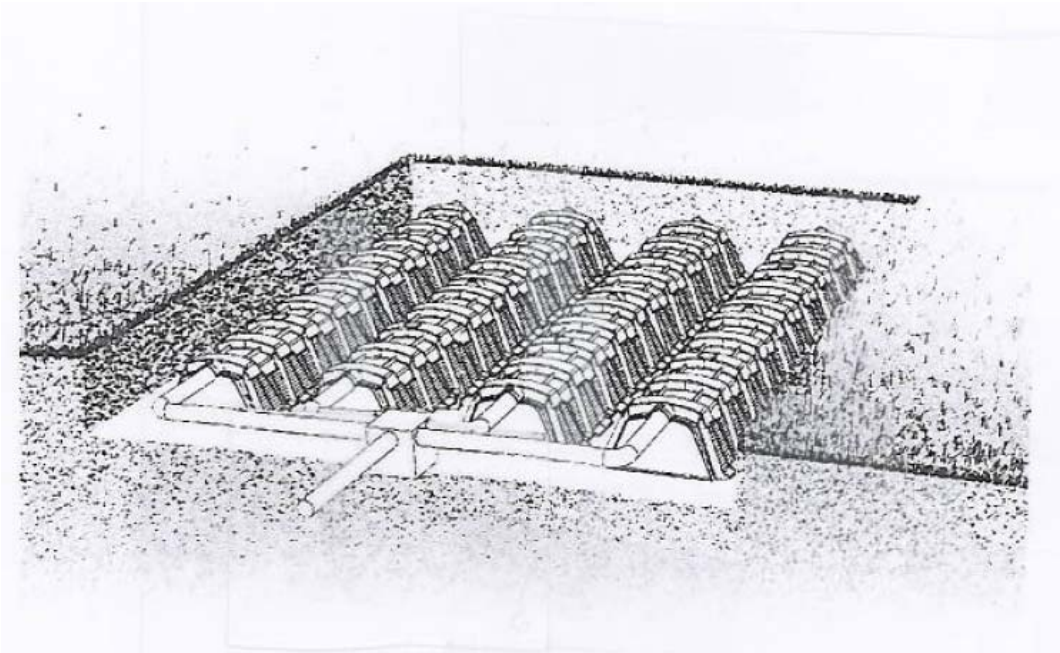


Figure 9. Example of Leaching Chamber Bed Installation

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.

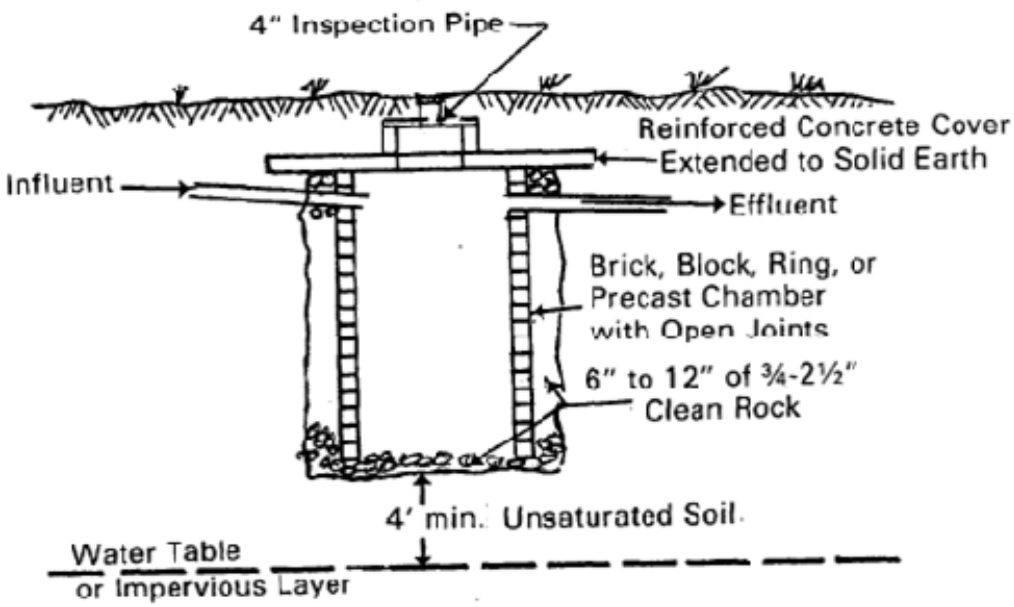


Figure 10. Seepage Pit Cross Section

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 below the seepage pit inlet.

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

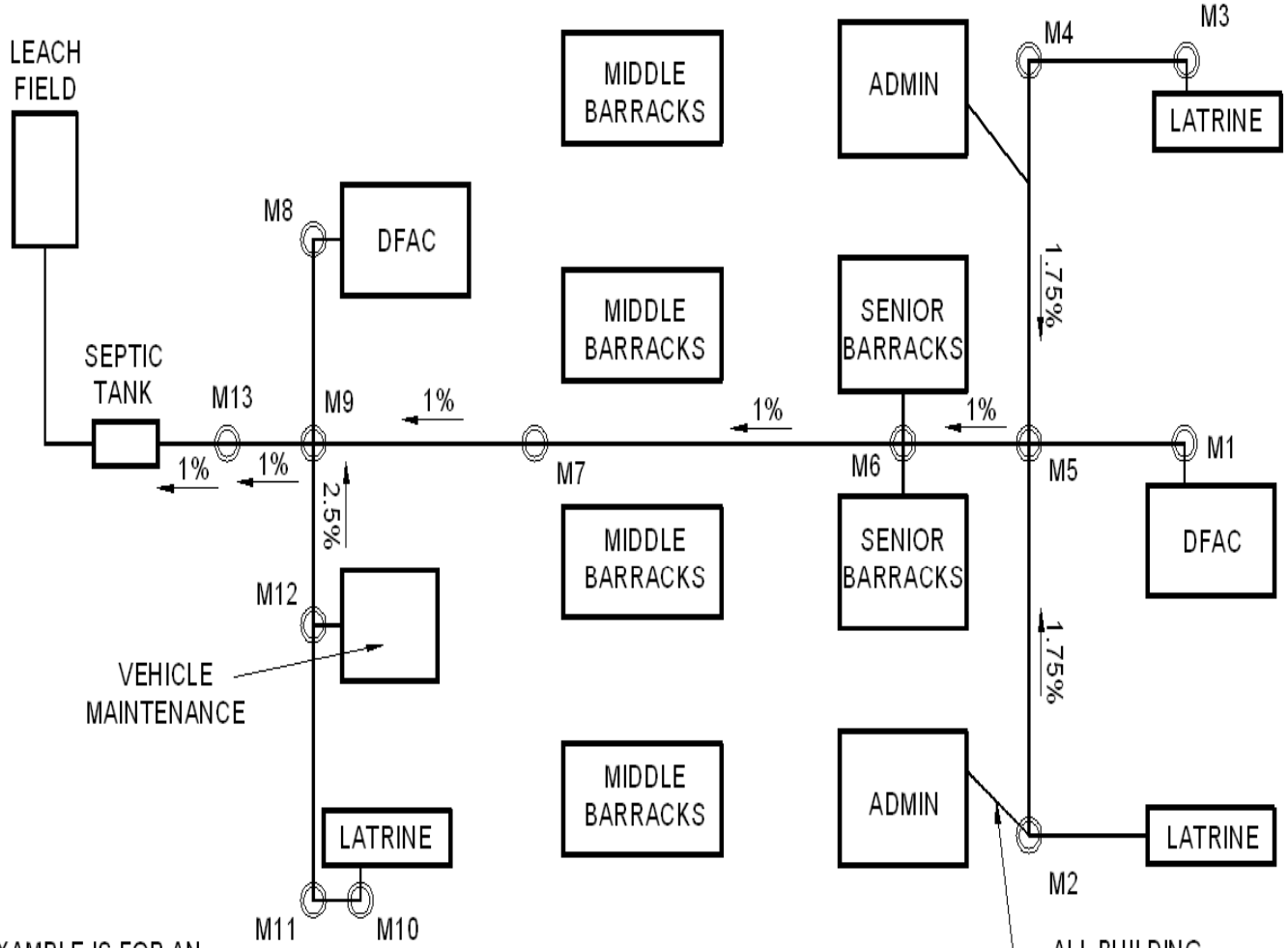
The objectives of the analysis for the system include:

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

AED Design Requirements
Sanitary Sewer & Septic Systems



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.

ALL BUILDING CONNECTIONS ARE AT 1%

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		

Indicates designer entered value
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 Lps	Slope	n	diameter mm	diameter m	Area (Full) mm^2	Area (Full) m^2	R mm	R m	Vel full,Vf m/s	flow Full, Qf m^3/s	flow Full, Qf Lps	Qs/Qf Fixture	Vs/Vf Fixture		d/D Fixture	Vs, m/s Fixture	Sized Correctly 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

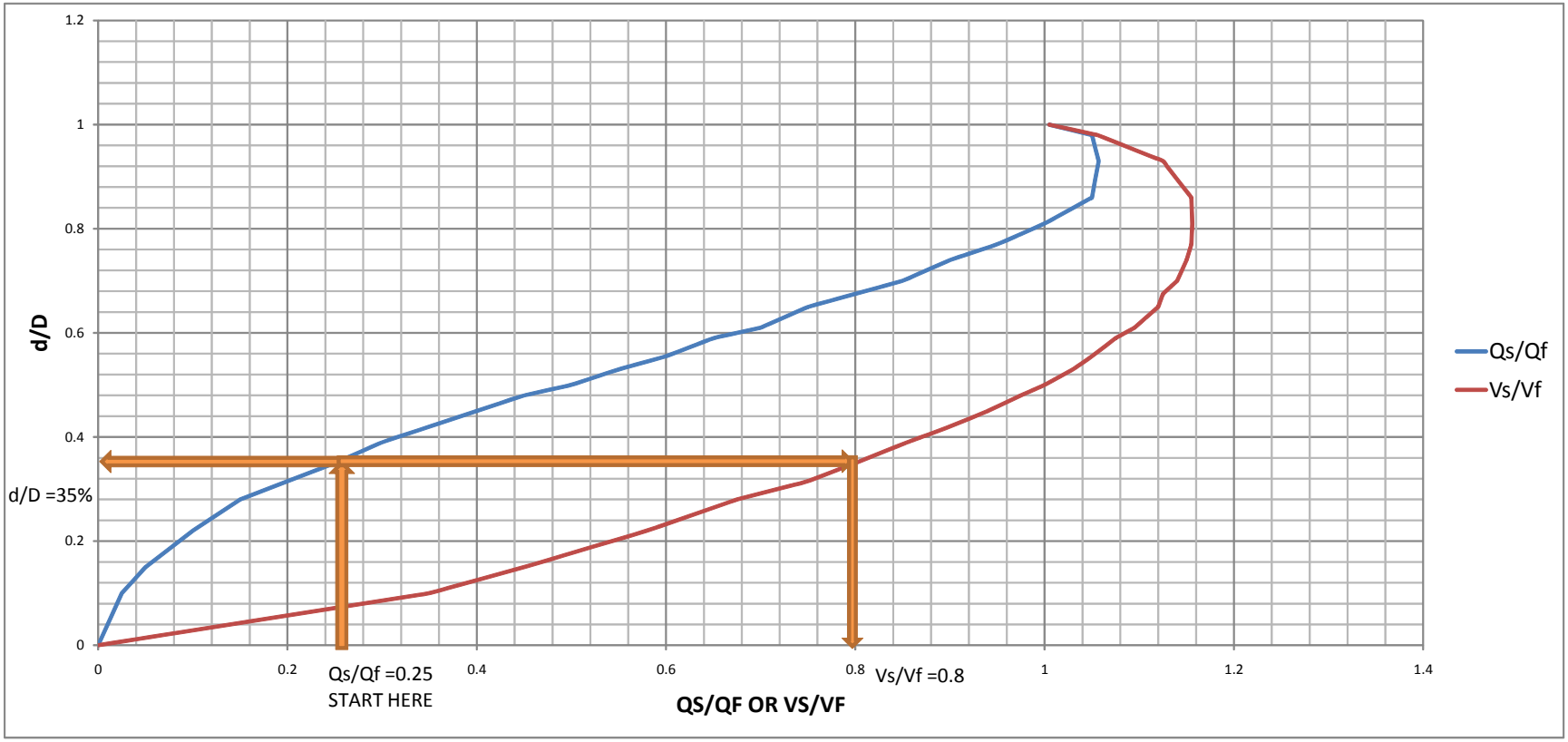
M5-M6 PEAK DIURNAL FLOW IS CALCULATED:
(8.3%*2+21.5%*2+6.0%)*5.6 Lps = 3.68 Lps
USE THE SAME METHOD TO CALCULATE THE ADF:
(8.3%*2+21.5%*2+6.0%)*1.72 Lps = 1.13 Lps

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 Lps	Adf Lps	Slope	n	diameter mm	diameter m	Area (Full) mm^2	Area (Full) m^2	R mm	R m	Vel full,Vf m/s	flow Full, Qf m^3/s	flow Full, Qf Lps	Qs/Qf Diurnal	Qs/Qf ADF	Vs/Vf Diurnal	Vs/Vf ADF		Vs, m/s Diurnal	Vs, m/s ADF	d/D Diurnal	d/D ADD	Sized Correctly Diurnal	Sized Correctly 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

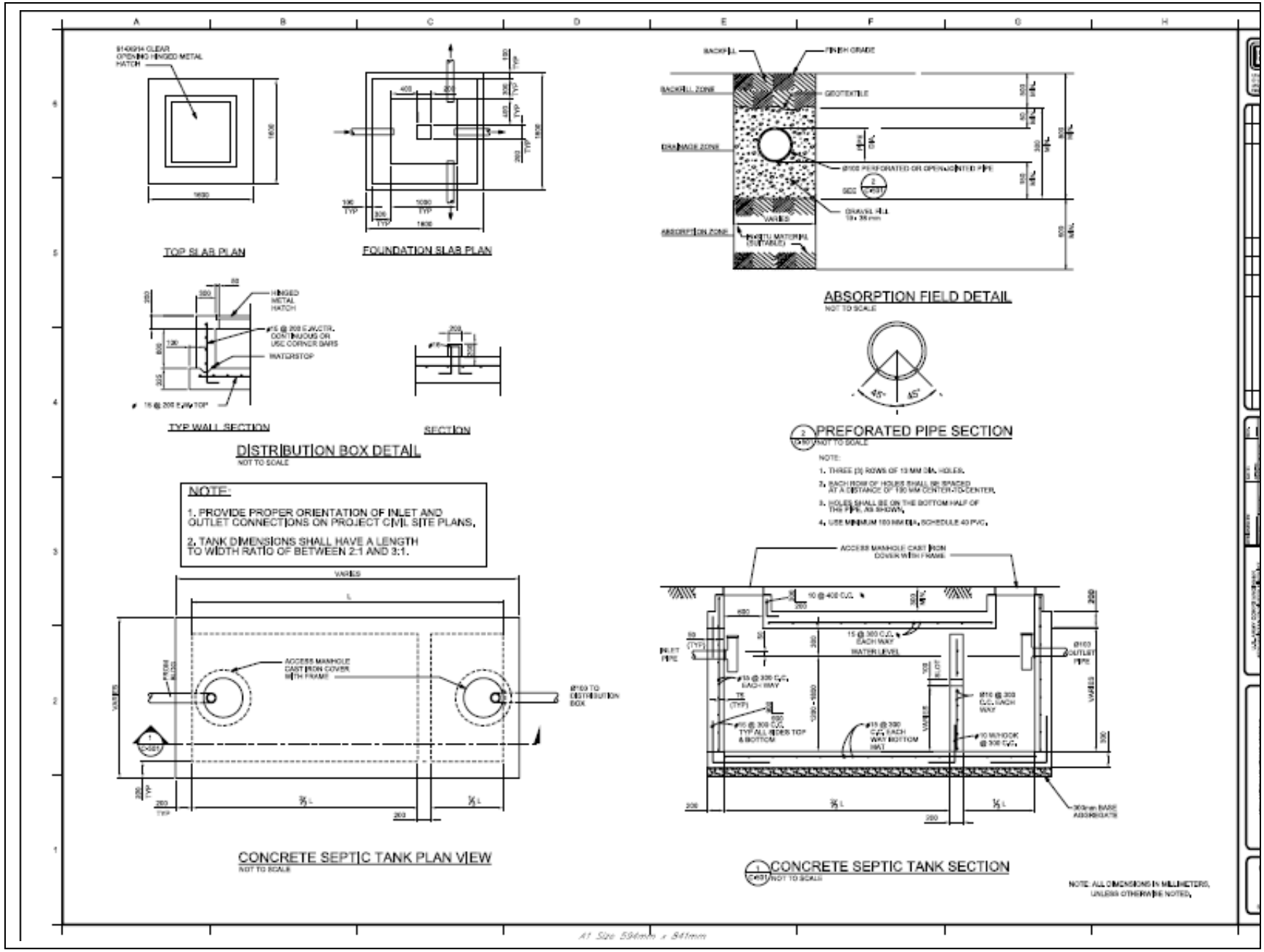
Absorption Bed and Trench Details

Dosing system Layout

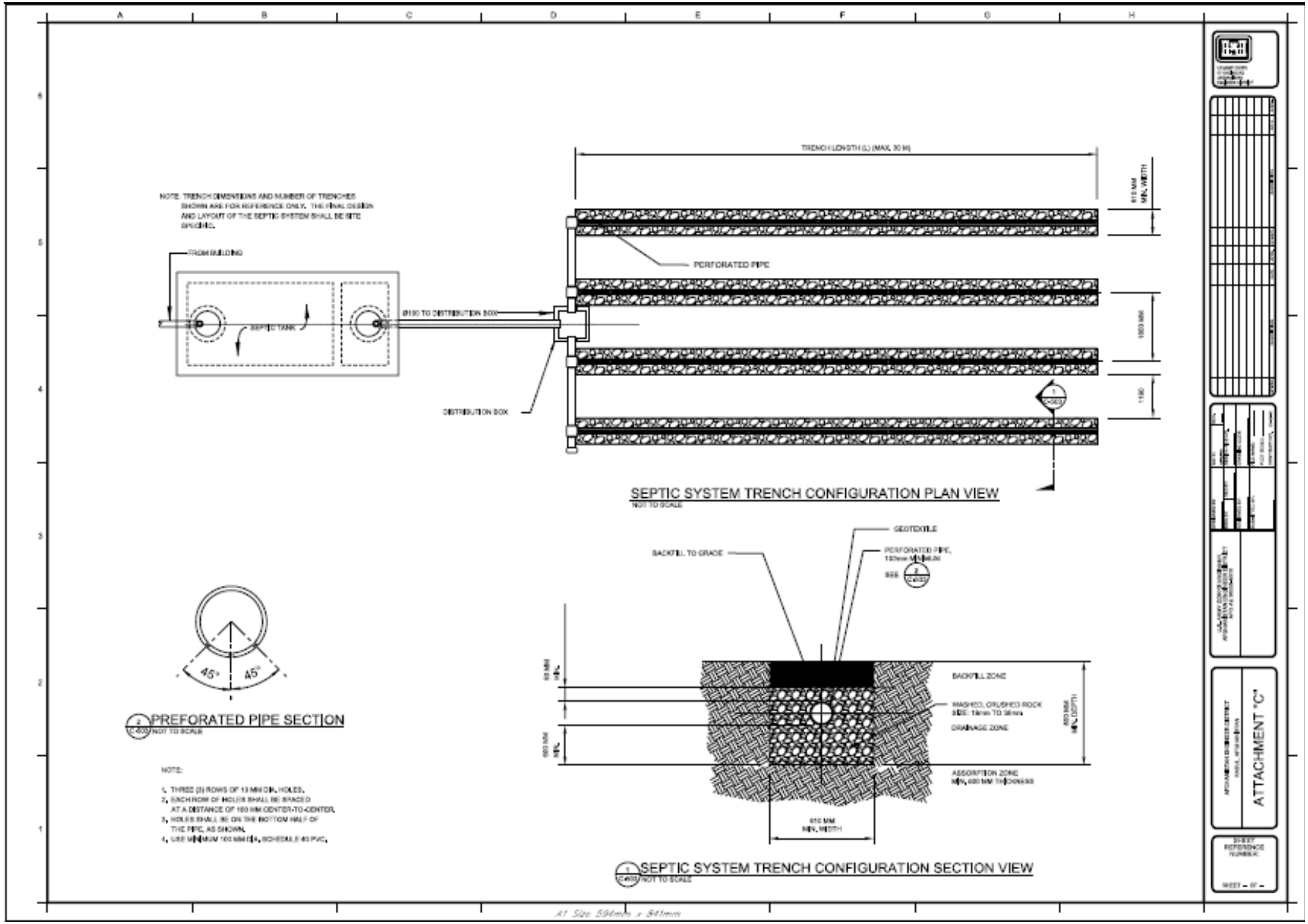
Leaching Chambers Option 1A and 1B (Traffic Loading)

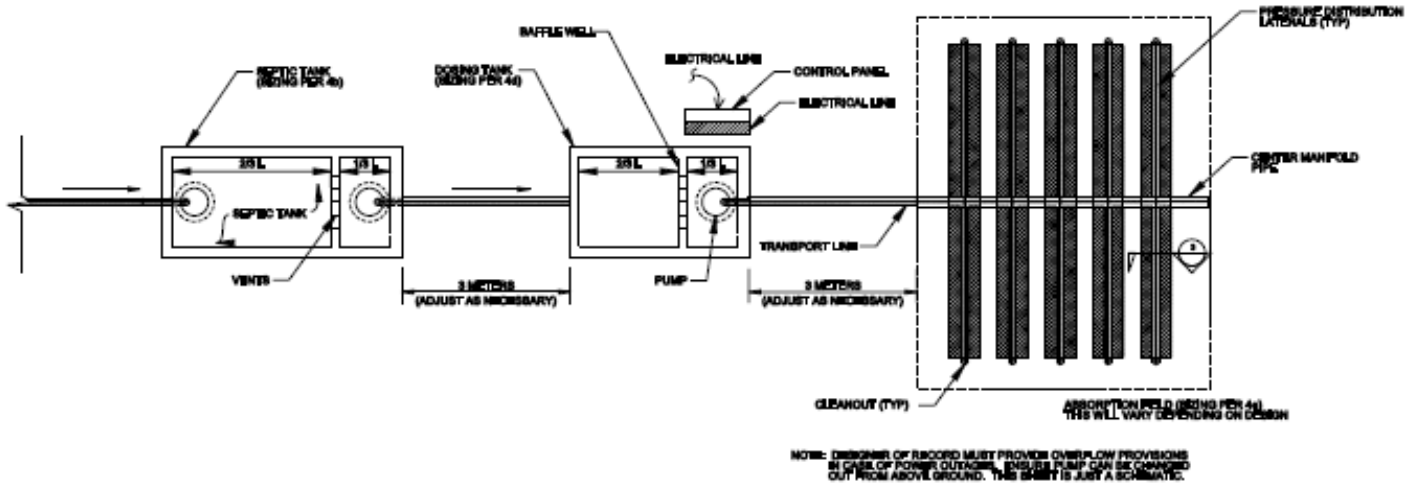
Leaching Chambers Option 2

AED Design Requirements
Sanitary Sewer & Septic Systems



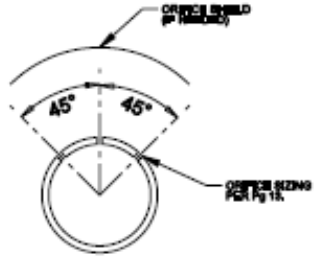
AED Design Requirements
Sanitary Sewer & Septic Systems



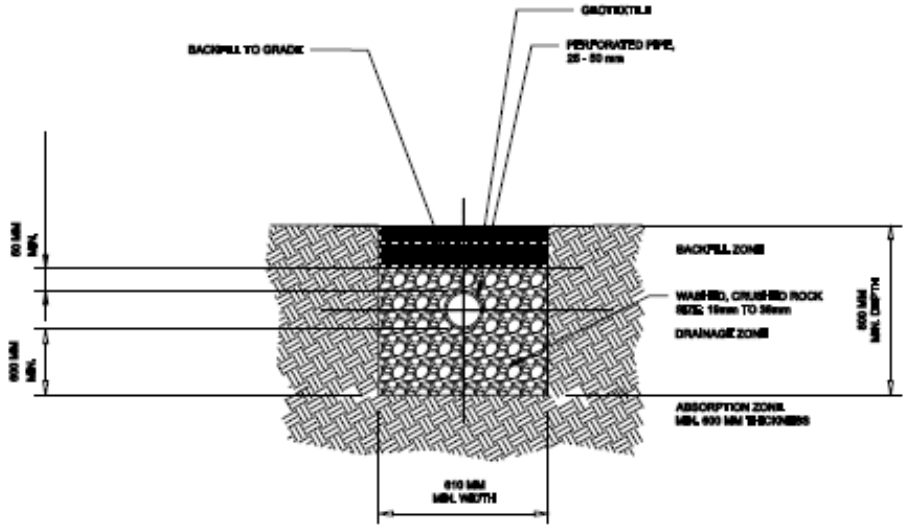


NOTE: DESIGNER OF RECORD MUST PROVIDE OVERFLOW PROVISIONS IN CASE OF POWER OUTAGES. ENSURE PUMP CAN BE CHANGED OUT FROM ABOVE GROUND. THIS SHEET IS JUST A SCHEMATIC.

1 **TYPICAL DOSING TANK (PRESSURE DISTRIBUTION SYSTEM) LAYOUT**
NOT TO SCALE



2 **PRESSURE DISTRIBUTION LATERAL W/ ORIFICE**
NOT TO SCALE

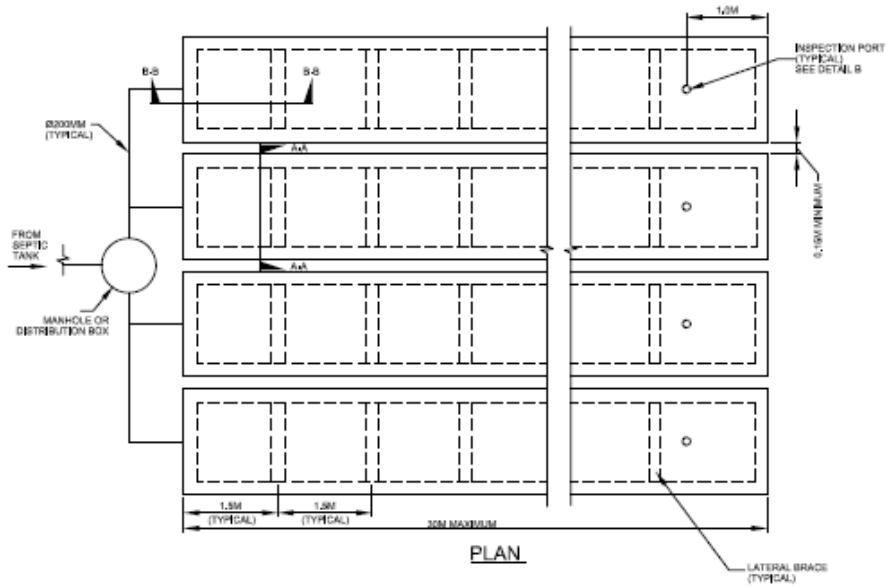


3 **TRENCH CONFIGURATION SECTION VIEW**
NOT TO SCALE

A1 Size 594mm x 841mm

ATTACHMENT "D"

AED Design Requirements
Sanitary Sewer & Septic Systems



PLAN

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 38MM TO 50MM WASHED CRUSH STONE.

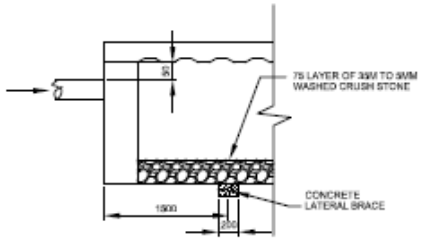
(6) LAY DOWN STEEL DECK, CHAIR REINFORCING IN CENTER OF 100MM SLAB, FORM EDGES, AND POUR SLAB.

(7) BACKFILL SPACE BETWEEN TRENCH WALL AND STONE MASONRY (OR CONCRETE) WALL WITH 38MM TO 50MM WASHED CRUSH STONE. PLACE FILL IN LIFTS NOT TO EXCEED 15 TO 30 CM.

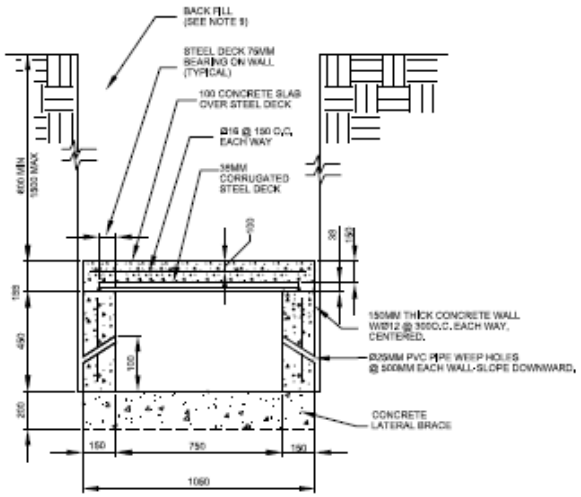
(8) USE A WELL-GRADED SOIL AS BACKFILL. A WELL-GRADED SOIL CONTAINS AN EVEN DISTRIBUTION OF PARTICLE SIZES, RANGING FROM SILT THROUGH SAND TO GRAVEL, WITH A MAXIMUM OF 10 PERCENT FINES (SOIL PASSING #200 SIEVE). COMPACT BACKFILL IN LIFTS NOT EXCEEDING 300MM TO A MINIMUM OF 95 PERCENT OF THE MODIFIED PROCTOR DENSITY. USE A VIBRATORY ROLLER WITH MAXIMUM GROSS VEHICLE WEIGHT OF 9,000 KILOGRAMS AND A MAXIMUM DYNAMIC FORCE OF 8,000 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, 0.28MM THICKNESS. SECTION MODULUS (S) PER METER OF WIDTH = 11.2CM

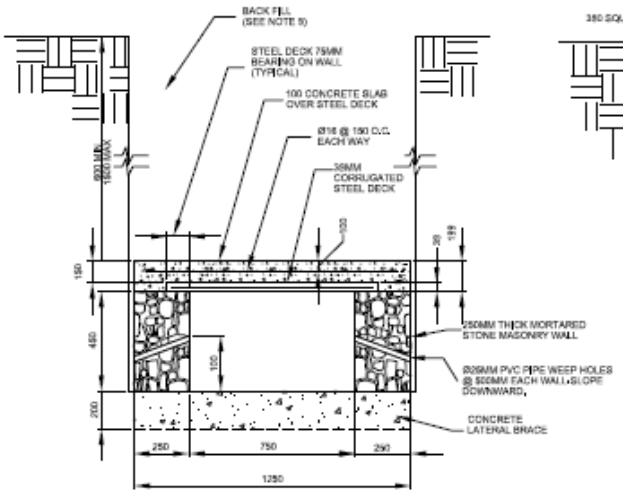
(10) LEACHING CHAMBERS HAVE BEEN DESIGNED FOR A MAXIMUM TRUCK AXLE LOAD OF 12,000 KG.



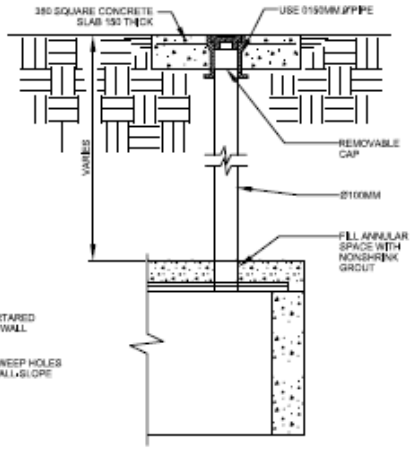
CROSS SECTION B-B



CROSS SECTION A-A
CONCRETE WALL OPTION 1A



CROSS SECTION A-A
STONE WALL OPTION 1B



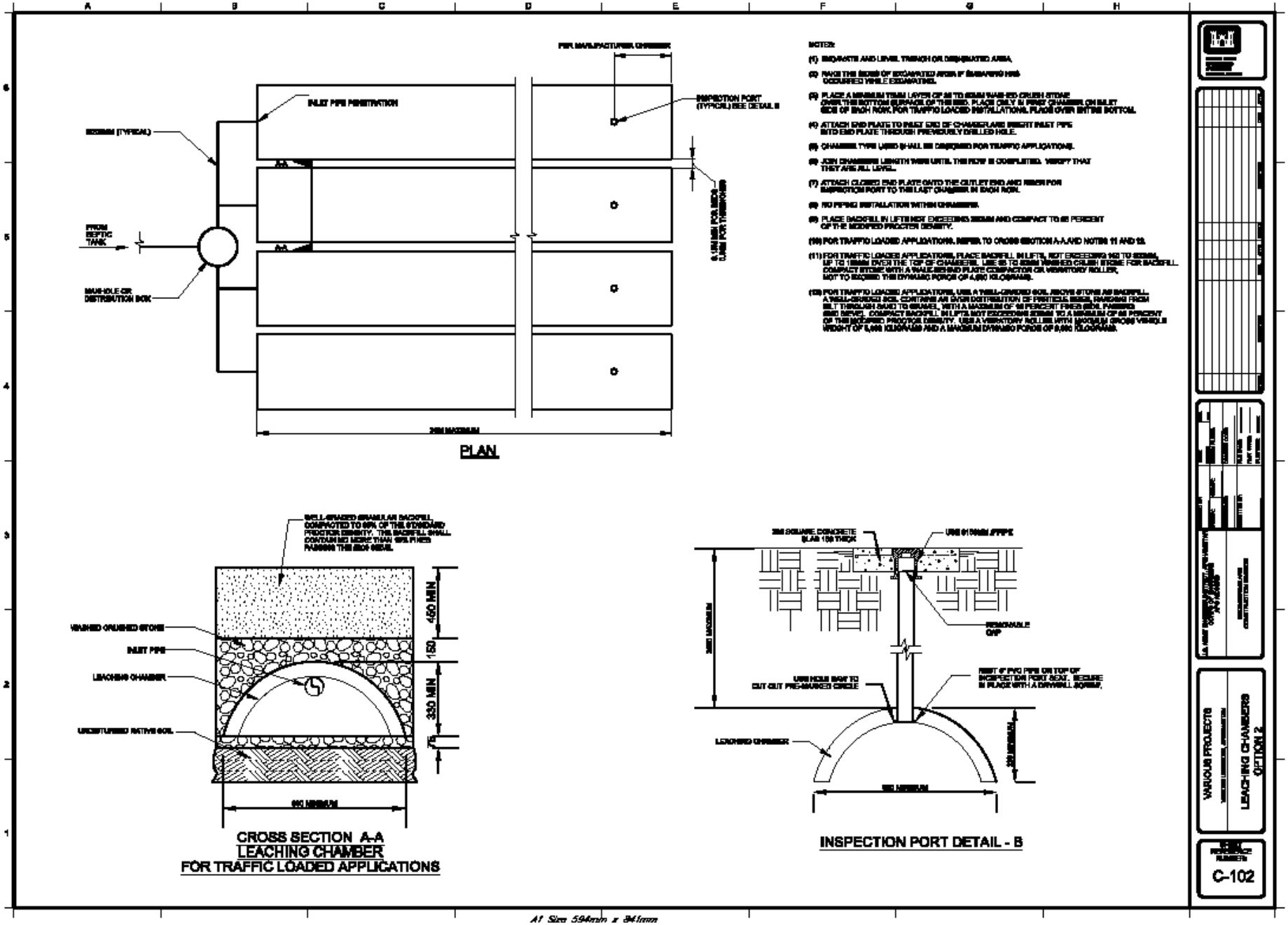
INSPECTION PORT DETAIL - B

VARIOUS PROJECTS
LEACHING CHAMBERS
OPTION 1A AND 1B
(TRAFFIC LOADING)

SHEET REFERENCE NUMBER
C-101

A1 Size 594mm x 841mm

AED Design Requirements
Sanitary Sewer & Septic Systems



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

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AED DESIGN REQUIREMENTS
FOR
SITE LAYOUT GUIDANCE
VARIOUS LOCATIONS,
AFGHANISTAN

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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

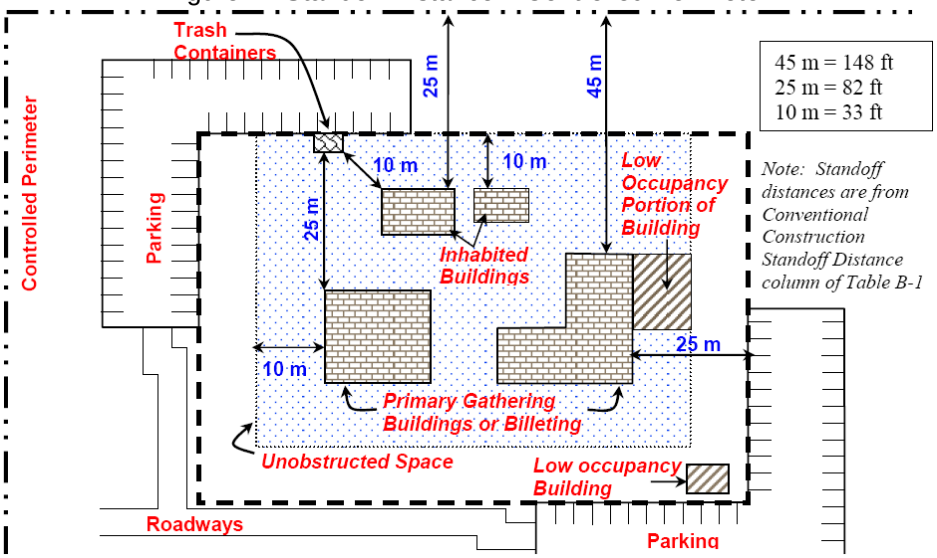


Figure 2. Standoff Distance – No Controlled Perimeter

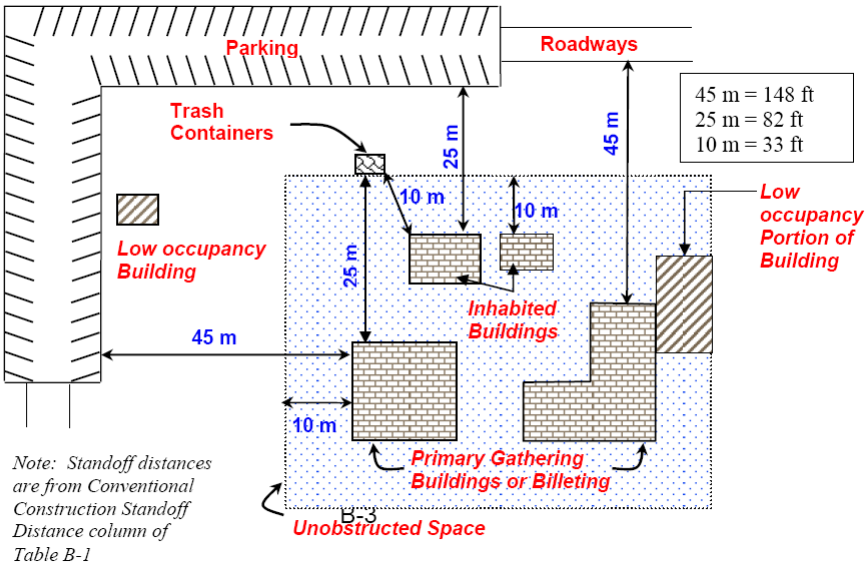


Figure 3. Parking and Roadway Control for Existing Buildings - Controlled Perimeter

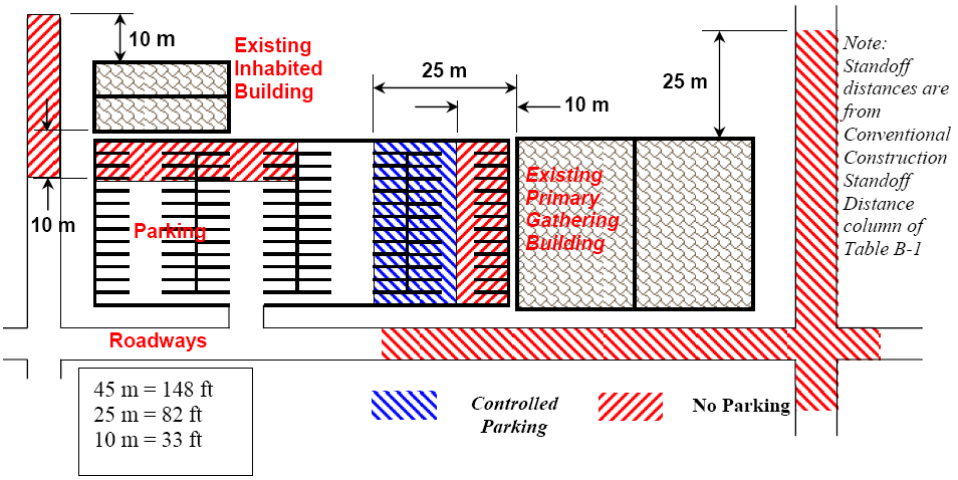
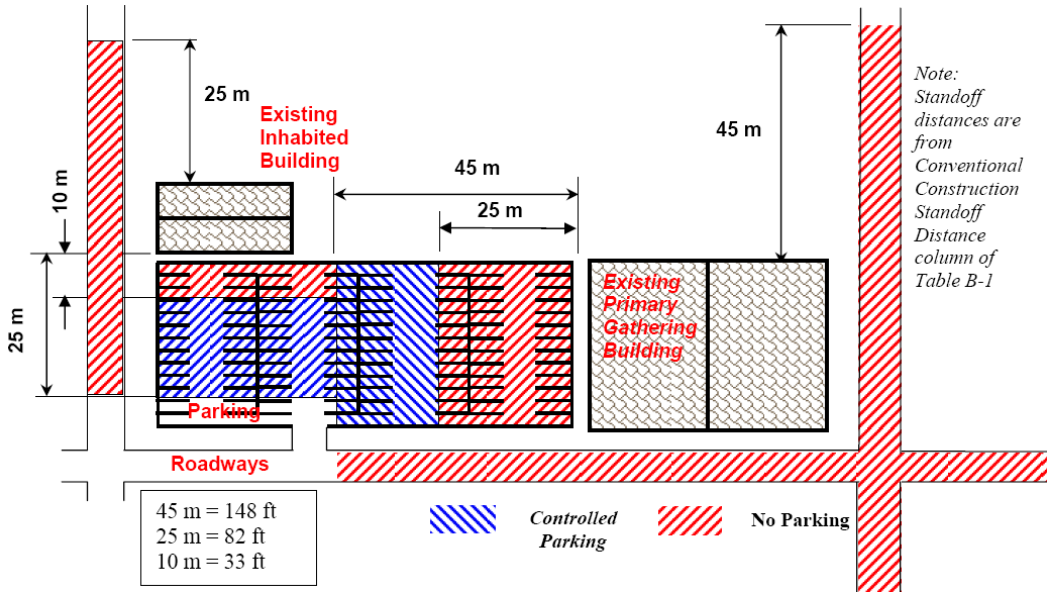


Figure 4. Parking and Roadway Control for Existing Buildings - No Controlled Perimeter



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

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AED DESIGN REQUIREMENTS
FOR
VERTICAL CURVE DESIGN
VARIOUS LOCATIONS,
AFGHANISTAN

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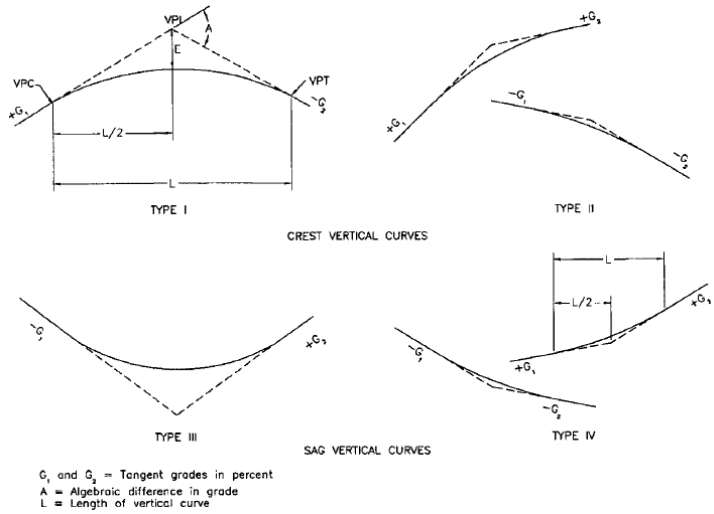
1. General

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)	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:
Voltage Drop Calculation Process

Various Locations,
Afghanistan

MARCH 2009

AED DESIGN REQUIREMENTS
Voltage Drop Calculation

The voltage drop of any insulated cable is dependent upon the length of the cable, the current on the cable and the impedance (ohm) per unit length of the cable.

Voltage drop on the cable shall be limited to the following:

- The voltage drop of the secondary service of 3%.
- The voltage drop of a feeder or branch circuit of 2%.

The combined voltage drop of feeder and branch circuit shall not exceed 5%.

There are two methods of determining the voltage drop of a cable. The difference depends upon whether the cable supplied meets American (U.S) standards or European standards.

If the cable meets European standards, the formula is as follows:

<u>European Formula (mV/A/m)</u>

VD = R x I x L / 1000

VD: The voltage drop (V).
R: The resistance value from voltage drop per ampere per meter table (mV/A/m) supplied by the cable manufacturer.
I : The load current (A)
L: The length of conductor (m)

The value R is determined from a table provided by the cable manufacturer. See Table 1 below as an example of a table that is provided by a manufacturer.

Below is an example calculation for determining voltage drop.

Determine the voltage drop of a 380V, 3 phase circuit with a current of 100A and a length of 150 m and a conductor size of 35 mm. This is a secondary service feed.

The voltage drop of secondary service is limited to 3% (380 x 3% = 11.4 Volts).

Volt drop = Resistance x Current x Length / 1000. Resistance is found on Table 1 below.

VD = 1.1 x 100 x 150 / 1000
= 16.5 V

The maximum voltage drop allowed is 11.4 V. To determine the size of cable that will be required to meet the voltage drop requirement, determine the value of R that will meet the requirement.

11.4 = R x 100 x 150 / 1000
R = 11.4 x 1000/100 x 150
= 0.76 mV/A/m

Referencing the table provided by the cable manufacturer (Table 1), the cable that has a resistance of .76mV/A/m or less is a 70 mm cable with a resistance of 0.55 mV/A/m.

Calculating the voltage drop for the 70mm cable results in:

VD = R x I x L / 1000
= 0.55 x 100 x 150 / 1000
= 8.25 V

The percentage voltage drop is:

Percentage Voltage Drop = 8.25 x 100 / 380 = 2.17 %

Therefore, in order to transmit a 3 phase current of 100A per phase over a length of 150 m, with a total voltage drop equal to or less than the maximum 11.4 volts, a 70 mm cable is needed.

This same procedure would be repeated for a feeder or branch circuit and the results added. The total voltage drop should not exceed 5%.

TABLE 1

VOLTAGE DROP PER AMPERE PER METER (mV). Conductor operating temperature: 70ºC

Conductor Cross Sectional Area	Two Core Cable D.C.	Two Core Cable SIngle Phase A.C.			Three or Four Core Cable Three phase A.C.		
mm	mV		mV			mV	
1.5	29		29			25	
2.5	18		18			15	
4	11		11			905	
6	7.3		7.3			604	
10	4.4		4.4			308	
16	2.8		2.8			204	
		R	**X**	**Z**	**R**	**X**	**Z**
25	1.75	1.75	0.170	1.75	1.50	0.145	1.50
35	1.25	1.25	0.165	1.25	**1.10**	0.145	1.10
50	0.93	0.93	0.165	0.94	0.80	0.140	0.81
70	0.63	0.63	0.160	0.65	**0.55**	0.140	0.57
95	0.46	0.47	0.155	0.50	0.41	0.135	0.43
120	0.36	0.38	0.155	0.41	0.33	0.135	0.35
150	0.29	0.30	0.155	0.34	0.26	0.130	0.29
185	0.23	0.28	0.150	0.29	0.21	0.130	0.25

240	0.180	0.190	0.150	0.24	0.165	0.130	0.21
300	0.145	0.155	0.145	0.21	0.136	0.130	0.185
400	0.105	0.115	0.145	0.185	0.100	0.125	0.160

If the cable meets European standards, the formula is as follows:

U.S Formula (NEC)

For three phase: **VD = 1.732 x L x R x I / 1000**
For single phase: **VD = 2 x L x R x I / 1000**

VD: The voltage drop (V)
L : The length of conductor (m)
R: The resistance value from Chapter 9, Table 8 (ohm/km).
I : The load current (A)

The value R is determined from the National Electrical Code (NEC), Chapter 9, Table 8 column Direct Current Resistance at 75 degrees C/ Copper/ Uncoated. See Table 2 below for the NEC table.

Below is an example calculation for determining voltage drop.

Determine the voltage drop of a 380V, 3 phase circuit with a current of 100A and a length of 150 m and a conductor size of 35 mm. This is a secondary service feed.

VD = 1.732 x Length x Resistance x Current / 1000. Resistance is found in Table 2 below:
= 1.732 x 100 x 0.802 x 100 /1000
= 13.89 V

The maximum voltage drop allowed is 11.4 V. To determine the size of cable that will be required to meet the voltage drop requirement, determine the value of R that will meet the requirement.

11.4 = 1.732 x 150 x R x 100 / 1000
R = 11.4 x 1000/1.732 x 150 x 100
= 0.438 ohm/km

Referencing the NEC table (Table 2) indicates that the cable size with a voltage drop of 0.438 ohm/km or less is 1/0 AWG (70 mm)cable with a resistance of 0.399 ohm/km.

Calculating the voltage drop for the 1/0 AWG (70mm) cable results in:

VD = 1.732 x L x R x I / 1000
= 1.732 x 150 x 0.399 x 100 / 1000

= 10.36 V

The percentage voltage drop is:

Percentage Voltage Drop = 10.36 x 100 / 380 = 2.73 %

Therefore, in order to transmit a 3 phase current of 100A per phase over a length of 150 m, with a total voltage drop equal to or less than the maximum 11.4 volts, a 1/0 AWG (70 mm2) cable is needed.

This same procedure would be repeated for a feeder or branch circuit and the results added. The total voltage drop should not exceed 5%.

TABLE 2

Table 8 Conductor Properties

Size (AWG or kcmil)	Area		Conductors							Direct-Current Resistance at 75°C (167°F)					
			Stranding			Overall				Copper					
				Diameter		Diameter		Area		Uncoated		Coated		Aluminum	
	mm²	Circular mils	Quantity	mm	in.	mm	in.	mm²	in.²	ohm/km	ohm/kFT	ohm/km	ohm/kFT	ohm/km	ohm/kFT
18	0.823	1620	1	—	—	1.02	0.040	0.823	0.001	25.5	7.77	26.5	8.08	42.0	12.8
18	0.823	1620	7	0.39	0.015	1.16	0.046	1.06	0.002	26.1	7.95	27.7	8.45	42.8	13.1
16	1.31	2580	1	—	—	1.29	0.051	1.31	0.002	16.0	4.89	16.7	5.08	26.4	8.05
16	1.31	2580	7	0.49	0.019	1.46	0.058	1.68	0.003	16.4	4.99	17.3	5.29	26.9	8.21
14	2.08	4110	1	—	—	1.63	0.064	2.08	0.003	10.1	3.07	10.4	3.19	16.6	5.06
14	2.08	4110	7	0.62	0.024	1.85	0.073	2.68	0.004	10.3	3.14	10.7	3.26	16.9	5.17
12	3.31	6530	1	—	—	2.05	0.081	3.31	0.005	6.34	1.93	6.57	2.01	10.45	3.18
12	3.31	6530	7	0.78	0.030	2.32	0.092	4.25	0.006	6.50	1.98	6.73	2.05	10.69	3.25
10	5.261	10380	1	—	—	2.588	0.102	5.26	0.008	3.984	1.21	4.148	1.26	6.561	2.00
10	5.261	10380	7	0.98	0.038	2.95	0.116	6.76	0.011	4.070	1.24	4.226	1.29	6.679	2.04
8	8.367	16510	1	—	—	3.264	0.128	8.37	0.013	2.506	0.764	2.579	0.786	4.125	1.26
8	8.367	16510	7	1.23	0.049	3.71	0.146	10.76	0.017	2.551	0.778	2.653	0.809	4.204	1.28
6	13.30	26240	7	1.56	0.061	4.67	0.184	17.09	0.027	1.608	0.491	1.671	0.510	2.652	0.808
4	21.15	41740	7	1.96	0.077	5.89	0.232	27.19	0.042	1.010	0.308	1.053	0.321	1.666	0.508
3	26.67	52620	7	2.20	0.087	6.60	0.260	34.28	0.053	0.802	0.245	0.833	0.254	1.320	0.403
2	33.62	66360	7	2.47	0.097	7.42	0.292	43.23	0.067	0.634	0.194	0.661	0.201	1.045	0.319
1	42.41	83690	19	1.69	0.066	8.43	0.332	55.80	0.087	0.505	0.154	0.524	0.160	0.829	0.253
1/0	53.49	105600	19	1.89	0.074	9.45	0.372	70.41	0.109	0.399	0.122	0.415	0.127	0.660	0.201
2/0	67.43	133100	19	2.13	0.084	10.62	0.418	88.74	0.137	0.3170	0.0967	0.329	0.101	0.523	0.159
3/0	85.01	167800	19	2.39	0.094	11.94	0.470	111.9	0.173	0.2512	0.0766	0.2610	0.0797	0.413	0.126
4/0	107.2	211600	19	2.68	0.106	13.41	0.528	141.1	0.219	0.1996	0.0608	0.2050	0.0626	0.328	0.100

Computer programs can be used to calculate the voltage drop, however the Contractor shall provide a sample hand calculation for a single feeder, branch circuit or secondary service to identify the formula that is being used to calculate voltage drop.

US Army Corps
of Engineers
Afghanistan Engineer District

AED Design Requirements:
Water Tanks & System Distribution

Various Locations,
Afghanistan

SEPTEMBER 2009 Version 1.2

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AED DESIGN REQUIREMENTS
FOR
WATER TANKS
VARIOUS LOCATIONS,
AFGHANISTAN

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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 which shall be used in the distribution system is 100mm (4 inch). The maximum velocity at the fixture unit flow shall be 5 feet per second (1.5 meters per second). The Contractor shall provide a water distribution system described as follows: Pipe diameters used in the network shall be 300mm (12 inch), 250mm (10 inch), 200mm (8 inch), 150mm (6 inch) and 100mm (4 inch), as required to maintain system velocities and pressures at the fixture flow that shall vary between 275 KPa (40 psi) to 275 KPa (75 psi). Pipes for building service connections may be smaller diameter.

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 less than 80mm (3 inch), screw joint, 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.

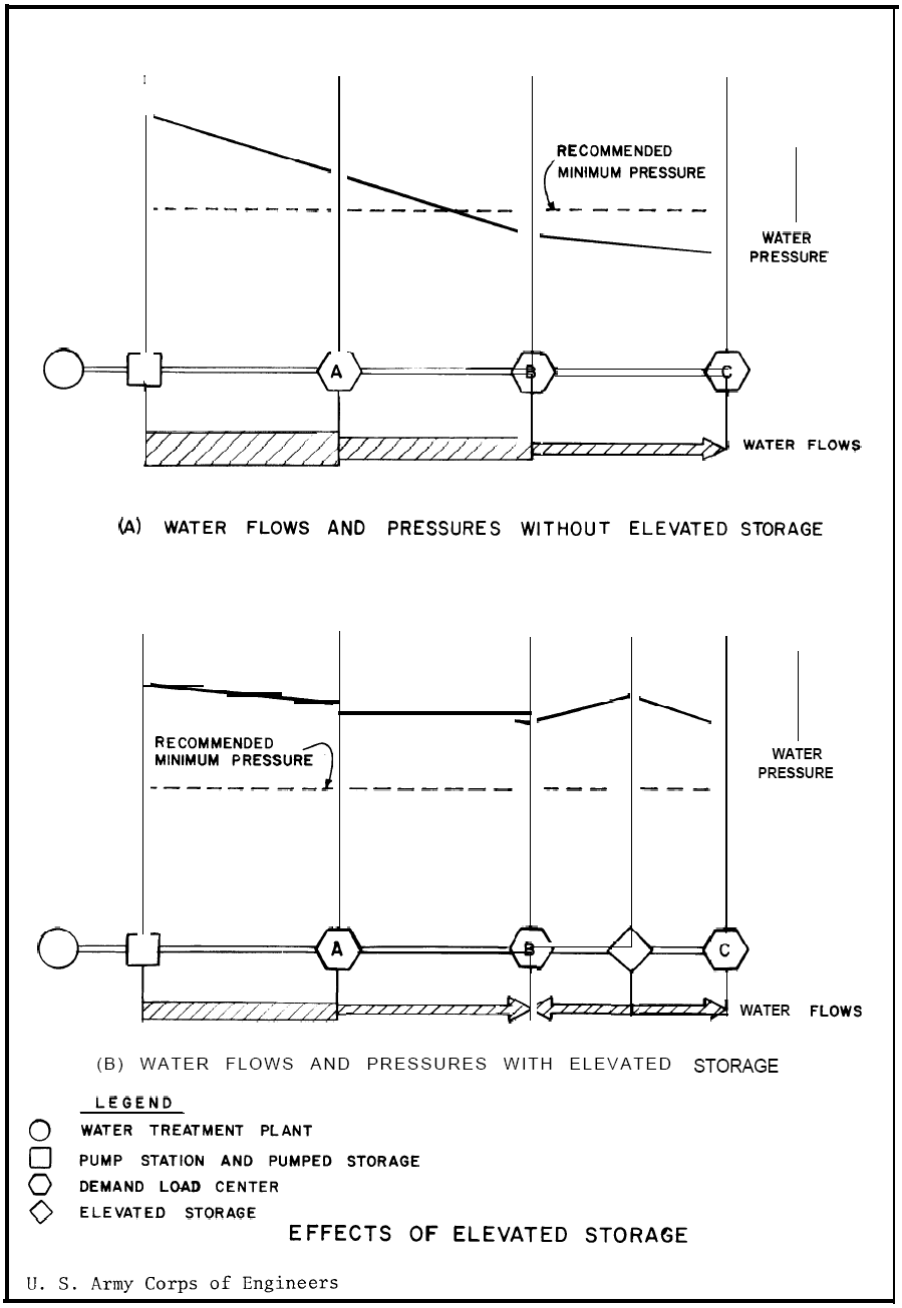
c) Pressure Provisions.

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

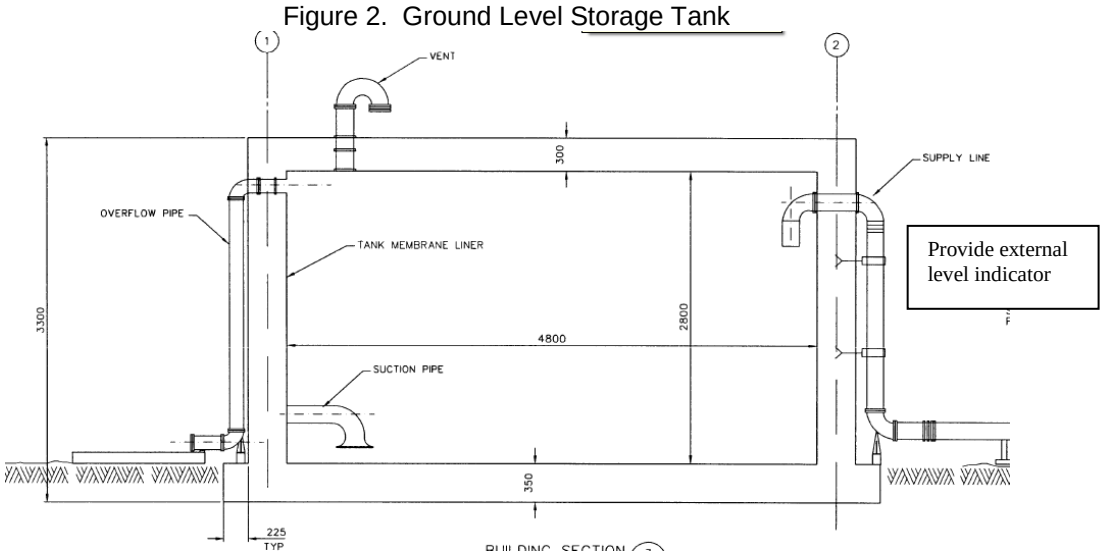
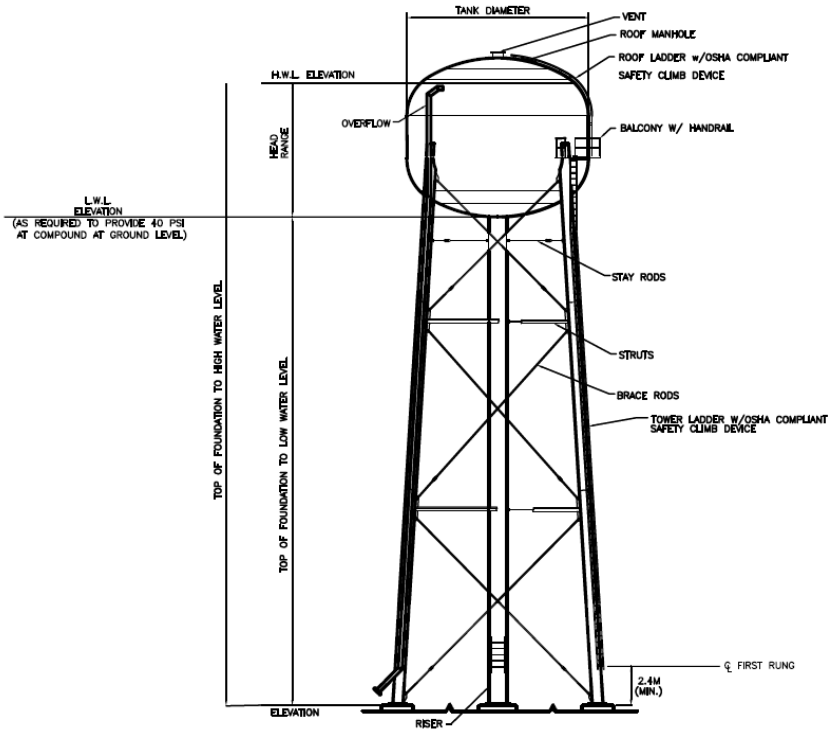


Figure 3. Standard Elevated Storage Tank



ELEVATION
NTS

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, pres-

sure 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 shall be based on the installation wide, total fixture unit flow. For installations with greater than 400 persons, the capacity shall be based on the installation wide, total fixture unit flow or 2 times the average daily flow (16 hour basis), whichever is greater. Three identical pumps shall be provided which are all sized to deliver 50% of the calculated capacity. Pumps shall automatically alternate to distribute wear and shall automatically turn on and off based on demand and system pressures. The total dynamic head (TDH) of the booster pumps shall be calculated to maintain a minimum, residual system pressure of 40 psi at the calculated capacity unless stated otherwise in the contract documents. Either a bladder style expansion tank or a hydropheumatic 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:

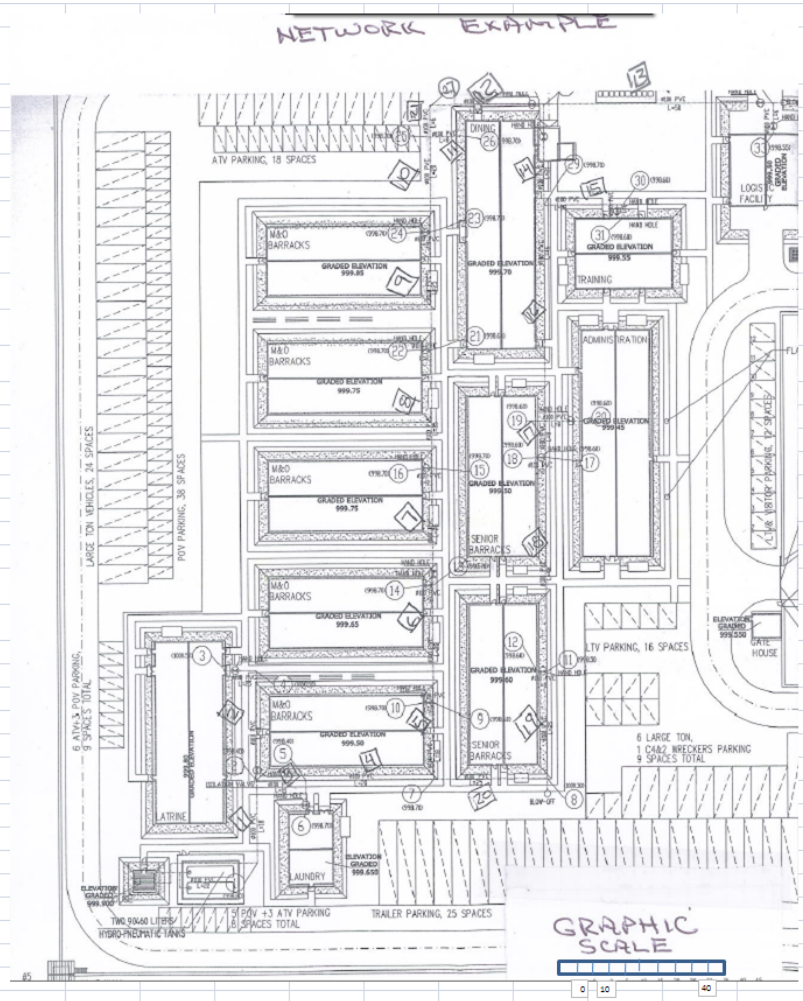
Water Distribution Hydraulic Analysis Example Network Description Table

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:

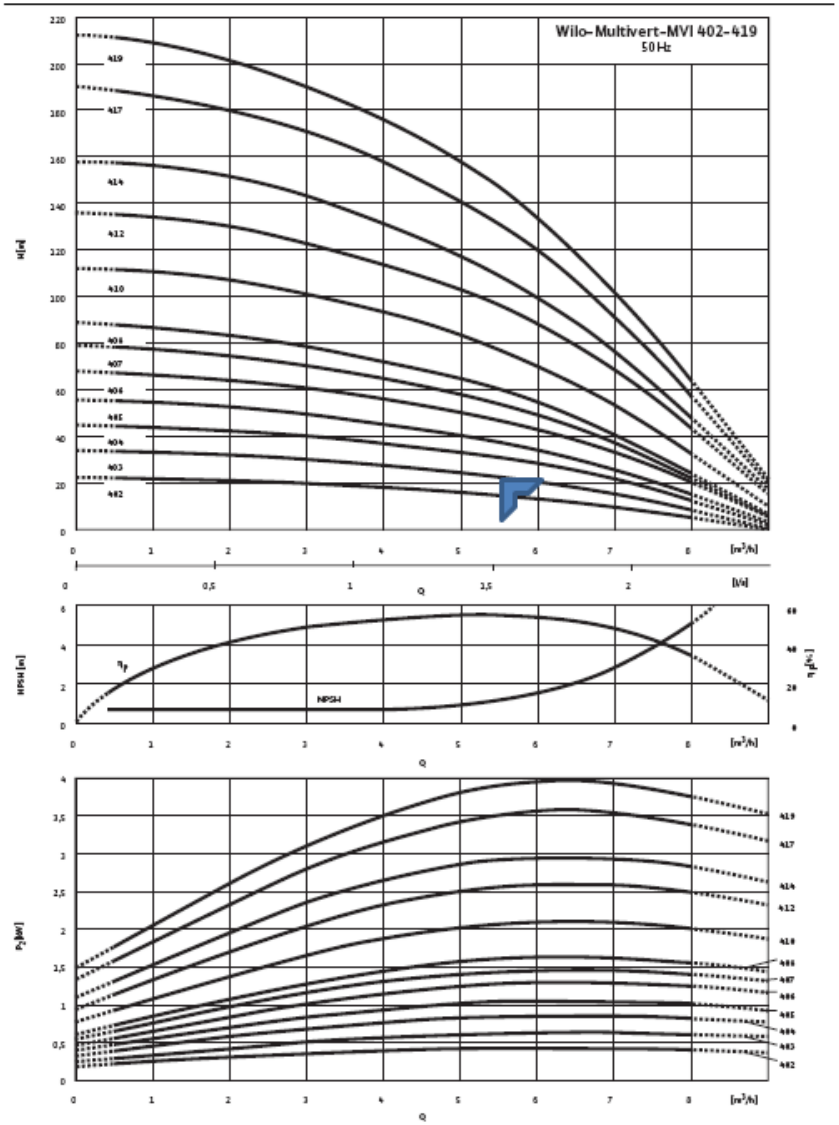
1. Verifying the minimum diameter for the distribution piping is large enough to minimize head loss throughout the system such that the technical criterion for minimum pressure is achieved. This is to be done with the minimum amount of elevated tank or booster pump energy requirements to minimize operation 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:

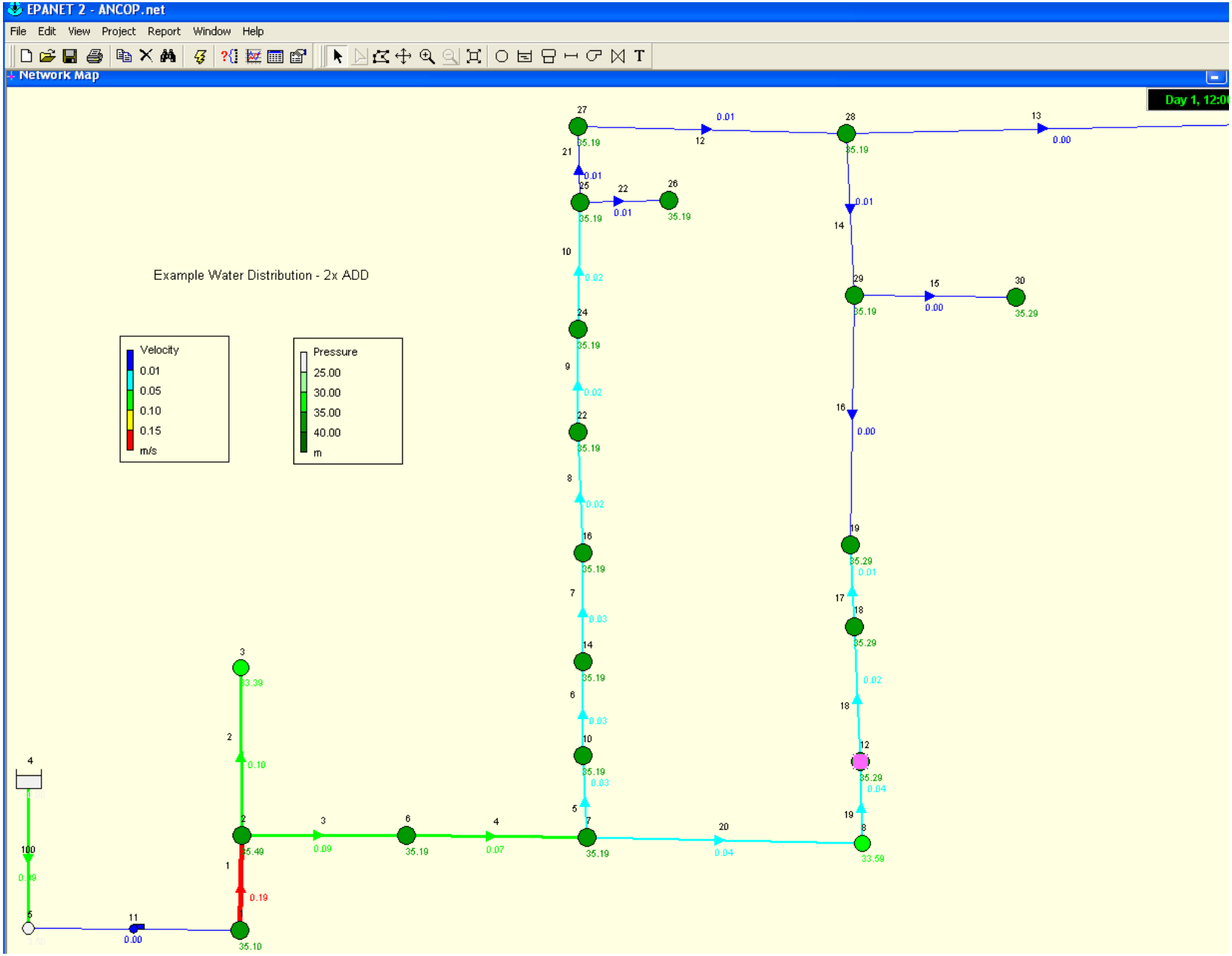
1. Set the design units to SI and defaults and labels
    a. Under project pull down tab edit the following
        i. Hydraulics – set head loss form to Hazen Williams (H-W) and flow units to liter per second (L/S)
        ii. Properties – set pipe length to 25 (meters) and roughness to 145 (conservative 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	_____

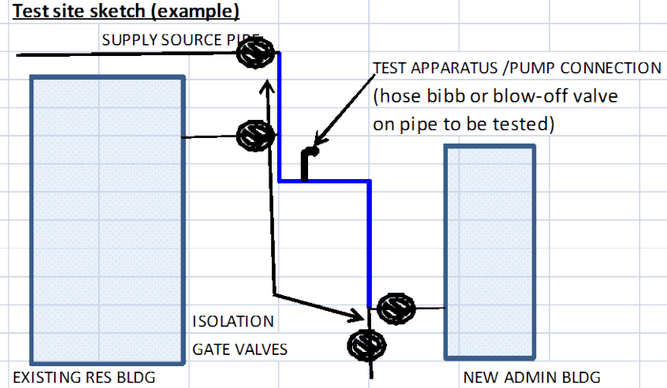
Pressure test data

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		

Test site sketch (example)



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.

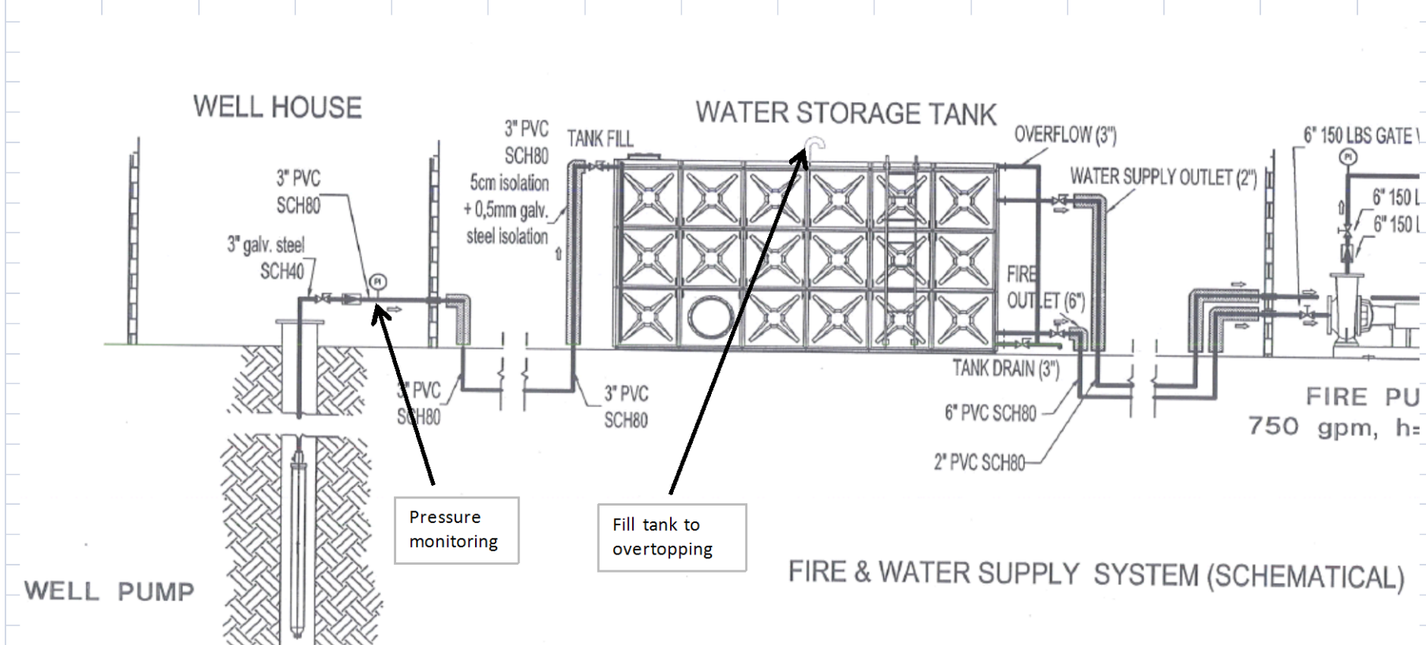
Low Pressure Hydrostatic Test Option

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

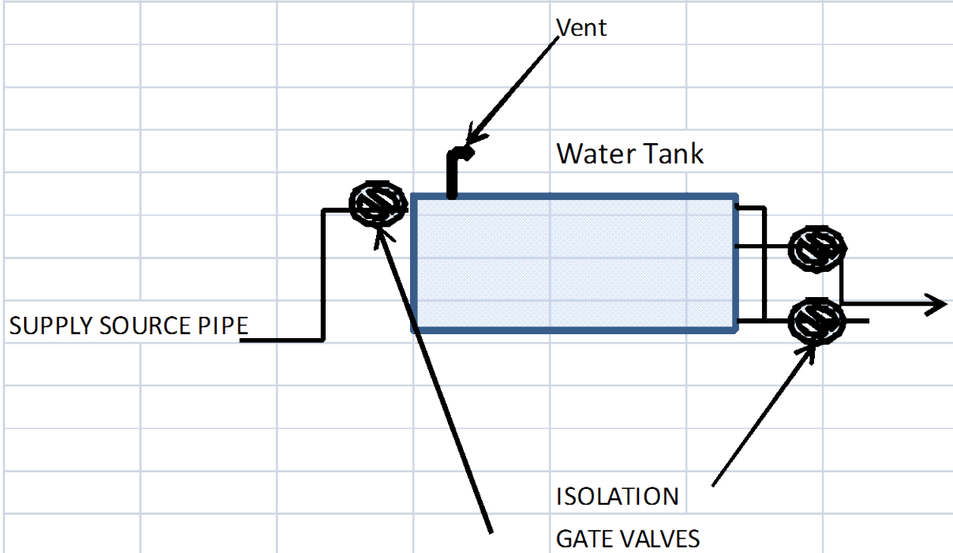
Test Information (shall be printed including names)

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		

US Army Corps
of Engineers
Afghanistan Engineer District

AED Design Requirements:
Well Pumps & Well Design

Various Locations,
Afghanistan

November 2009, Version 1.0

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FOR
WELL PUMPS & WELL DESIGN
VARIOUS LOCATIONS,
AFGHANISTAN

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1. General

The purpose of this document is to provide an overview of the information needed by Contractors to design and construct wells for the Afghanistan Engineering District (AED). The document is organized in the following manner. The main body of the document provides information, concepts and guidelines related to site planning, design processes and construction methods. Appendix A provides details and requirements to be followed when applying the concepts presented in the main body. **Appendix A contains the specific information needed to satisfy the requirements of AED projects containing wells and well construction.** Appendix B is a flow chart which provides an overview of the design and construction process. This chart identifies specific activities required by AED and designates when submittals are required during the well design and construction process. Appendix C provides information on acceptable and unacceptable construction techniques and materials used on AED projects. Photographs are provided to help Contractors understand what AED will and will not accept during a well construction project. Appendix D provides information to Contractors which should be used when sizing and selecting pumps for wells on AED projects.

2. Wells

Ground water is subsurface water occupying the zone of saturation. A water bearing geologic formation which is composed of permeable rock, gravel, sand, earth, etc., and yields water in sufficient quantity to be economical is called an aquifer. Unconfined water is found in aquifers above the first impervious layer of soil or rock. This zone is often referred to as the water table. Water infiltrates by downward percolation through the air-filled pore spaces of the overlying soil material. The water table is subjected to atmospheric and climatic conditions, falling during periods of drought or rising in response to precipitation and infiltration. A confined aquifer is defined as an aquifer underlying an impervious bed. Areas of infiltration and recharge are often some distance away from the point of discharge. This water is often referred to as being under artesian conditions. When a well is installed into an artesian aquifer, the water in the well will rise in response to atmospheric pressure in the well. The level to which water rises above the top of the aquifer represents the confining pressure exerted on the aquifer. Materials with interconnecting pore spaces such as unconsolidated formations of loose sand and gravel may yield large quantities of water and, therefore, are the primary target for location of wells. Dense rocks such as granite, slate, etc. form poor aquifers and wells constructed in them do not yield large quantities of water. However, wells placed in fractured rock formations may yield sufficient water for many purposes and water may be of higher quality.

The design features that the engineer or geologist shall consider for drinking water wells constructed on USACE-AED projects includes the following well features:

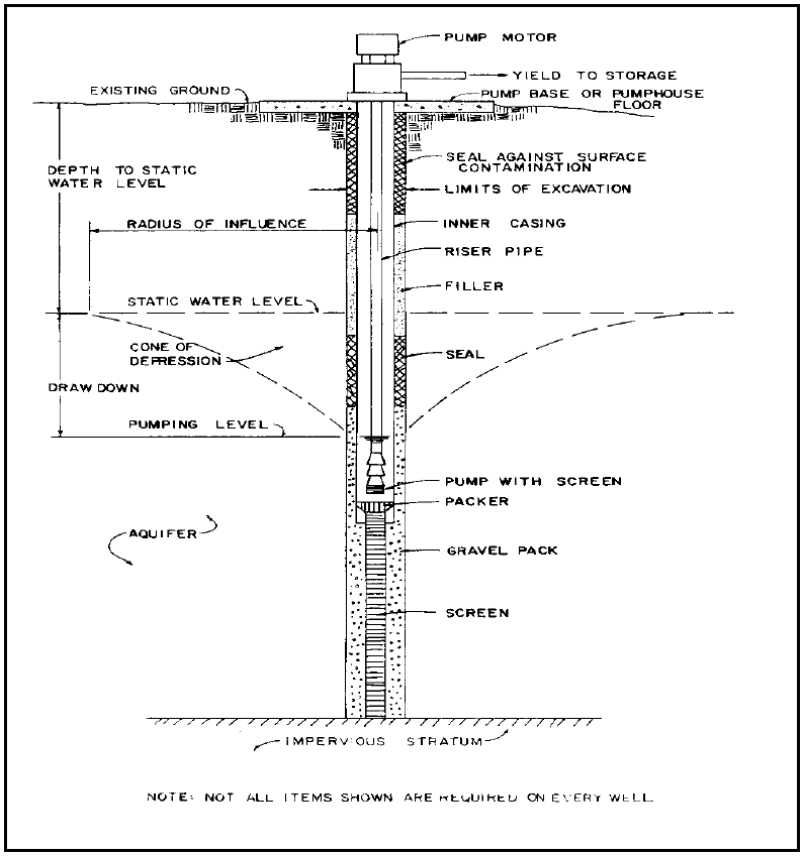
- Well casing diameter
- Casing materials
- Well and casing depth
- Well screen length and diameter
- Well screen slot openings
- Aquifer and well development and gravel pack material
- Collection and analysis of pump test and water quality data
- Selection of well pump

The design and construction documents must describe the determination of these features in as much detail as needed to provide a permanent record of the well construction. Without this documentation, later attempts to evaluate the potential long term yield of a well, well pump problems, water quality, expected / actual yield, and the potential to increase production will be meaningless.

The well section in Figure 1 illustrates typical features of a well, ground water terms, and approximate location of these features. Most wells in USACE-AED projects will contain electric-powered

submersible pumps. However hand-powered pumps may also be specified depending upon the contract technical requirements. Note that the radius of influence caused by pumping is the extent of the cone of depression of the ground water surface around the well. The cone of depression primarily depends on the pumping rate and the aquifer transmissivity. Transmissivity is the rate at which water flows through a vertical strip of aquifer one meter wide and extends through the full saturated thickness under a hydraulic gradient of 1 (100%). Values of transmissivity range from 12 to12,000 m^2/day. If the aquifer transmissivity is 12 m^2/day, a typical 300 mm diameter well, in fine grained material, assumed to have 40 meters of well depth, penetrating the saturated aquifer, with 12 meters of drawdown, producing about 2 liter/second has a radius of influence of approximately 50 meters. This well is adequate for the average daily flow for a 400-man cantonment.

Figure 1. Diagrammatic Section of Gravel-Packed Well



In general, the greater the aquifer transmissivity, the wider the radius of influence of the well. Pumping test data obtained by USACE-AED indicate transmissivity for both fine grained and rock aquifers are at the lower end of the range given above.

3. Types of Wells

Wells are constructed by a variety of methods. There is no single optimum method; the choice depends on the purpose of the well, size, depth, formations being drilled through, experience of local well contractors, and cost. The most common methods of installing wells are compared in Table 1. The performance of different drilling methods in different formations is given in Reference 1. The most common type of construction on USACE-AED projects has been drilling using percussion and rotary drilling equipment.

Shallow hand dug wells or caissons using collector systems may be required at PRT project sites or locations where temporary water supplies are needed. These types of wells are not covered in this design guide. They are briefly covered in Reference 2.

Table 1. Types and Methods of Well Installation

Type	Diameter	Maximum Depth (ft)	Lining or Casing	Suitability	Disadvantages	Method of Construction
Dug	3 to 20 feet	40	wood, masonry, concrete or metal	Water near surface. May be constructed with hand tools.	Large number of manhours required for construction. Hazard to diggers.	Excavation from within well.
Driven	2 to 4 inches	50	pipe	Simple using hand tools.	Formations must be soft and boulder free.	Hammering a pipe into the ground.
Jetted	3 or 4 inches	200	pipe	Small dia. wells on sand.	Only possible in loose sand formations.	High pressure water pumped through drill pipe.
Bored	up to 36 inches	50	pipe	Useful in clay formations.	Difficult on loose sand or cobbles.	Rotating earth auger bracket.
Collector	15 feet	130	Reinforced concrete caisson	Used adjacent to surface recharge source such as river, lake or ocean.	Limited number of Installation Contractors	Caisson is sunk into aquifer. Preformed radial pipes are jacked horizontally through ports near bottom.
Drilled	Up to 60 inches	4000	pipe	Suitable for variety of formations.	Requires experienced Contractor & specialized tools.	a. Hydraulic rotary* b. Cable tool percussion* c. reverse circulation rotary d. hydraulic–percussion e. air rotary

4. Water Quality Evaluation

Both well location and method of construction are of major importance in protecting the quality of water derived from a well. Groundwater may become contaminated as a result of leakage from sources as diverse as improperly sealed wells, septic tanks, garbage dumps, industrial and animal wastes, and setback requirements shall be observed for well site selection. In addition, it is important that the well screen, gravel pack, and well pump be carefully designed as a system to insure that wells constructed in fine grained aquifers do not pump silt and fine sand. Pumping sand and silt will limit usefulness of the water and create problems for both the well and the user.

a) Selection of a Well Site. Prior to selecting the well location, a thorough survey of the area should be undertaken. The following information should be obtained and considered:

- Local hydro geology such as terrain, soil type, depth, and thickness of water bearing zone.
- Location of nearby wells, both drilled and hand dug that may limit the well yield or be impacted by the new well
- Location, construction, and disposal practices of nearby sewage and industrial facilities.
- Locations of sewers, septic tanks, cesspools, animal farms, pastures, and feed lots.
- Chemical and bacteriological quality of ground water, especially the quality of water from nearby wells.
- Histories of water, oil, and gas well exploration and development in area.
- Location and operating practices of nearby industrial and municipal landfills and dumps.

- Direction and rate of travel of ground water if studies have been conducted (see Reference 7 as an example for the Kabul Groundwater Basin).

Recommended minimum distances for well sites from commonly encountered potential sources of pollution are shown in Table 2. It is emphasized that these are minimum distances which can serve as rough guides for locating a well from a potential source of groundwater contamination. The distance may be greater, depending on the geology of the area. In general, very fine sand and silt filter contaminants in ground water better than limestone, fractured rock, coarse sand and gravel. Chemical contaminants may persist indefinitely in untreated groundwater. If at all possible, a well should be located up gradient of any known nearby or potential sources of contamination. It is a good practice to consult local authorities for aid in establishing safe distances consistent with the subsurface geology of the area. Dry wells should be abandoned and plugged in conformance to local regulations.

.

Table 2. Minimum Distances from Pollution Sources.

Source	Minimum Horizontal Distance
Building Sewer	15 m (50 ft)
Disposal Field / Septic Tank	30 m (100 ft)
Seepage Pit	30 m (100 ft)
Dry Well , Abandoned Well	15 m (50 ft)
Cesspool / Leaching Pits	45 m (150 ft)

Note: the above minimum horizontal distances apply to wells at all depths. Greater distances are recommended when feasible.

Well site planning should also consider the proximity to existing wells both on the project site and in the local community. Equation 3, shown in Section 5, Well Hydraulics, can be used with aquifer pump test data from existing wells to estimate the radius of influence. Reference 8 provides experience for the radius of influence for shallow dug wells. In some regions, depending upon the aquifer and recharge, only one well producing 1.5 L/s on a daily basis may be sustainable in one square kilometer.

The grouping of wells must be carefully considered because of mutual interference between wells when their cones of depression overlap. Minimum well spacing shall be 75 m (250 ft).

The drawdown at a well or any other location on the water table is a function of the following:

- Number of wells being pumped.
- Distance from point of measurement to pumping wells.
- Volume of discharge at each well.
- Penetration of each well into aquifer.

For simple systems of 2 or 3 wells, the method of super position may be used. The procedure is to calculate the drawdown at the point (well) of consideration and then to add the drawdown for each well in the field. For multiple wells, the discharge must be recalculated for each combination of wells, since multiple wells have the effect of changing the depth of water. For large systems the following conditions should be noted:

- Boundary conditions may change.
- Change in recharge could occur.
- Recharge may change water temperature, an increase in water temperature increases coefficient of permeability.
- Computer analysis may be helpful to recalculate the combinations.

It is seldom practicable to eliminate interference entirely because of pipeline and other costs, but it can be reduced to manageable proportions by careful well field design. When an aquifer is recharged in roughly equal amounts from all directions, the cone of depression is nearly symmetrical about the well and is about the same in all directions. If, however, substantially more recharge is obtained from one direction; e.g., a stream, then the surface elevation of the water table is distorted, being considerable higher in the direction of the stream. The surface of the cone of depression will be depressed in the direction of an impermeable boundary because little or no recharge is obtained from the direction of the impermeable boundary.

Where a source of recharge such as a stream, exists near the proposed well field, the best location for the wells is spaced out along a line as close as practicable to and roughly parallel to the stream. On the other hand, multiple water supply wells should be located parallel to and as far as possible from an impermeable boundary. Where the field is located over a valley, the wells should be located along and as close to the valley's center as possible. In hard rock country, wells are best located along fault zones and lineaments in the landscape where recharge is greatest. These are often visible using aerial photographs. Special care should be exercised to avoid contamination in these terrains since natural filtration is limited.

b) Sampling and Analysis. It is mandatory to review water quality requirements contained in the U.S. Department of Defense drinking water standards (see reference 9). Wells supplying water to US Forces must be sampled and analyzed for all of the chemical constituents named in the drinking water standards. Analytical requirements of reference 9 may be supplemented and expanded by contract section 01015, Technical Requirements, to include the more comprehensive sampling requirements given in TM 5-813-3/AFM 88-10, Vol. 3. Major contaminants of concern, heavy metals and arsenic, are rarely encountered in significant concentrations in natural ground waters. There are, however, high concentrations of boron reported in some wells in the Kabul province. Wells supplying water to Afghanistan installations must to meet the World Health drinking water standards (see reference 10).

5. Well Hydraulics

The following definitions are necessary to an understanding of well hydraulics:

- Static Water Level. The distance from the ground surface to the water level in a well when no water is being pumped.

- Pumping Level. The distance from the ground surface to the water level in a well when water is being pumped. Also called dynamic water level.

- Drawdown. The difference between static water level and pumping water level.

- Cone of Depression. The funnel shape of the water surface or piezometric level which is formed as water is withdrawn from the well.

- Radius of Influence. The distance from the well to the edge of the cone of depression.

- Permeability. The ease of which water moves through the rock or sediment.

- Hydraulic Conductivity. Also called coefficient of permeability. The rate at which water moves

through the formation (gallons per day per square foot. It is governed by the size and shape of the pore spaces.

The well discharge equation (Eq 1) below is used to determine that amount of water that can be expected from a well. The formula assumes certain simplifying conditions. However, these assumptions do not severely limit the use of the formulas. The assumptions are as follows: 1) the aquifer is of constant thickness, is not stratified and is of uniform permeability; 2) the piezometric surface is level, laminar flow exists and the cone of depression has reached equilibrium; and 3) the pumping well reaches the bottom of the aquifer and is 100 percent efficient. The following equation is used to calculate the discharge from a water table well.

$$Q=(K(H^2-h^2))/(1055 \log (R/r)) \qquad \text{Eq. 3}$$

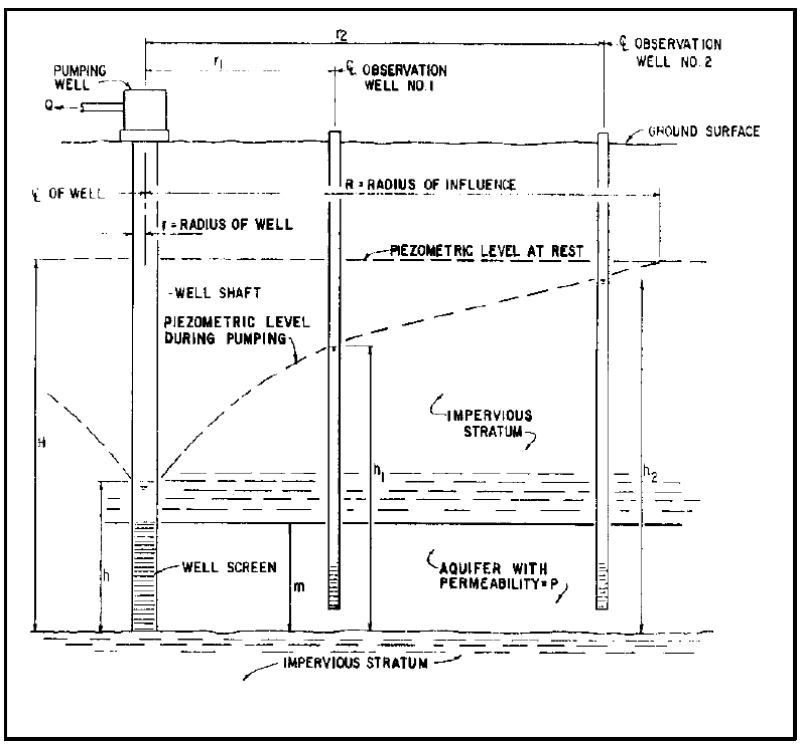
Where:

Q=pumping rate (m^3/day)
K=hydraulic conductivity of water bearing unit (m/day)
H=static head from bottom of aquifer (m)
h=pumping head from bottom of aquifer (m)
R=radius of influence (m)
r=radius of well (m)

Hydraulic conductivity is equal to the aquifer transmissivity dived by the saturated aquifer depth.

Figure 2 shows the relationship of the terms used in Equation 1 for available yield from a water table well. An existing well or monitoring well must be used to estimate the radius of influence of the proposed production well.

Figure 2. Diagram of Water Table Well

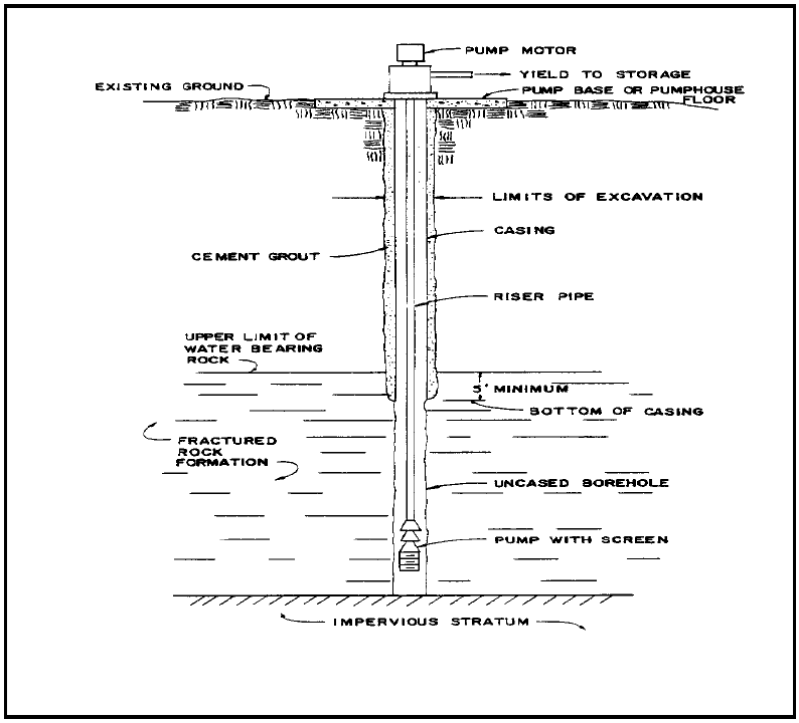


6. Well Design and Construction

The general sequence which should be followed in AED projects is to choose a well site, drill or dig a test hole, test the hydraulic capacity of the well, receive AED concurrence, install permanent well structural elements, develop the well, perform performance tests, install the permanent well pump, and finish the well. See Appendix B for further details regarding this sequence. Test wells and permanent wells should be at least 20 meters below the static water table. The pump, at actual capacity, should have a minimum of 2 meters of submergence at the lowest drawdown depth reached during the pump tests described later in this guide. Well screens shall have a minimum of 2 meters submergence at the lowest drawdown depth occurring during well testing. Permanent wells shall not operate with any portion of the well screen above the lowest drawdown level.

Well design methods and construction techniques are basically the same for water wells constructed in consolidated or unconsolidated formations and only one aquifer is being penetrated. Typically, wells constructed in an unconsolidated formation require a screen to line the lower portion of the borehole. An artificial gravel pack may or may not be required. A diagrammatic section of a gravel packed well is shown in Figure 1. Wells constructed in sandstone, limestone or other creviced rock formations can often utilize an uncased borehole in the aquifer, and do not normally require screens or gravel pack. A well in rock formation is shown in Figure 3. Additional well designs for consolidated and unconsolidated formations are shown in reference 11.

Figure 3. Well in Rock Formation



a) Well casing diameter. The inside diameter for well casings shall selected based on information provided in Table 3. The minimum diameter for any well casing shall be 150mm (6 in). Larger diameters shall be installed based on required well yield and the potential for future expansion. The diameter of a well has a significant effect on the well's construction cost. The diameter shall be uniform from top to bottom. In rare circumstances, construction may be initiated with a certain

diameter casing, but drilling conditions may make it desirable to reduce the casing size at some depth. However, the diameter must be large enough to accommodate the pump. In addition, the diameter of the intake section must be sufficient to assure that the upward velocity of the flow in the pump discharge pipe is 1.5 m/sec or less. The well shall be designed to be straight and plumb. Other factors that control diameter are (1) yield of the well, 2) screen intake entrance velocity, and (3) construction method. The pump size, which is related to yield, usually dominates. Approximate well diameters for various yields are shown in Table 3. Well diameter affects well yield but not to a major degree. Doubling the diameter of the well diameter will produce only about 10 to15 percent more water. Table 4 gives the theoretical changes in yield that result from changing from one well diameter to a new well diameter. For artesian wells, the yield increase resulting from diameter doubling is generally less than 10 percent. Consideration should be given to future expansion and installation of a larger pump. This may be likely in cases where the capacity of the aquifer material (such as coarse grained gravels and conglomerate material) is greater than well yield required for the current project

Table 3. Well Diameter vs. Anticipated Yield
In SI and U.S. Customary Units

Anticipated Well Yeild	Nominal Size of Pump Bowls	Optimum Size of Well Casing	Smallest Size Well Casing
(lpm)	(mm)	(mm)	(mm)
<380	100	150 ID	125 ID
285-660	125	200 ID	150 ID
570-1515	150	250 ID	200 ID
1325-2460	200	300 ID	250 ID
2270-3400	250	350 OD	300 ID
3200-4900	300	400 OD	350 OD
4550-6800	350	500 OD	400 OD
6050-11400	400	600 OD	500 OD
(gpm)	(in)	(in)	(in)
<100	4	6 ID	5 ID
75-175	5	8 ID	6 ID
150-400	6	10 ID	8 ID
350-650	8	12 ID	10 ID
600-900	10	14 OD	12 ID
850-1300	12	16 OD	14 OD
1200-1800	14	20 OD	16 OD
1600-3000	16	24 OD	20 OD
3000-6000	20	30 OD	24 OD

Note: If provided, contract section 01015 technical requirements or the guide specifications (Attachment A) shall supersede the minimum diameters suggested in this table

Table 4. Change in Yield for Variation in Well Diameter

Original Well Diameter	New Well Diameter						
	150 mm (6")	300 mm (12")	450 mm (18")	600 mm (24")	750 mm (30")	900 mm (36")	1200 mm (48")
150 mm (6")	100%	110%	117%	122%	127%	131%	137%
300 mm (12")	90	100	106	111	116	119	125
450 mm (18")	84	93	100	104	108	112	117
600 mm (24")	79	88	95	100	104	107	112
750 mm (30")	76	85	91	96	100	103	108
900 mm (36")	73	82	88	92	96	100	105
1200 mm (48")	69	77	82	87	91	94	100

Note: The above gives the theoretical increase or decrease in yield that result from changing the original well diameter to the new well diameter. For example, if a 300 mm well is enlarged to a 900mm well, the yield will be increased by 19 percent.

The values in the above table are valid only for wells in unconfined aquifers (water table wells) and are based on the following equation:

$$Y_2/Y_1=(\log R/r_1)/(\log R/r_2) \qquad \text{Eq 2}$$

Where:

Y_2=yield of new well
Y_1=yield of original well
R=Radius of cone of depression (mm)
r_2=diameter of new well (mm)
r_1=diameter of original well (mm)

b) Well and casing depth. Depth of a well is usually determined from the logs of test holes or from logs of other nearby wells that utilize the same aquifer. However for contract purposes, a minimum depth is usually specified in either the contract 01015 (Technical Requirements) or in the water well guide specification. A guide specification is provided in this guide as (Appendix A). A well that is screened the full length of the water bearing stratum has a potential for greater discharge than a unit that is not fully screened. Where the water bearing formations are thick, cost may be the deciding factor in how deep the wells are installed. Cost, however, is normally balanced by the savings from a potentially long-term source of water. Well casing should not be founded on bedrock, since the weight of the casing and any other loads transferred to the casing from the construction features may exceed the buckling strength of the casing. The wall friction of the casing after sealing, grouting and well gravel packing should be designed to bear the vertical load on the casing.

c) Casing material. The preferred casing material is steel (ASTM A53 Grade B or ASTM A139 Grade B). Use of PVC must be approved by AED prior to installation. The PVC pipe must be at least Schedule 80 or SDR 17. PVC shall NOT be used for wells deeper than 80m. The casing in a well developed in a sand and gravel formation should extend a minimum of 3 m below the lowest estimated pumping level. In the percussion method of drilling, and where sloughing is a problem, it is customary to drill and jack the casing to the lower extremity of the aquifer, install the appropriate size screen inside the casing, and then pull the casing back, exposing the screen to the water bearing formation. PVC casing shall not be driven or jacked. In consolidated formations, steel casing should be driven 0.50 m into bedrock and cemented in place for its full depth. The wall thickness and pipe strength for the casing material depends on the hydraulic collapse and buckling strengths required for the well. Substitution of other pipe PVC (also called uPVC) for ASTM SCH 80 pipe material shall first be approved by providing a shop submittal (Form 4025) with material specifications sufficient to evaluate the pipe strength for the proposed well application. Reference 3 provides design information used in well casing design.

Table 5 provides the minimum, allowable wall thickness when using Schedule 80 PVC pipe for casing. PVC does not possess the collapse and buckling strength of steel pipe and is limited to relatively shallow wells. PVC casing may be used, only if well depths are less than 80 meters AND with the approval of AED.

Table 5. Minimum PVC SCH 80 Pipe Casing Wall Thicknesses

Casing Diameter		Wall Thickness	
(mm)	(in)	(mm)	(in)
150	6	11	0.43
200	8	13	0.50
250	10	15	0.59

Note: AED does not allow the use of PVC casing in wells deeper than 80 meters.

The minimum wall thickness for steel pipe used for casing is 8 mm. For various diameters, the following Table 6 provides minimum pipe wall thicknesses:

Table 6. Minimum steel pipe casing wall thicknesses by well diameter

Nominal Diameter, mm (in)		Wall Thickness, mm (in)	
150	(6)	8	(.250)
200	(8)	8	(.250)
250	(10)	8	(.279)
300	(12)	9	(.330)
350	(14)	10	(.375)
400	(16)	10	(.375)
450	(18)	10	(.375)
500	(20)	10	(.375)

d) Well screens. Well screen shall be 304 stainless steel, unless specified otherwise. Well screens are designed based on the type of aquifer material encountered during drilling which is why it is important to obtain accurate depth information correlated to samples of the material at various depths and changes in strata. Well screens may be sized for aquifers in which the natural development of the formation is feasible. The minimum inside diameter of the well screen shall be 150mm. Such wells completed in sand and gravel with open-end casings, not equipped with a screen on the bottom, usually have limited capacity due to the small intake area (open end of casing pipe) and tend to pump large amounts of sand. A properly designed screen allows the permeability of the water bearing materials around the screen to be utilized. For a well completed in a sand-gravel formation, use of a well screen surrounded by the natural formation (when developed) will usually provide much more water than if the installation is left open-ended or provided with a fine gravel pack. The screen functions to restrain sand and gravel from entering the well, which would diminish yield, damage pumping equipment, and deteriorate the quality of the water produced. Wells developed in hard rock areas do not need screens if the wall is sufficiently stable and sand pumping is not a problem. Appendix B shows examples of acceptable and unacceptable well screens.

The well screen aperture opening, called slot size is selected based on sieve analysis data of the aquifer material for a naturally developed well. For a homogeneous aquifer formations where the well

is naturally developed (without a gravel pack), the slot size is selected as one that will retain 40 to 50 percent of the aquifer sand. Use 40 percent where the water is not particularly corrosive and a reliable sample is obtained. Use 50 percent where water is very corrosive and/or the sample may be questionable. Most of the water in Afghanistan is highly corrosive, that is it contains a high concentration of total dissolved solids, and unless the well is drilled into rock corrosive water should be assumed. Where a formation to be screened has layers of differing grain sizes and graduations, multiple screen slot sizes may be used. Where fine sand overlies a coarser material, extend the fine slot size at least one meter into the coarser material. This reduces the possibility that slumping of the lower material will allow finer sand to enter the coarse screen. The coarse aperture size should not be greater than twice the fine size. For a filter packed well, the screen should retain 90 to 100 percent of the filter material. Screen aperture size should be determined by a laboratory experienced in this work, based on a sieve analysis of the material to be screened. Consult manufacturer's literature for current data on screens.

Screen length depends on aquifer characteristics, aquifer thickness, and available drawdown. The minimum screen length shall be two to four meters depending upon the screen material and openings. Minimum total length of PVC screens installed in the permanent well shall be four (4) meters. Minimum total length of stainless steel screens shall be two (2) meters. For a homogeneous, confined, artesian aquifer, 70 to 80 percent of the aquifer should be screened and the maximum drawdown should not exceed the distance from the static water level to the top of the aquifer. For a non-homogeneous, artesian aquifer, it is usually best to screen the most permeable strata.

Homogeneous, unconfined (water-table) aquifers shall be equipped with screens covering the lower one-third to one-half of the aquifer. A water-table well is usually operated so that the pumping water level is slightly above the top of the screen. For a screen length of one-third the aquifer depth, the permissible drawdown will be nearly two-thirds of the maximum possible drawdown. This draw-down corresponds to nearly 90 percent of the maximum yield. Screens for non-homogeneous water table aquifers are positioned in the lower portions of the most permeable strata in order to permit maximum available drawdown. The following equation shall be used to determine if a screen length greater than the minimum screen length stated above is required:

$$L=Q/(AV(7.48)) \qquad \text{Eq 3}$$

Where

L=length of screen (ft)
Q=discharge (gpm)
A=effective open area per foot of screen length (ft^2/ft). Approximately ½ of the actual open area which can be obtained from screen manufacturers.
V=velocity (fpm) above which a sand particle is transported; is related to permeability as follows:

Hydraulic conductivity		Velocity sand transport	
m/day	gpd/ft^2	m/min	ft/min
204	5,000	3.05	10
163	4,000	2.74	9
122	3,000	2.44	8
102	2,500	2.13	7
82	2,000	1.83	6
61	1,500	1.52	5
41	1,000	1.22	4
20	500	0.91	3
10	250	0.61	2

The screen diameter shall be selected so that the entrance velocity through the screen openings will not exceed 0.03 m/s (0.1 foot per second). The entrance velocity is calculated by dividing the well yield in cubic feet per second by the total area of the screen openings in square feet. This will ensure the following:

- The hydraulic losses in the screen opening will be negligible.
- The effect of incrustation will be minimal.
- The effect of corrosion will be minimal.

Various procedures may be used for installation of well screens. For cable-tool percussion and rotary drilled wells, the pull-back method may be used. A telescope screen, that is one of such a diameter that it will pass through a standard pipe of the same size, is used. The casing is installed to the full depth of the well, the screen is lowered inside the casing, and then the casing is pulled back to expose the screen to the aquifer.

In the bail down method, the well and casing are completed to the finished grade of the casing; and the screen, fitted with a bail-down shoe is let down through the casing in telescope fashion. The sand is removed from below the screen and the screen settles down into the final position.

For the wash-down method, the screen is set as on the bail-down method. The screen is lowered to the bottom and a high velocity jet of fluid is directed through a self closing bottom fitting on the screen, loosens the sand and allowing the screen to sink to it final position. If filter packing is used, it is placed around the screen after being set by one of the above methods. A seal, called a packer, is provided at the top of the screen. Lead packers are expanded with a swedge block. Neoprene packers are self sealing.

In the hydraulic rotary method of drilling, the screen may be attached directly to the bottom of the casing before lowering the whole assembly into the well.

Well screen shall be 304 stainless steel, unless specified otherwise.

e) Filter Packing. Filter packing (sometimes referred to as gravel packing) is primarily sand and gravel placed around the well screen to stabilize the aquifer and provide a radius of high permeability around the screen. This differs from the naturally developed well in that the zone around the screen is made more permeable by the addition of coarse material. Filter-pack material is more effective when it is composed of clean rounded sand or gravel. Grain size of the filter pack is selected on the basis of information obtained from sieve analyses of the material in the aquifer. The well screen aperture size will be selected so that between 85 and 100 percent of the filter pack is larger than the screen openings. Criteria for sizing the filter pack are as follows:

Perform sieve analyses on all strata within the aquifer. The ASTM standard sieve sizes to be used in performing these analyses are:

Opening size		ASTM seive
mm	in	No
80	3	4
50	2	10
40	1 1/2	20
25	1	40
20	3/4	60
9.4	3/8	3/8
1.16	0.046	16
0.1	0.004	140

The results of the analysis of any particular sample should be recorded as the percent (by weight) of the sample retained on each sieve and the cumulative percent retained on each sieve (i.e., the total of the percentages for that sieve and all larger sieve sizes). Based on these sieve analyses, determine the aquifer stratum which is composed of the finest material.

Using the results of the sieve analysis for the finest aquifer material, plot the cumulative percent of the aquifer material retained versus the size of the mesh for each sieve. Fit a smooth curve to these points. Find the size corresponding to a 70 percent cumulative retention of aquifer material. This size should be multiplied by a factor between 4 and 6, 4 if the formation is fine and uniform and 6 if the formation is coarse and non-uniform. Use 9 if the formation includes silt. The product is the 70 percent retained size (i.e., the sieve size on which a cumulative 70 percent of the sample would be retained) of the material to be used in the packing.

A uniformity coefficient of 2.5 for the filter pack is desirable. The uniformity coefficient is defined as 40 percent of the retained grain size divided by 90 percent retained size. Lower size represents a more uniform material and is more meaningful for values less than 5.

The plot of cumulative percent retention versus grain size for the filter pack should be approximately parallel to same plot for the aquifer material, should pass through the 70 percent retention value and should have 40 and 90 percent retention values such that the uniformity coefficient is less than 2.5. Filter pack material will be specified by determining the sieve sizes that cover the range of the curve and then defining an allowable range for the percent retention on each sieve. Gradations of sand and gravel packs for typical formation material are provided in the guide specification. However, the contractor shall verify these gradations are suitable for the specific well as not all aquifers are the same.

The thickness of the filter pack will range from a minimum of 50 mm (2 in) to approximately 200 mm (8 in). A filter envelope thicker than about 200 mm (8 in) will not greatly improve yield and can adversely affect removal of fines, at the aquifer-filter pack interface, during well development. Filter pack should extend full length of the screen but not above the top of the aquifer. A tremie pipe may be used to evenly distribute the filter material around the screen and also to prevent bridging of the sand grains. It is important that the filter used for packing be clean and that it also be disinfected by immersion in strong chlorine solution (50 mg/L or greater available chlorine concentration, prepared by dissolving fresh chlorinated lime or other chlorine compound in water) just prior to placement. Dirty filter must be thoroughly washed with clean water prior to disinfection and then handled in a manner that will maintain it in as clean a state as possible.

f) Grouting and Sealing. The well should be constructed to prevent water that is polluted or of otherwise unsuitable quality from entering the well. Grout should extend from the surface to the top of the bentonite seal overlying filter pack of the well. Grouting and sealing of wells are necessary to protect the water supply from pollution, to seal out water of unsatisfactory chemical quality, to protect the casing from exterior corrosion and to stabilize soil, sand or rock formations which tend to cave. When a well is constructed there an annular space between the drill hole and the casing is normally produced, which, unless sealed by grouting, provides a potential pollution channel. A bentonite seal with a minimum thickness of three meters shall be placed directly above the filter pack to prevent vertical infiltration of contaminants through filter material into the well. The wellhead must be grouted and sealed at the surface to prevent contaminants from migrating along the casing into the aquifer.

The well casing and the grout seal should extend from the surface to the depth necessary to prevent surface contamination via channels through soil and rock strata. The depth required is dependent on the character of the formations involved and the proximity of sources of pollution, such as sink holes and sewage disposal systems. The grout around the casing should extend from the top of the bentonite seal to the surface of the well. Local regulations may govern the composition and placement of the grout. Materials for sealing and grouting should be durable and readily placed.

Normally, Portland cement grout will meet these requirements. Grout is customarily specified as a neat cement mixture having a water-cement ration of not over 23 L (6 gal) per 43 kg (94-pound) sack of cement. Small amounts of bentonite clay may be used to improve fluidity and reduce shrinkage. Grout can be placed by various methods, but to ensure a satisfactory seal, it is essential that grouting be:

- Done as one continuous operation.
- Completely placed before the initial set occurs.
- Introduced at the bottom of the space to be grouted.

Establishment of good circulation of water through the annular space to be grouted is a highly desirable initial step toward a good grouting job. This assures that the space is open and provides for the removal of foreign material.

Formations containing water of poor quality above the target aquifer may be sealed off by grouting an outer casing in place before installing the deeper well casing. If the undesirable aquifer is the lower one, care should be taken during drilling so as not to penetrate or breach the confining unit separating the two aquifers. Any portion of the confining unit that is breached should be replaced with grout.

g) Accessibility. The well location shall be readily accessible for pump repair, cleaning, disinfection, testing and inspection. The top of the well shall never be below surface grade. At least 600 mm (2 ft) of clearance beyond any building projection shall be provided.

h) Details Relating to Water Quality. In addition to grouting and sealing, features that are related to water quality protection are:

Location. The well or wells should be located on the highest ground practicable, certainly on ground higher than nearby potential sources of surface pollution. The surface near the site should be built up, by fill if necessary, so that surface drainage will be away from the well in all directions. Where flooding is a problem, special design will be necessary to insure protection of wells and pumping equipment from contamination and damage during flood periods and to facilitate operation during a flood.

Concrete Cover. The well casing should be surrounded at the surface by a concrete slab having a minimum thickness of 100 mm (4 in) and extending outward from the casing a minimum of 600 mm (2 ft) in all directions. The slab should be finished a little above ground level and slope slightly to provide drainage away from the casing in all directions.

Casing Height. The well casing should extend at least 500 mm (20 in) above the level of the concrete surface slab in order to provide ample space for a tight surface seal at the top of the casing. The type of seal to be employed depends on the pumping equipment specified.

Well House. While not universally required, it is usually advisable to construct a permanent well house, the floor of which can be an enlarged version of the surface slab. The floor of the well house should slope away from the casing toward a floor drain at the rate of about 1 mm per 50 mm (1/8 inch per foot). Floor drains should discharge through carefully jointed 100 mm (4 in) or larger pipe of durable water-tight material to the ground surface 6 m (20 ft) or more from the well. The end of the drain should be fitted with a coarse screen. Well house floor drains ordinarily should not be connected to storm or sanitary sewers to prevent contamination from backup. The well house should have a large entry door that opens outward and extends to the floor. The door should be equipped with a good quality lock. The well house design should be such that the well pumps motor, and drop-pipe can be removed readily. The well house protects valves and pumping equipment and also provides some freeze protection for the pump discharge piping beyond the check valve. Where freezing is a problem, the well house should be insulated and a heating unit installed.

The well house should be of fireproof construction. The well house also protects other essential items. These include:

- Flow meter
- Depth gage
- Pressure gage
- Screened casing vent
- Sampling tap
- Water treatment equipment (if required)
- Well operating records

Security. The well building shall be protected from unauthorized use by a security fence having a lockable gate.

7. Well Pumps

a) Pump Type. Many types of well pumps are on the market to suit the wide variety of capacity requirements, depth to water and power source. Electric power is used for the majority of pumping installations. Where power failure would be serious, the design should permit at least one pump to be driven by an auxiliary engine, usually gasoline, diesel or propane. The most appropriate type is dictated by many factors for each specific well. Factors that should be considered for installation are:

- Capacity of well
- Capacity of system
- Size of well
- Depth of water
- Type of well
- Power source
- Standby equipment
- Well drawdown
- Total dynamic head

There are several types of well pumps. The most common are line shaft turbine, submersible turbine, or jet pumps. The first two operate on exactly the same principal; the difference being where the motor is located. Line shaft turbine pumps have the motor mounted above the waterline of the well and submersible turbine pumps have the motor mounted below the water line of the well. Jet pumps operate on the principal of suction lift. A vacuum is created sufficient to "pull" water from the well. This type of pump is limited to wells where the water line is generally no more than 8 m (25 ft) below the pump suction. It also has small capacity capability.

USACE-AED projects commonly employ jet pumps or small submersible turbine pumps. However, there may be projects where other types are preferable. For deep wells with high capacity requirements, submersible or line shaft turbine pumps are usually used and are driven by electric motors. A number of pump bowls may be mounted in series, one above the other to provide the necessary discharge pressure. Characteristics of pumps used in wells are listed in Table 7.

Table 7. Characteristics of Pumps Used in Water Supply Systems

Type of Pump	Practical suction lift	Usual well-pumping depths	Usual pressure heads	Advantages	Disadvantages	Remarks
Reciprocating: 1. Shallow well . . . 2. Deep well . . .	22–26 ft. 22–26 ft.	22–26 ft. Up to 600 feet	100–200 ft. Up to 600 feet above cylinder.	1. Positive action. 2. Discharge against variable heads. 3. Pumps water containing sand and silt. 4. Especially adapted to low capacity and high lifts.	1. Pulsating discharge. 2. Subject to vibration and noise. 3. Maintenance cost may be high. 4. May cause destructive pressure if operated against closed valve.	1. Best suited for capacities of 5–25 gpm against moderate to high heads. 2. Adaptable to hand operation. 3. Can be installed in very small diameter wells (2" casing). 4. Pump must be set directly over well (deep well only).
Centrifugal: 1. Shallow well a. straight centrifugal (single stage)	20 ft. maximum	10–20 ft.	100–150 ft.	1. Smooth, even, flow. 2. Pumps water containing sand and silt. 3. Pressure on system is even & free from shock. 4. Low-starting torque. 5. Usually reliable and good service life.	1. Loses prime easily. 2. Efficiency depends on operating under design heads & speed	1. Very efficient pump for capacities above 50 gpm & heads up to about 150 feet.
b. Regenerative vane turbine type (single impeller)	28 ft. maximum	28 ft.	100–200 ft.	1. Same as straight centrifugal except not suitable for pumping water containing sand or silt. 2. They are self-priming.	1. Same as straight centrifugal except maintains priming easily.	1. Reduction in pressure w/increased capacity not as severe as straight centrifugal.
2. Deep well a. Vertical line shaft turbine (multi-stage)	Impellers submerged	50–300 ft.	100–800 ft.	1. Same as shallow well turbine.	1. Efficiency depends on operating under design head & speed. 2. Requires straight well large enough for turbine bowls and housing. 3. Lubrication & alignment of shaft critical. 4. Abrasion from sand.	

Depending upon contract technical requirements, a hand pump may also be included in the well pump building. This hand pump allows water to be extracted from the well during periods when power is not available. Hand pump components should be stainless steel and the head and flow requirements must be selected based on site conditions.

b) Pump Capacity. The design capacity of the pump must equal the system requirements. Well pump capacity shall be capable of supplying one average day flow (ADF) in a 16 hour period unless stated otherwise in the contract documents. However, the capacity of the pump must never exceed the capacity of the well. Pump manufacturers publish charts giving the pump discharge capacity for their particular pumps at various operating pressures. The total dynamic head (TDH) of the system must be calculated accurately from the physical arrangement and is represented by the following:

$$TDH=H_S+H_D+H_F+(V^2/2g) \qquad \text{Eq 1}$$

Where:

H_s=suction lift; vertical distance from the waterline at drawdown under full capacity to the pump centerline, m
H_D=discharge head; vertical distance from the pump centerline to the pressure level of the discharge pipe system, m
H_F=friction head; loss of head on pipe lines and fittings, m
$V^2/2g$=velocity head; head necessary to maintain flow, m

The brake horsepower of the motor used to drive the pump may be calculated from the following equation:

$$P=(HQ)/(102*e) \qquad \text{Eq 2}$$

Where:

P=break power required, kW
H=total dynamic head, m
Q=volume of water discharged, L/s
e=Combined efficiency of pump and motor, from manufacturer's data

Appendix C contains information for designers on selection of water well pumps.

8. Development and Disinfection

After the structure of the well is installed, there remain two very important operations to be performed before the well can be put into service. Well development is the process of removing the finer material from the aquifer around the well screen, thereby cleaning out and opening up passages in the formation so that water can enter the well more freely. Disinfection is the process of cleaning and decontaminating the well of bacteria that may be present due to the drilling action.

a) Well Development. Three beneficial aspects of well development are to correct any damage or clogging of the water bearing formation which occurred as a side effect of drilling, to increase the permeability of the formation in the vicinity of the well and to stabilize the formation around a screened well so that the well will yield sand-free water.

A naturally developed well relies on the development process to generate a highly permeable zone around the well screen or open rock face. This process depends upon pulling out the finer materials from the formation, bringing them into the well, and pumping them out of the well. Development work should continue until the movement of fine material from the aquifer ceases and the formation is stabilized.

Artificial filter packing provides a second method of providing a highly porous material around the screen. This involves placement of a specially graded filter in the annular space between the screen and the wall of the excavation. Development work is required if maximum capacity is to be attained.

Development is necessary because many drilling methods cause increases in the density of the formation around the hole. Methods utilizing drilling fluids tend to form a mud cake. Good development will eliminate this "skin effect" and loosen up the sand around a screen. Removal of

fines leaves a zone of high porosity and high permeability around the well. Water can then move through this zone with negligible head loss.

Methods of development in unconsolidated formations include the following:

Mechanical surging is the vigorous operation of a plunger up and down in the well, like a piston in a cylinder. This causes rapid movement of water which loosen the fines around the well and they can be removed by pumping. This may be unsatisfactory where the aquifer contains clay streaks or balls. The plunger should only be operated when a free flow of water has been established so that the tool runs freely.

Air surging involves injecting air into a well under high pressure. Air is pumped into a well below the water level causing water to flow out. The flow is continued until it is free of sand. The air flow is stopped and pressure in an air tank builds to 700 to 1,000 kilopascals (100 to 150 psi). Then the air is released into the well causing water to surge outward through the screen openings.

Over pumping is simply pumping at a higher rate than design. This seldom brings best results when used alone. It may leave sand grains bridged in the formation and requires high capacity equipment.

Back washing involves reversal of flow. Water is pumped up in the well and then is allowed to flow back into the aquifer. This usually does not supply the vigorous action which can be obtained through mechanical surging.

High velocity jetting utilizes nozzles to direct a stream of high pressure water outward through the screen openings to rearrange the sand and gravel surrounding the screen. The jetting tool is slowly rotated and raised and lowered to get the action to all parts of the screen. This method works better on continuous slot well screens better than perforated types of screens.

Development in rock wells can be accomplished by one of the surging methods listed above or by one of the following methods.

Explosives can be used to break rock formations. However it may be difficult to tell in advance if the shooting operation will produce the required result.

Acidizing can be used in wells in limestone formations. Fractures and crevices are opened up in the aquifer surrounding the well hole by the action of the acid dissolving the limestone.

Sand fracturing is the action of forcing high pressure water containing sand or plastic beads in to the fractures surround a well. This serves to force the crevices open.

b) Disinfection of Completed Well. The disinfection of the completed well shall conform to Guide Specifications (Attachment A). Bacteriological samples must be collected and examined in accordance with Standard Methods for the Examination of Water and Wastewater.

c) Disinfection of Flowing Artesian Wells. Flowing artesian wells often require no disinfection, but if a bacteriological test, following completion of the well, shows contamination, disinfection is required. This can be accomplished as follows. The flow from the well will be controlled either by a cap or a standpipe. If a cap is required, it should be equipped with a one-inch valve and a drop-pipe extending to a point near the bottom of the well. With the cap valve closed, stock chlorine solution will be injected, under pressure, into the well through the drop-pipe in an amount such that when the chlorine solution is dispersed throughout all the water in the well, the resultant chlorine concentration will be between 50 and 100 mg/L. After injection of the required amount of stock chlorine solution, compressed air will be injected through the drop-pipe, while simultaneously partially opening the cap valve. This will permit the chlorine solution to be mixed with the water in the well. As soon as chlorine

is detected in the water discharged through the cap valve, the air injection will be stopped, the cap valve closed and the chlorinated water allowed to remain in the well for 12 hours. The well will then be allowed to flow to waste until tests show the absence of residual chlorine. Finally, samples for bacteriological examination will be collected in accordance with Standard Methods for the Examination of Water and Wastewater. If the well flow can be controlled by means of a standpipe, disinfection can be accomplished as described for a water table well.

9. As-Builts

Upon completion of installing the well and well pump, 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, logs and any other changes made to the original drawings. Refer to Contract Sections 01335 and 01780A of the specific project for additional details.

10. References

1. Groundwater and Wells, Fletcher Driscoll, Johnson Division, 1986.
2. UFC 3 230 07a Water Supply Sources General Considerations, 2004
3. Handbook of Groundwater Development, Roscoe Moss Company, 1990
4. ASTM F480-06b Standard Specification for Thermoplastic well Casing Pipe and Couplings Made in Standard Dimension Ratios, SCH 40 and SCH 80
5. ASTM D 1785-06 Standard Specification for PVC Plastic Pipe, Schedules 40, 80 and 120
6. ASTM A 53 Pipe, Steel, Black and Hot-dipped, Zinc-coated, Welded and Seamless
7. Inventory of Ground-Water Resources in the Kabul Basin, Afghanistan, U.S. Geological Survey, Scientific Investigations Report, 2005-5090.
8. Guidelines for Sustainable Use of Groundwater in Afghanistan, Norwegian Church Aid, 2002
9. Sanitary Control and Surveillance of Field Water Supplies, TB MED 577, Department of Defense 2005
10. World Health Organization, Guidelines for Drinking Water Quality, 2006
11. AWWA A-100-06 Water Wells
12. Water Wells and Pumps, Michale,A.M.; Khepar, S.D., and Sondhi, S.K., McGraw Hill, 2008

Appendix A

Guide Specification for Drinking Water Wells
USACE-AED Various Locations, Afghanistan

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1. Applicable Publications
2. Location and Depth of Well
3. Local Conditions
4. Protection of Existing Facilities
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Attachment A Water Well Construction Process

1. APPLICABLE PUBLICATIONS: Publications listed below form a part of this specification to the extent referenced. Publications are referred to by the basic designation .

a. Sanitary Control and Surveillance of Field Water Supplies, TB MED 577, Department of Defense 2005

b. World Health Organization, Guidelines for Drinking Water Quality, 2006

1.3 American Society for Testing and Materials (ASTM) Publications.

A-53 Pipe, Steel, Black and Hot-dipped, Zinc-coated, Welded and Seamless

C-150 Portland Cement

1.4 American Water Works Association (AWWA) Publications.

Standard Methods for the Examination of Water and Wastewater

A100-06 Standard for Water Wells

C200 Steel Water Pipe 6 Inches & Larger

C206 Field Welding of Steel Water Pipe

2. LOCATION AND DEPTH OF WELL: The test well to be constructed shall be located as shown on the drawings, or where directed by the Contracting Officer (CO) or his representative (COR). The well shall be to such depth as may be necessary to penetrate a desirable water-bearing stratum. The minimum well depth for each well shall be 20 meters below the static water table. Additional depth to be determined based on the experience of the driller and similar wells in the area may be required by estimating the additional depth that may be required to provide a minimum 2 meters of submergence over the well pump during the dynamic drawdown of water during test pumping. It may be necessary for the well depth to penetrate the full thickness of the aquifer to bedrock to achieve sufficient depth.

3. LOCAL CONDITIONS: Hydrogeological and water-well information for the immediate vicinity should be obtained locally from private and/or government organizations. The driller should drill test hole as hereinafter specified to assure the water availability of the aquifer. The method of drilling and the type of well construction may vary depending on the local geology. Wells completed into unconsolidated aquifers may need to be screened depending upon the type of aquifer material or if completed into consolidated (rock) aquifers, open-hole construction may be approved.

4. PROTECTION OF EXISTING FACILITIES: The existing facilities such as building structures, utilities, walks, trees, etc., except as otherwise specified in these specifications, shall be protected from damage during construction of the wells, and if damaged, shall be repaired by the Contractor at his expense. Water pumped from the well shall be conducted to a place where it will be possible to dispose of the water without damage to property or the creation of a nuisance.

5. PROTECTION OF QUALITY OF WATER: The Driller shall take all necessary precautions during construction to prevent contaminated water, gasoline or other contaminated materials from entering the well either through the opening or by seepage through the ground surface. The Driller shall exercise extreme care in performance of his work in order to prevent the breakdown or caving of the strata overlying that from which the water is to be drawn.

6. TEST WELL DEVELOPMENT: Before permanent well construction, at least one test well of at least 150mm in diameter and minimum 20 meters below the static water table shall be drilled into the water bearing stratum or the top of bedrock. A GPS instrument will be used to determine the geographic coordinates of the well. This information shall meet requirements of the World Geodetic System 1984 (WGS 84 and the correct UTM Zone – 41, 42, or 43) in decimal degrees. The test hole shall be used to determine the location and character of the water bearing strata and to obtain samples of the various formations. Samples of drilling cuttings shall be taken at every change of strata and at depth intervals not to exceed 1.5 meters. A split spoon sample shall be taken at each major change in strata as

indicated by the nature of the cutting samples. A driller's log shall be made based on the cuttings obtained. The final selection of the screen settings, the proper gravel filter pack material, and the depth of grouting will submitted to the COR for approval prior to any additional work. The drill cuttings shall be divided, put into suitable containers and labeled. These samples shall be approximately half a liter each. If the test hole fails to indicate the presence of water bearing strata or is abandoned for any other reason, the test hole shall be grouted from the bottom to the top with cement grout as hereinafter specified and in a manner approved by the COR. At the completion of the test hole, a drillers log shall be prepared containing the following information that shall be included in the Driller Log Submittal:

a) Depth of water strata
b) Depth of different material strata contacts
c) Color, size, and soil description of cuttings
d) Penetration rate (meters per day)
e) Types and amount of drilling fluid gain or loss
f) Type schedule and length of well casing

7. TEST WELL PUMPING. To determine the expected yield from the well and to assure acceptable water quality, a pump test shall be performed in the candidate test well. A temporary casing and screen, if required, shall be used to construct the test well for execution of a 3 phase step-drawdown test and a 24-hour constant volume capacity test per AWWA 100-6 requirements. (The intervals provided in Table 6 later in this specification may be used to record data during the 3 phase step-duration and the constant volume capacity tests.) The 3 phase step-drawdown test shall test the well at 75%, 100% and 150% of the design flow required for the well. The duration of the test shall be adequate to develop the straight line plots defined by AWWA. A temporary pump with the capacity to pump at the rates may be used for this test. The static water level in the well should be measured prior to installing the pump and the water level at the end of the pumping period should be measured. The temporary casing shall be a minimum diameter of 150mm and extend to the top of the water bearing strata being tested in unconsolidated formations. The Contractor shall isolate other water bearing strata present from discharging into the test well. After completion of the test well, all data pertaining to the construction of the well shall be shown on a sketch of the test well with all pertinent depths of construction.

8. APPROVAL OF TEST WELL. **CONSTRUCTION OF THE WELL SHALL STOP.** Before any further construction in the test well, AED shall receive, review and approve the Test Well Approval Submittal and shall authorize construction of the permanent well. The contents of the Test Well Approval Submittal shall include:

(a) Location of well on site plan.
(b) Size of well diameter and depth.
(c) Driller Log Submittal (Section 6, above)
(d) Depth below top of well of static water table.
(e) Casing and screen diameters and lengths installed in the test well
(f) Well Screen and Gravel Pack Submittal (Section 9.5, below)
(g) Proposed permanent casing diameter and material

(h) Proposed permanent screen design with sand analysis results
(i) Proposed permanent grouting and sealing
(j) Proposed permanent gravel/filter pack design and materials and supporting calculations
(k) Proposed placement depths of gravel pack material, if used.
(l) Test well pumping rate, liters per second
(m) Test well drawdown and recovery records (see Table 6)
(n) Water quality results (see Table 4)

Test wells not approved, shall be abandoned in accordance with AWWA A 100-106.

9. WELL CONSTRUCTION:

9.1 GENERAL: **Construction of the permanent well shall not begin before AED review and approval of the Test Well Submittal.** Failure to follow the construction and submittal procedures outlined, may at AED's discretion, result in rejection of the well and, the contractor having to remove the well casing and screen, re-drill the well and reinstall the proper features per the approved design. Well construction shall be based on well design guidelines set forth in the latest edition of the US Army Corps of Engineers Afghanistan Engineer District (AED) Design Requirements, Well Pumps and Well Design guide. Test wells and permanent wells should be at least 20 meters below the static water table. The pump, at actual capacity, should have a minimum of 2 meters of submergence at the lowest drawdown depth reached during the pump tests described later in this guide. Well screens shall have a minimum of 2 meters submergence at the lowest drawdown depth occurring during well testing. Permanent wells shall not operate with any portion of the well screen above the lowest drawdown level. The execution of the work shall be by a competent crew performed under the direct supervision of an experienced well driller acceptable to the COR. The well shall be drilled essentially straight, plumb and circular from top to bottom. The type of well construction, as determined by the test hole, shall be either an artificially gravel packed well, a naturally developed well or a rock wall well. The presence of cavities and voids in the formations being penetrated will make the construction of a screened well difficult due to loss of the filter material into the cavity. Grouting may also be affected by these voids. If cavities and voids are encountered in the test hole, the Contractor shall consider this in the design of the completed well and submit this information with the results of the test hole and test well. The well shall be constructed and developed to produce clear water with a minimum of drawdown. Wells not meeting the criteria for water quality, Section 10 WELL DEVELOPMENT, will not be acceptable. Well and casing dimensions shall generally be within the following ranges shown in Table 1.

TABLE 1 Minimum Required Well and Casing Diameters

anticipated water yield, l/s	well casing/ screen ID, mm	well diameter, mm
<6.3	150	200
6.4 to 11.0	200	250
11.1 to 25.25	250	300

9.2 SURFACE CASING: Surface casing shall be used on all wells. This casing may be left in place or removed at the option of the driller. If the surface casing is to be left in place, the surface casing shall be grouted into place for the full length of the casing. In unconsolidated aquifer material, casing shall be extended to the top of the well screen. In wells developed in rock formations, the hole may be left open with the casing extended three meters into the formation. All well surface casing shall be extended 0.5 meters above grade.

9.3 WELL CASING: The well casing shall be steel unless PVC is specifically authorized for use by AED. The casing diameter shall be selected per information provided in the AED Design Requirements – Well Pumps and Well Design, latest edition. The casing shall be constructed of ASTM A-53 black steel pipe conforming to AWWA C200 or PVC plastic Schedule 80 meeting ASTM 1785 and ASTM F480 specifications. Only standard weight black steel pipe shall be used as casing for wells over 80 meters deep. The minimum steel casing thickness shall be 8 mm. Steel casing may be provided with drive shoes at the option of the driller. Steel pipe couplings shall be per ASTM F480. PVC casing may be used if well depths are less than 80 meters. Driving of PVC casing will not be allowed. PVC pipe couplings shall be per ASTM F480. All casing used in the permanent well shall be new and unused. Cast iron pipe shall not be used for well casing or screens. Sufficient casing centralizers shall be used to keep casings centered in the hole. Each section of casing will be joined with standard couplings; full-threaded joints, proper welding or solvent cement PVC welds so the joints are sound and watertight. Well casing alignment shall not interfere with the proper installation and operation of the pump.

9.4 CEMENTING WELL CASING: The annular space between the well casing and the walls of the hole shall be filled with cement grout or crushed angular gravel as hereinafter specified. The grout shall be proportioned of Portland cement conforming to ASTM Specification C-150, Type I or II and the minimum quantity of water (not over 23 liters per 42.3 kg of cement) required to give a mixture of such consistency that it can be forced through the grout pipes. The method for placing the grout will be by the forcing of the grout from the bottom of the space to be grouted towards the surface. The minimum depth of grouting shall be three meters unless approved in writing by the COR. The grout shall also seal off any other water bearing strata above the zone that is producing the water to the well. The grouting shall be done continuously and in a manner that will insure the entire filling of the annular space in one operation without damaging the well casing. No drilling operations or other work in the wells will be permitted within 24 hours after the grouting operation to allow the grout to properly set. Up to 5% bentonite may be added to the mixture to reduce shrinkage of the grout. The addition of bentonite will require additional water at the ratio of 2.5 liters of water for each percent of bentonite added. The bentonite slurry shall contain at least 20 percent solids by weight by having a density of 4.3 kg/L. Aggregate requirements for sealing the solid wall casing and edge of the borehole area shown below:

Sieve Size (mm)	Percent total weight passing (%)
12.5	100
4.75	75 +/- 13
1.18	25 +/- 15
.075	8 +/- 4

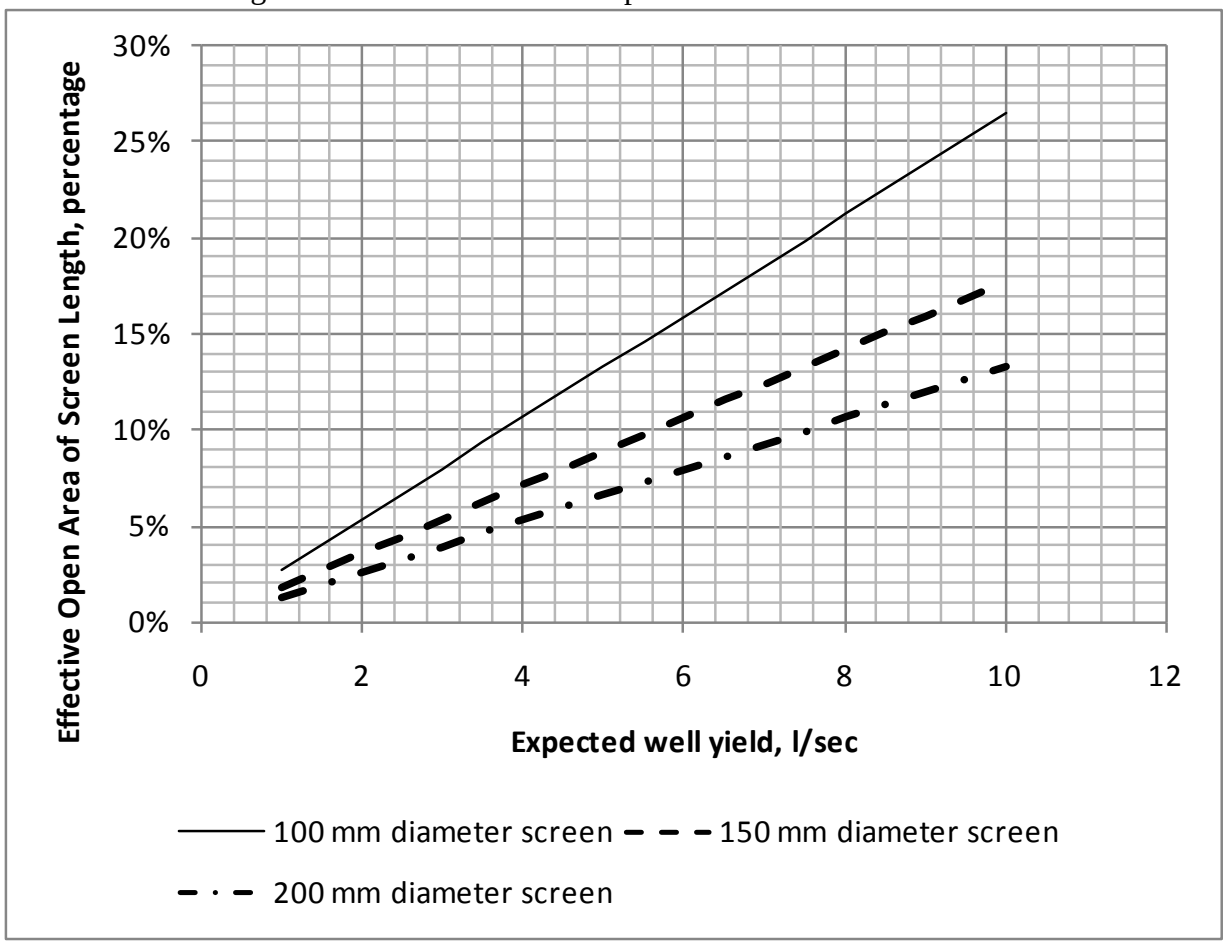
9.5 WELL SCREEN: The well screen and attached fittings shall be constructed entirely of corrosion resistant stainless steel unless specifically authorized by AED. Steel screens shall be made from Type 304 stainless steel meeting the requirements of AWWA A100. High chloride concentrations will quickly corrode other metals. When allowed, PVC pipe screens shall be manufactured from Schedule 80 PVC pipe. Screens made of UPVC pipe material shall not be permitted. The screen slot pattern shall be continuous slot, wire wound or slotted PVC pipe design. Slot openings shall be continuous around the screen, spaced to provide open area required for maximum entrance velocity criteria, but be consistent with the strength requirements, and should be V-shaped toward the inside of the screen to reduce clogging. Slot sizes shall be typical sizes per manufacturer specifications (see Table 2). Metal screens shall be used for wells greater than 80 meters in depth. The minimum inside diameter of the well screen shall be 150mm. Minimum total length of PVC screens installed in the permanent well shall be four (4) meters. Minimum total length of steel screens shall be two (2) meters. Well screens shall provide adequate transmitting capacity to limit the entrance velocity to less than 0.03 m/sec (0.1 ft/s). The selected screen slot design shall be based on standard manufacturer specifications and meet the minimum effective open area required for entrance velocity criteria. Well screen design information shall be submitted to the project COR for approval prior to installation. In a gravel pack well, the screen shall have a slot size based on the gradation of the filter material which is indicated in 9.7 GRAVEL-PACKED WELL. Screens shall be machine milled by a recognized screen manufacturer and not hand cut in the field. Field fabricated screens are not permitted. Appendix B shows examples of unacceptable and acceptable well screens. The well screen shall be directly connected to the top of the inner casing. The bottom of the screen shall be sealed with a positive closure. A wash-down shoe may be used if desired. The screen used in a gravel-packed well shall be carefully lowered into the water-bearing strata and be centered in the hole. In a naturally developed well (see 9.6 NATURALLY DEVELOPED WELL), the slot size of the screen will be based on a mechanical sieve size analysis of the natural water bearing sediments. The well screen and all accessories required for satisfactory installation shall be essentially standard products of reliable manufacturers regularly engaged in the production of such equipment. Field welding of screen components shall be accomplished using products made to weld such products together in a reliable manner. Well Screen and Gravel Pack Submittal information shall include:

(a) Number of screens and depth of setting in the well.
(b) Size of well screen inside and outside diameters and length.
(c) Pipe material (material schedule and specification).
(d) Standard slot opening, mm
(e) Effective open area of screen (sq cm per meter)
(f) Transmitting capacity (liters/meter)
(g) Sieve analysis of the material to be screened
(h) Calculations supporting screen slot size
(i) Photo of screen slot pattern
(j) Results of Gravel Pack selection analysis.

TABLE 2 Typical Effective Open Area for Slotted Well Screens

Nominal Well Screen Diameter	Screen Slot Size		Steel Continuous Slot		PVC Continuous Slot		PVC Slotted Pipe	
mm	No	mm	cm2/m	%	cm2/m	%	cm2/m	%
100	20	0.508	931	25	-	-	-	-
100	60	1.524	1,905	52	1,100	30	381	11
150	30	0.762	1,693	25	1,206	18	550	8
150	60	1.524	2,857	41	1,968	29	995	14
150	95	2.413	3,492	51	-	-	-	-
200	30	0.762	1,629	16	-	-	-	-
200	60	1.524	2,857	28	-	-	-	-
200	95	2.413	3,851	38	-	-	-	-
200	125	3.175	4,529	45	-	-	-	-

Figure 1 Minimum Effective Open Area versus Well Yield



9.6 NATURALLY DEVELOPED WELL:

9.6.1 GENERAL: In water bearing coarse grained material defined as gravel, cobbles, and conglomerate more water can be obtained by natural development of the permeable zone around the well screen than using a gravel pack. After setting the surface casing, a naturally developed well shall be initially drilled by reaming the test hole from the ground surface to the lower level of the water-bearing strata. The well casing and screen shall not be less than 150mm in diameter. The hole shall be of sufficient size to leave a concentric annular space of not less than 50mm between the outside of the casing and the walls of the hole. A grouting basket shall be used to keep the grout from invading the zone around the well screen. The space around the casing shall be filled with cement grout as hereinbefore specified. After grouting is completed, drilling operations shall not be resumed for at least 24 hours to allow proper setting of the grout. After the grout has set, the well shall be developed by pumping or bailing until the fine grained material is removed.

9.7 GRAVEL-PACKED WELL:

9.7.1 GENERAL: Gravel-packed wells shall be constructed in fine grained aquifer materials consisting of silty sand, sand and fine gravel (less than material 3 mm in size). The space around the well screen shall be filled with gravel pack material as described hereafter; either gradation A or gradation B shall be used based on a mechanical sieve size analysis of the natural water bearing sediments. The term “gravel-packed well” does not actually mean that the filter material is always gravel sized material. The filter material may be sand sized. The gradation depends upon the size of the aquifer material that yields the water. Coarse grain aquifers shall be considered to be any water bearing stratum that has more than 50 percent of the individual particle sizes greater than 6 mm in diameter. Aquifers with greater proportion of fine sand and silt (less than 6 mm size) shall be considered fine grained aquifers. After setting the surface casing, a gravel-packed well shall be initially drilled by reaming the test hole from the ground surface to the lower-most level of the water bearing strata. The casing and screen shall be as herein specified and shall be not less than 150mm in diameter. The hole shall be of sufficient size to leave a concentric annular space of not less than 50mm between the outside of the screen and casing and the walls of the hole. The gravel pack shall be 1.5 times the length of the screen section it surrounds. For example if the screen is 3 meters in length the gravel pack shall be 4.5 meters in length straddling the screen on the top and bottom. The space around the screen shall be filled with filter material as hereinafter specified. The well casing shall be grouted in with cement grout as hereinafter specified. After grouting is completed, drilling operations shall not be resumed for at least 24 hours to allow proper setting of the grout. The hole below the outer casing shall penetrate the water bearing strata a sufficient depth to install the well screen and produce the required yield without causing excessive velocities through the well screen. The casing shall be connected directly to the top of the well screen and extend up to one foot above the ground surface.

9.7.2 GRAVEL-PACK: After the screen and casing have been installed, filter material shall be installed around the screen by filling the entire space between the screen and the walls of the hole with filter material. The filter shall have a wall thickness of 50mm, measured from the outer edge of the screen to the wall of the hole. Filter material shall be of properly sized, graded, well-rounded natural sand and gravel suitable for the strata encountered. Angular aggregate shall not be used. The filter material shall be of such size as will allow the maximum flow of water into the well and prevent the infiltration of sand. It shall be washed siliceous material, reasonably smooth and round and free of flat or elongated pieces as well as dirt, vegetable matter or other foreign matter. The Driller shall demonstrate to the COR that the filter material is suitable for the conditions prior to placement and submit a gradation of the material for approval. In no case will improperly sized filter material be added around the screen.

Gravel pack gradations shall be based on the aquifer particle size. Silty sand particle size aquifers shall use Gradation A (Table 3), and gravel and larger particle size material shall use Gradation B (Table 4).

TABLE 3 Gradation A Gravel Pack Materials

Material for gravel pack in silty sand aquifers - Shall consist of stone containing rounded shapes and surfaces with no flat surfaces having the following gradation:

Sieve Size	% Total Wt. Passing
12.5 mm	100
4.75 mm	75 +/- 13
1.18 mm	25 +/- 15
0.425mm	8 +/- 4

TABLE 4 Gradation B Gravel Pack Materials

Material for gravel pack in sandy gravel including cobbles - Shall consist of stone containing rounded shapes and surfaces with no flat surfaces have the following gradation:

Sieve Size	% Total Wt. Passing
40 mm	100
32 mm	75 +/- 13
16 mm	25 +/- 15
9 mm	8 +/- 4

All aggregate shall contain less than 5% by weight rock powder, silt, clay, shale, clay lumps, coal, lignite, soft stone, or other deleterious materials.

9.7.3 PIPE OR CONDUCTOR FOR FILTER PLACEMENT: If possible, a pipe or conductor having an inside nominal diameter of not less than 25mm shall be lowered to the bottom of the well between the drilled hole and the screen. It shall be so arranged and

connected at the surface of the ground to water pumping and filter placement equipment so that water and filter material, fed at uniform rates, are discharged through it as the filter fills the hole from the bottom up. The filter sand and water conductor shall be raised at the rate that will keep the bottom of the pipe approximately at the filter material level in the hole.

9.8 ROCK WELL: A rock well shall be initially drilled from the ground surface to a point at least 3 meters below the top of consolidated material (bedrock), but not less than 10 meters below the top of the rock and the bottom of the casing shall be set at this bottom elevation. The finished internal diameter of the casing shall be not less than 150mm, and the hole shall be drilled to a sufficient diameter so as to leave a concentric annular space not less than 37mm (1 1/2-inches) between the casing and the walls of the hole. A temporary casing may be used to prevent caving of the hole walls, but the temporary casing must be removed when the grouting of the permanent casing is performed. This space shall be filled with cement grout in a manner as previously specified. After the grouting is completed, drilling operations shall not be resumed for at least 24 hours to allow proper setting of the grout. Drilling into the water-bearing rock strata shall be resumed after the grout has set. A hole at least 150mm in diameter, concentric with the casing above, shall be drilled into the water-bearing rock a sufficient depth to produce the required amount of water without causing excessive velocities of the water through the rock.

10. WELL DEVELOPMENT: After completion, the well shall be thoroughly developed. The developing equipment shall be of sufficient capacity to remove all drilling fluids, sand, rock cuttings or any other foreign matter. The wells shall be thoroughly cleaned from top to bottom before beginning the well tests. The recommended type of development for stainless steel screened wells is hydro-jetting; however surge blocks, air-development or other development techniques are permissible at the option of the Contractor. The well shall be disinfected before removing the test pump and collecting samples for determining the water quality (see 13. DISINFECTION).

11. WELL PUMPS:

11.1 PERMANENT WELL PUMP: The pump and motor diameter shall be at least 25 mm smaller than the inside diameter of the well screen or casing, whichever is smaller, in order to allow it to be removed for servicing after the buildup of scale on the outside of the pump and inside of the screen and casing. Before any installation of the permanent pump in the well, a pump design approval shall be submitted to the COR for approval. Pump Design Approval submittals shall include the following information:

(a) Capacity of well from test well submittal.
(b) Size of permanent well diameter and depth.
(c) Depth of static water level below top of well.
(d) Power source and pump motor electrical power requirements.
(e) Expected well draw down.
(f) Pump discharge piping diameter, length, fittings and appurtenances (valves, meters, etc.)
(g) System total dynamic head required by well pump.

(h) Pump design discharge rate.
(i) Selected pump curve and duty point from manufacturer.
(j) Pump/motor product material specifications from manufacturer.

11.2 HAND PUMP: A standard hand pump with seal and air gap shall be installed on the permanent well discharge piping to discharge at concrete pad around well if required by the contract technical requirements. The hand pump shall be capable of pumping at a minimum pressure head of 138 Kpa (20 psi) in the event there is either a loss of power supply or a pump failure in the water well system.

11.3 WELL HEAD COMPLETION: A stainless steel check valve (if not on the pump discharge piping), isolation valve, sampling port, and well discharge tantalizer shall be installed on the pump discharge pipe line, as minimum appurtenances, between the well head and the water storage treatment tanks. A water level measurement port shall be provided at the well head. Additional appurtenances such as flow meter, air release valve, and chlorination treatment piping may be required depending upon the contract technical requirements. Any structure built over the well must have a large door in the roof that allows for easy removal of the pump piping and pump using an overhead crane. Examples of well head construction are shown in Appendix B.

12. TESTS:

12.1 TEST FOR QUALITY OF WATER: During the testing of the test well and again during the yield and drawdown test in the permanent well, the Contractor shall schedule to obtain a preliminary sample of the water in suitable containers and of sufficient quantity to have bacterial, physical and chemical analyses made in accordance with the following Table 5 to determine if the water is potable. The word "potable" for purpose of this contract is further defined as water that is suitable for drinking by the public, i.e., good, clear water free from objectionable amounts of harmful bacteria and chemical and physical properties, as defined by References 1.1 or 1.2. Sampling shall be performed by qualified personnel who must obtain sampling kits and schedule their site visit to obtain the samples as directed. The coordination with the COR for the sampling and analysis should begin at the beginning of the contractual period. Complete requirements for water quality are found in Appendix A of TM 5-813-3 (UFC 3 230 08a Water Supply Water Treatment, January 2004).

TABLE 5

WATER QUALITY ANALYSIS TABLE

Physical Characteristics

Color	Temperature
Threshold odor number	pH value
Turbidity	

Chemical Characteristics (Expressed as mg/L)

Arsenic	Total Hardness as $Ca(CO)_3$
Barium	Cadmium
Chromium	Copper
Lead	Mercury
Selenium	Silver
Zinc	Sulphates as SO_4
Fluoride as F	Chlorides as Cl
Manganese as Mn (dissolved and total)	Conductivity
Iron as Fe (dissolved and total)	Nitrites as NO_2
Total Dissolved Solids	Nitrates as NO_3
total coliform/fecal coliform (bacteria)	

12.2 TEST FOR SAND: After the final well is fully developed and while the development pump is still installed, a test to confirm the actual degree of sand which will infiltrate the well shall be conducted. The well shall be allowed to rest for at least one hour, then pumping shall begin at the full design well yield. Driller shall pump or bail the well until the water pumped from the well shall be substantially free from clay, silt, and sand (< 8.0 mg/l) as measured by an Imhoff cone container or until the water is clear (which shall be determined by the COR. The Imhoff cone container results shall be based on the average results of five samples taken from a large barrel or other tank during the pump test. The samples shall be taken 15 minutes after the start of the pump test, after 25%, 50% and 75% of the pump test duration, and near the end of the pump test. After completion of the well, unless the permanent pump is immediately installed, the Contractor shall cap the well to prevent contamination of the well.

12.3 TEST FOR YIELD AND DRAWDOWN AND RECOVERY: Upon completion of the permanent production well, the Driller shall install the pump with discharge piping of sufficient size and length to conduct the water being pumped to a point of safe discharge and all equipment necessary for measuring the rate of flow and the water level in the well. A continuous 6-hour pumping test shall be conducted with the pumping rate and drawdown recorded at appropriate intervals indicated in Table 6 between 0 min and 360 min.

TABLE 6 Time Intervals for Drawdown and Recovery Readings

0 minutes	120 minutes	660 minutes
2 minutes	150 minutes	720 minutes
4 minutes	180 minutes	780 minutes
6 minutes	210 minutes	840 minutes
8 minutes	240 minutes	900 minutes
10 minutes	270 minutes	960 minutes
15 minutes	300 minutes	1020 minutes
20 minutes	330 minutes	1080 minutes
25 minutes	360 minutes	1140 minutes
30 minutes	390 minutes	1200 minutes
40 minutes	420 minutes	1260 minutes
50 minutes	450 minutes	1320 minutes
60 minutes	480 minutes	1380 minutes
80 minutes	540 minutes	1440 minutes
100 minutes	600 minutes	

Repeat the reading schedule above for duration of recovery portion of the test after the pump is shut off.

13. DISINFECTION: Immediately after the well is completed, unless the permanent pump is ready for installation, the well shall be sterilized by adding chlorine or hypochlorite in such volume and strength and shall be so applied that a concentration of at least 50 ppm shall be obtained in all parts of the well. The chlorine or hypochlorite shall be prepared and introduced into the well in a manner approved by the COR and shall remain in the well for a period of at least twelve hours. Section A1-10 of AWWA Specification A100 describes acceptable methods of sterilization of a well. After the contact period, the well shall be pumped until the residual chlorine content of the well water removed is 1 ppm or less.

14. ABANDONMENT OF WELLS: In the event that the Contractor fails to construct a well of the required capacity, or should he abandon the well because of loss of tools or for any other cause, the Contractor shall fill the entire hole with grout and remove the casing to the satisfaction of the site COR.

15. CLEAN-UP: Upon completion of the well and other incidentals, all debris and surplus materials resulting from the work shall be removed from the job site. The drilling fluid shall be pumped out and properly disposed of and the excavation for the sump backfilled suitable to the site COR.

16. QUALITY CONTROL: The Driller shall establish and maintain quality control for operations under this section to assure compliance with specification requirements and maintain records of his quality control for all materials, equipment, and construction operations, including but not limited to the following:

(a) Protection of existing facilities.
(b) Protection of quality of water.
(c) Drilling, logging, and testing of test holes.
(d) Drilling operations for well.
(e) Setting of casings, screens, and grouting of casings.
(f) Placement of filter material, if used.
(g) Well development.
(h) All testing of finished well.
(i) Well disinfection.
(j) Filling abandoned test hole or well if required.

A copy of these records and tests, as well as records of corrective action taken, shall be maintained by the Driller for review if requested by the COR.

17. SUBMITTALS: Appendix B contains a flow chart of the well construction process and identifies the points where submittals are expected. The following summarizes the submittals previously listed in greater detail in this specification.

(a) Driller Log Submittal.
(b) Test Well Approval Submittal.
(c) Well Screen and Gravel Pack Submittal.
(d) Pump Design Approval Submittal

18. PAYMENT: Payment for the production water well, except test hole (and test well), will be included in the contract lump sum price for "Water Well" which payment shall constitute full compensation for all costs in connection therewith. Payment for test hole and test well, if permanent well cannot be developed, as determined by the Contracting Officer, will be made at the contract unit price per linear foot for "Test Hole (and Test Well)", which payment shall include all labor, mobilization and demobilization of equipment, drilling, testing, materials, and other incidentals necessary for the test hole and (test well). Payment for test hole and (test well) used in the permanent well will be included in the contract lump sum price for "Water Well." Payment for linear footage of hole drilled not resulting in a permanent well will be paid for as linear footage of "Test Hole (and Test Well)."

Appendix B - Water Well Construction Process

	Preliminary Well Development		Production Well Development		Well Pump and Wellhead Completion
1	Pre Construction Meeting	10	Well Installation	19	**Submittal** Pump Design Approval
2	Site Preparation	11	Install Screen and Final Casing	20	Final Well Pump and Discharge Piping Installation
3	Surface Casing Installation	12	Install Gravel Pack, Bentonite Seal and Surface Grout Seal	21	Install pump Water Level Sensor, Piping, etc
4	Pilot Test Hole Drilling	13	Well Development	22	Disinfect Well
5	Geologic Logging	14	Swab and Airlift	23	Well Head Completion Including Building
6	Groundwater Sampling	15	Pump Performance and Sand Tests	24	***Submit*** Well Completion Report
7	Conslutant Determines Well Design	16	Well Completion and Testing	25	***Submit*** Contents of previous Submittals
8	** Submittal** Test Well Approval	17	Aquifer Test Using Test Pump	26	***Submit*** Pump Equipment and Other material and Product Warranties
9	Contractor Reams Pilot Borehole to Final Diameter	18	Analyze Pump Test Data and Calculate Pump duty Point (TDH, Q)		

Appendix C

Examples of Unacceptable and Acceptable Well Construction (Source: Statement of work Military Water Well Construction, Testing and Completion, United States Forces, Afghanistan, 2009)

Figure 4. Example of unacceptable, locally fabricated well screen. Slots were cut into the steel casing using a cutoff wheel or grinder (yellow ovals), creating 0.05% openings per foot. Folded knife is 4-in long.

Figure 5. Example of approved stainless steel slotted screen with threaded connections (folded knife is 4-in long). Minimum opening area is 15% of the screen surface per foot.

Figure 6. PVC Schedule 40 slotted well screens

Figure 6. Unacceptable wellhead completion examples. Upper panel - casing sticks up <1-inch above the floor, has an inadequate sanitary seal, and has inadequate well pad slope and floor drainage. Lower panel – no sanitary seal or well house over well, and the well pad has no slope Both examples are missing totalizers, water sampling ports, water level measuring ports.

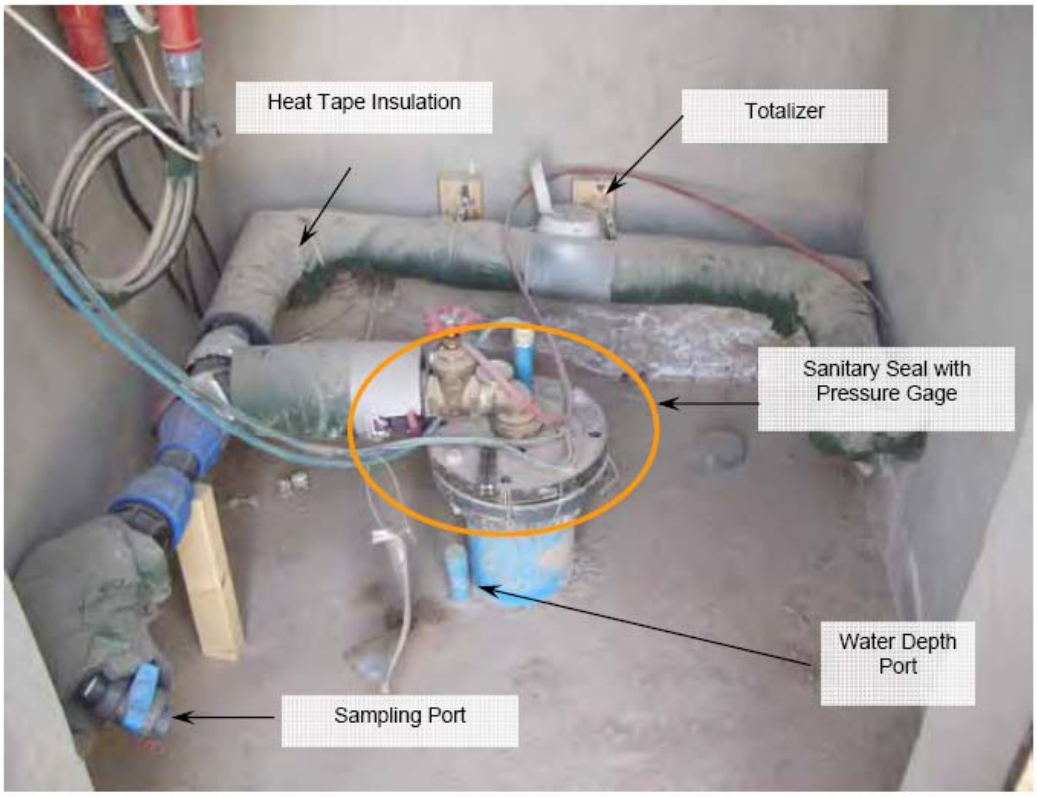


Figure 7. Acceptable wellhead completion, with totalizer (rear center), raw water sample port (lower left), sanitary port for measuring water level (left of 8-in blue well casing), and adequate sanitary seal on well casing (center).

Appendix D

Well Pump Hydraulic Sizing Example

1. General

The purpose of this appendix is to provide design and submittal requirements to contractors for any project requiring well pump design and installation. Water well pump design and selection are covered in Reference 12.

2. Well Data

Submersible well pumps are the most common water well pumps installed at USACE-AED project sites. For submersible pumps the drawdown depth below grade (BG) must be first determined from pumping tests at the rate or (if step drawdown test) within the range of the pumping rates used in the test.

3. Pump Curve Selection

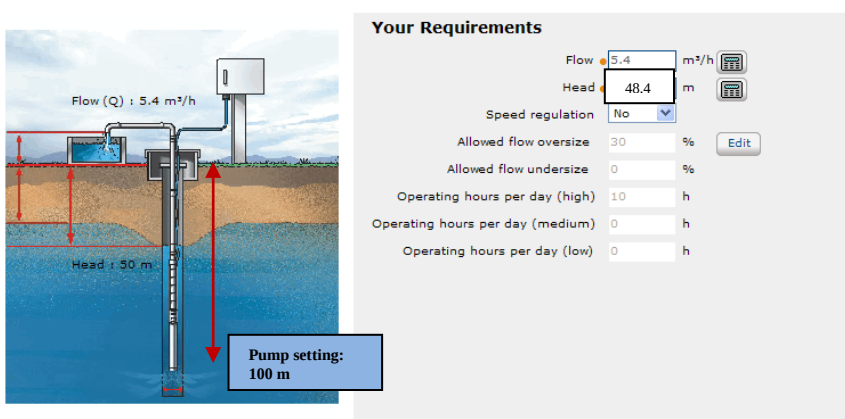
a) System Head Curve. The first step in the pump selection process is to determine the region of operating head curve for the pump piping system. The head curve is determined from the total dynamic head (TDH) required in the piping system at the design flow rate. The TDH is the total discharge head (h_d) minus the total suction head (h_s). For submersible pumps, h_s is a positive value, in other words the level of the dynamic water level above the pump intake is subtracted instead of added from the total discharge head to obtain TDH:

$$TDH=h_d-(h_s)$$

The total discharge head (h_d) is the sum of the static discharge head (h_{sd}), the discharge velocity head (h_{vd}) at the pump discharge flange and the friction head (h_{fd}) in the discharge line.

$$h_d=h_{sd}+h_{vd}+h_{fd}$$

A sketch of the well installation is required to be submitted in the design analysis showing information similar to the figure to the right.



In this example, the pump setting (h_{sd}) is 100 m (BG), the dynamic drawdown is 50 m, and therefore h_s is equal to -50m. The well pump discharge pipe diameter is 50 mm (2 in) and the design flow is 1.5 l/s (5.4 m^3/h) based on 10 hours per day average operation time. Velocity head, h_{vd} is estimated at 0.002 m, and friction loss, h_{fd}, for 125 m of discharge piping (to the tank inlet) is approximately 1.5 m. Therefore h_d=100-0.002-1.6=98.398 m, TDH = 98.398-50=48.398, say 48.4 m.

Velocity head is calculated as shown by the following equation.

$$h_v=v^2/2*g_c$$

Where:

v=Q/A=flow velocity at the discharge of the pump (m/sec)
g_c=gravitational constant=9.81 m/sec^2
Q=flow (m^3/sec)
A= pipe area (m^2) =π*(D^2/4)
D= pipe diameter (m)

Discharge friction head (h_{fd}) is the head required to overcome resistance to flow in the pipe and fittings. It is dependent upon the size, condition and type of pipe, number and type of pipe fittings, flow rate and nature of the liquid is calculated by the following equation.

$$h_f=L[(v/(kC))*(4/D)^{0.63}]^{1/0.54}$$

Where:

L=equivalent length of pipe and fittings (m)
v=flow velocity in pipe (m/sec)
k=0.85.
C=Hazen-Williams coefficient, assumed 150 for PVC and new SCH 40 steel pipe
D=pipe diameter (m)

The discharge friction head (h_{fd}) will need to be calculated for specific flow rates from the pump.

b) Pump Power

Pump brake power is calculated using the following equation:

$$P=(HQ)/(102*e)$$

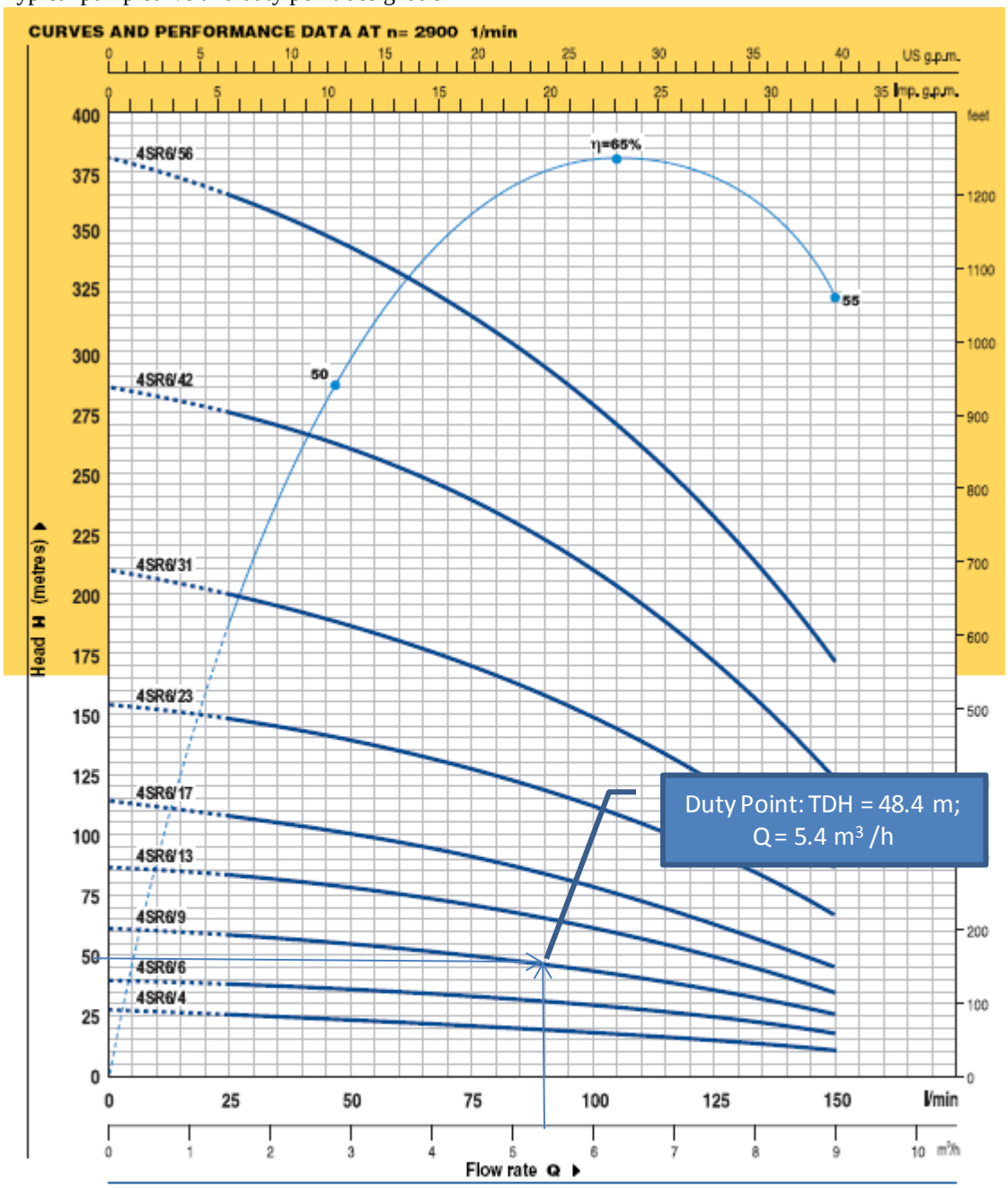
Where:

P=break power required, kW
H=total dynamic head, m
Q=volume of water discharged, L/s
e=Combined efficiency of pump and motor, from manufacturer's data

For this example given here the brake power required is 1.1 kW, based on 1.5 L/s discharge rate, 48.4 meter TDH, and efficiency of 0.64.

The pump curve is shown in the next figure along with the duty point calculated for the well pump installation. Because the well pump rate and drawdown obtained during the pump test are interrelated, it is important that the well pump be selected based on actual data obtained from tests and not assumed from hypothetical calculations of drawdown. Furthermore, the pump motor power must be limited that required to produce the pump test flow rate. Larger pumps than required may result in the pump drawing down the well significantly farther than occurred during the test; increased entrance velocities through the screen can occur resulting in clogging the screen and ultimately destroying the well capacity, and leading to pump motor failure.

Typical pump curve and duty point designation



TYPE		POWER		Q m³/h	0	1.5	3.0	4.5	6.0	7.5	9.0
Single-phase	Three-phase	kW	HP	Q l/min	0	25	50	75	100	125	150
4SR6m/4	4SR6/4	0.55	0.75		27	26	24	22	19	15	11
4SR6m/6	4SR6/6	0.75	1		40	38	36	33	29	24	17
4SR6m/9	4SR6/9	1.1	1.5		61	58	54	50	44	35	26
4SR6m/13	4SR6/13	1.5	2		87	83	78	71	61	49	35
4SR6m/17	4SR6/17	2.2	3	H metres	114	107	100	91	79	62	45
—	4SR6/23	3	4		154	148	138	128	112	92	67
—	4SR6/31	4	5.5		210	200	186	170	149	121	86
—	4SR6/42	5.5	7.5		285	276	258	240	212	170	124
—	4SR6/56	7.5	10		390	365	340	315	280	233	173

Q = Flow rate H = Total manometric head

Tolerance of the performance curves according to EN ISO 9906 App. A.

GUIDE SPECIFICATION FOR DRINKING WATER WELLS

VARIOUS LOCATIONS, AFGHANISTAN

1.0 GENERAL

This specification is to be used in the design and construction of the new water well at ?????????????. Additional guidance for information contained in this specification can be found in the document entitled AED Design Requirements – Well Pumps and Well Design – Sep 09. The publications listed below form a part of this specification to the extent referenced. The publications are referred to in the text by the basic designation only.

1.1 LOCATION AND DEPTH OF WELL

The test well to be constructed shall be located as shown on the drawings, or where directed by the Contracting Officer (CO) or his representative (COR). The well shall be to such depth as may be necessary to penetrate a desirable water-bearing stratum. The minimum well depth for each well shall be 20 meters below the static water table. Additional depth to be determined based on the experience of the driller and similar wells in the area may be required by estimating the additional depth that may be required to provide a minimum 10 meters of submergence over the well pump during the dynamic drawdown depth of water during pumping. It may be necessary for the well depth to penetrate the full thickness of the aquifer to bedrock to achieve sufficient depth.

1.2 LOCAL CONDITIONS

Hydrogeological and water-well information for the immediate vicinity should be obtained locally from private and/or government organizations. The driller should drill test hole as hereinafter specified to assure the water availability of the aquifer. The method of drilling and the type of well construction may vary depending on the local geology. Wells completed into unconsolidated aquifers may need to be screened depending upon the type of aquifer material or if completed into consolidated (rock) aquifers, open-hole construction may be approved.

2.0 PROTECTION OF EXISTING FACILITIES

The existing facilities such as building structures, utilities, walks, trees, etc., except as otherwise specified in these specifications, shall be protected from damage during construction of the wells, and if damaged, shall be repaired by the Contractor at his expense. Water pumped from the well shall be conducted to a place where it will be possible to dispose of the water without damage to property or the creation of a nuisance.

3.0 PROTECTION OF QUALITY OF WATER

The Driller shall take all necessary precautions during construction to prevent contaminated water, gasoline or other contaminated materials from entering the well either through the opening or by seepage through the ground surface. The Driller shall exercise extreme care in performance of his work in order to prevent the breakdown or caving of the strata overlying that from which the water is to be drawn.

4.0 TEST WELL DEVELOPMENT

4.1 GENERAL

Before permanent well construction, at least one test well of at least 150mm in diameter and minimum 20 meters below the static water table shall be drilled into the water bearing stratum or the top of bedrock. A GPS instrument will be used to determine the geographic coordinates of the well. This information shall meet requirements of the World Geodetic System 1984 (WGS 84 UTM Zone 42) in decimal degrees. The test hole shall be used to determine the location and character of the water bearing strata and to obtain samples of the various formations. Samples of drilling cuttings shall be taken at every change of strata and at depth intervals not to exceed 1.5 meters. A split spoon sample shall be taken at each major change in strata as indicated by the nature of the cutting samples. A driller's log shall be made based on the cuttings obtained. The final selection of the screen settings, the proper gravel filter pack material, and the depth of grouting will submitted to the COR for approval prior to any additional work. The drill cuttings shall be divided, put into suitable containers and labeled. These samples shall be approximately half a liter each. If the test hole fails to indicate the presence of water bearing strata or is abandoned for any other reason, the test hole shall be grouted from the bottom to the top with cement grout as hereinafter specified and in a manner approved by the COR. At the completion of the test hole, a drillers log shall be prepared containing the following information that shall be included in the submittals:

4.2 DRILLER'S LOG CONTENTS

a) Depth of water strata
b) Depth of different material strata contacts
c) Color, size, and soil description of cuttings
d) Penetration rate (meters per day)
e) Types and amount of drilling fluid gain or loss
f) Type schedule and length of well casing

4.3 TEST WELL PUMPING

To determine the expected yield from the well and to assure acceptable water quality, a pump test shall be performed in the candidate test well. A temporary casing and screen, if required, shall be used to construct the test well for completion of a 4-hour capacity test using a temporary pump with the capacity to pump the required average daily demand supplied by the pump. The static water level in the well should be measured prior to installing the pump and the water level at the end of the pumping period should be measured. The temporary casing shall be a minimum diameter of 150mm and extend to the top of the water-bearing strata being tested in unconsolidated formations. The Contractor shall isolate other water bearing strata present from discharging into the test well. After completion of the test well, all data pertaining to the construction of the well shall be shown on a sketch of the test well with all pertinent depths of construction.

4.4 SUBMITTAL FOR APPROVAL OF TEST WELL

Before any further construction in the test well, a copy of the results of the test hole drilling log and test well capacity test results shall be submitted to the COR for approval. Upon approval the permanent well construction may begin. Contents of the test well approval submittal shall include:

(a) Location of well on site plan.

(b) Size of well diameter and depth.

(c) Drilling log showing depths of aquifer strata.

(d) Depth below top of well of static water table.

(e) Casing and screen diameters and lengths installed in the well.

(f) Setting of casings, screens, and grouting of casings.

(g) Placement depths of gravel pack material, if used.

(h) Test well pumping rate, liters per second

(i) Test well drawdown record (see Table 5)

(j) Water quality result (see Table 4)

4.5 ABANDONMENT OF UNAPPROVED WELLS

Test wells not approved will be abandoned in accordance with specifications in Reference 1.4, AWWA A 100.

5.0 WELL CONSTRUCTION

5.1 GENERAL

Well construction shall be based on well design guidelines set by the US Army Corps of Engineers Afghanistan Engineer District Well Pumps and Well Design Guide, September 2009. The execution of the work shall be by a competent crew performed under the direct supervision of an experienced well driller acceptable to the COR. The well shall be drilled essentially straight, plumb and circular from top to bottom. The type of well construction, as determined by the test hole, shall be either an artificially gravel-packed well, a naturally developed well or a rock wall well. The presence of cavities and voids in the formations being penetrated will make the construction of a screened well difficult due to loss of the filter material into the cavity. Grouting may also be affected by these voids. If cavities and voids are encountered in the test hole, the Contractor shall consider this in the design of the completed well and submit this information with the results of the test hole and test well. The well shall be constructed and developed to produce clear water with a minimum of drawdown. Wells not meeting the criteria for water quality, Section 10 WELL DEVELOPMENT, will not be acceptable. Well and casing dimensions shall generally be within the following ranges shown in Table 1.

TABLE 1 Minimum Required Well and Casing Diameters

anticipated water yield, l/s	well casing/ screen ID, mm	well diameter, mm
<6.3	150	200
6.4 to 11.0	200	250
11.1 to 25.25	250	300

5.2 SURFACE CASING

Surface casing shall be used on all wells. This casing may be left in place or removed at the option of the driller. If the surface casing is to be left in place, the surface casing shall be grouted into place for the full length of the casing. In unconsolidated aquifer material, casing shall be extended to the top of the well screen. In wells developed in rock formations, the hole may be left open with the casing extended three meters into the formation. All well surface casing shall be extended 0.5 meters above grade.

5.3 WELL CASING

The well casing shall be of the size hereinafter specified and be constructed either of black steel meeting the AWWA C200 or PVC plastic Schedule 80 meeting ASTM 1785 and ASTM F480 specifications. Only standard weight black steel pipe shall be used as casing for wells over 80 meters deep and it shall conform to ASTM A-53 steel pipe conforming to AWWA Specification C-200, as applicable. The minimum steel casing thickness shall be 8 mm. Steel casing may be provided with drive shoes at the option of the driller. Steel pipe couplings shall be per ASTM F480. PVC casing may be used if well depths are less than 80 meters. The PVC pipe must be at least Schedule 80 or SDR 17. Driving of PVC casing will not be allowed. PVC pipe couplings shall be per ASTM F480. All casing used in the permanent well shall be new and unused. Cast iron pipe shall not be used for well casing or screens. Sufficient casing centralizers shall be used to keep casings centered in the hole. Each section of casing will be joined with standard couplings; full-threaded joints, proper welding or solvent cement PVC welds so the joints are sound and watertight. Well casing alignment shall not interfere with the proper installation and operation of the pump.

5.4 CEMENTING WELL CASING

The annular space between the well casing and the walls of the hole shall be filled with cement grout or crushed angular gravel as hereinafter specified. The grout shall be proportioned of Portland cement conforming to ASTM Specification C-150, Type I or II and the minimum quantity of water (not over 23 liters per 42.3 kg of cement) required to give a mixture of such consistency that it can be forced through the grout pipes. The method for placing the grout will be by the forcing of the grout from the bottom of the space to be grouted towards the surface. The minimum depth of grouting shall be three meters unless approved in writing by the COR. The grout shall also seal off any other water bearing strata above the zone that is producing the water to the well. The grouting shall be done continuously and in a manner that will insure the entire filling of the annular space in one operation without damaging the well casing. No drilling operations or other work in the wells will be permitted within 24 hours after the grouting operation to allow the grout to properly set. Up to 5% bentonite may be added to the mixture to reduce shrinkage of the grout. The addition of bentonite will require additional water at the ratio of 2.5 liters of water for each percent of bentonite added. The bentonite slurry shall contain at least 20 percent solids by weight by having a density of 4.3 kg/L. Aggregate requirements for sealing the solid wall casing and edge of the borehole area shown below:

Sieve size, mm	Percent total weight passing
12.5	100
4.75	75 +/- 13

5.5 WELL SCREEN

The well screen and attached fittings shall be constructed entirely of corrosion resistant stainless steel, unless otherwise specified. Steel screens shall be made from Type 304 stainless steel meeting the requirements of AWWA A100 because of high chloride concentrations that corrode metals. Screens made of UPVC pipe material shall not be permitted. The screen slot pattern shall be continuous slot, wire wound or slotted PVC pipe design. Slot openings shall be continuous around the screen, spaced to provide open area required for maximum entrance velocity criteria, but be consistent with the strength requirements, and should be V-shaped toward the inside of the screen to reduce clogging. Slot sizes shall be typical sizes per manufacturer specifications (see Table 2). Vertically slotted metal casing may be used in only in lower yielding deep (greater than 300 m) wells where extra pipe strength is required and shall require COR approval. PVC pipe screens shall be manufactured from Schedule 80 PVC pipe. Metal screens shall be used for wells greater than 80 meters in depth. The minimum inside diameter of the well screen shall be 150mm. Minimum total length of PVC screens installed in the permanent well shall be four (4) meters. Minimum total length of steel screens shall be two (2) meters. Well screens shall provide adequate transmitting capacity to limit the entrance velocity to less than 0.03 m/sec (0.1 ft/s). The selected screen slot design shall be based on standard manufacturer specifications and meet the minimum effective open area required for entrance velocity criteria. Well screen design information shall be submitted to the project COR for approval prior to installation. In a gravel pack well, the screen shall have a slot size based on the gradation of the filter material which is indicated in 9.7_ GRAVEL-PACKED WELL. PVC slotted screens shall be machine milled by a recognized screen manufacturer and not hand cut in the field. Field fabricated screens are not permitted. Appendix B shows examples of unacceptable and acceptable well screens. The well screen shall be directly connected to the top of the inner casing. The bottom of the screen shall be sealed with a positive closure. A wash-down shoe may be used if desired. The screen used in a gravel-packed well shall be carefully lowered into the water-bearing strata and be centered in the hole. In a naturally developed well (see 9.6 NATURALLY DEVELOPED WELL), the slot size of the screen will be based on a mechanical sieve size analysis of the natural water bearing sediments. The well screen and all accessories required for satisfactory installation shall be essentially standard products of reliable manufacturers regularly engaged in the production of such equipment. Field welding of screen components shall be accomplished using products made to weld such products together in a reliable manner.

5.5.1 WELL SCREEN SUBMITTAL

Well screen submittal information shall include:

(a) Number of screens and depth of setting in the well.

(b) Size of well screen inside and outside diameters and length.

(c) Pipe material (material schedule and specification).

(d) Standard slot opening, mm

(e) Effective open area of screen (sq cm per meter)

(f) Transmitting capacity (liters/meter)

(g) Sieve analysis of the material to be screened

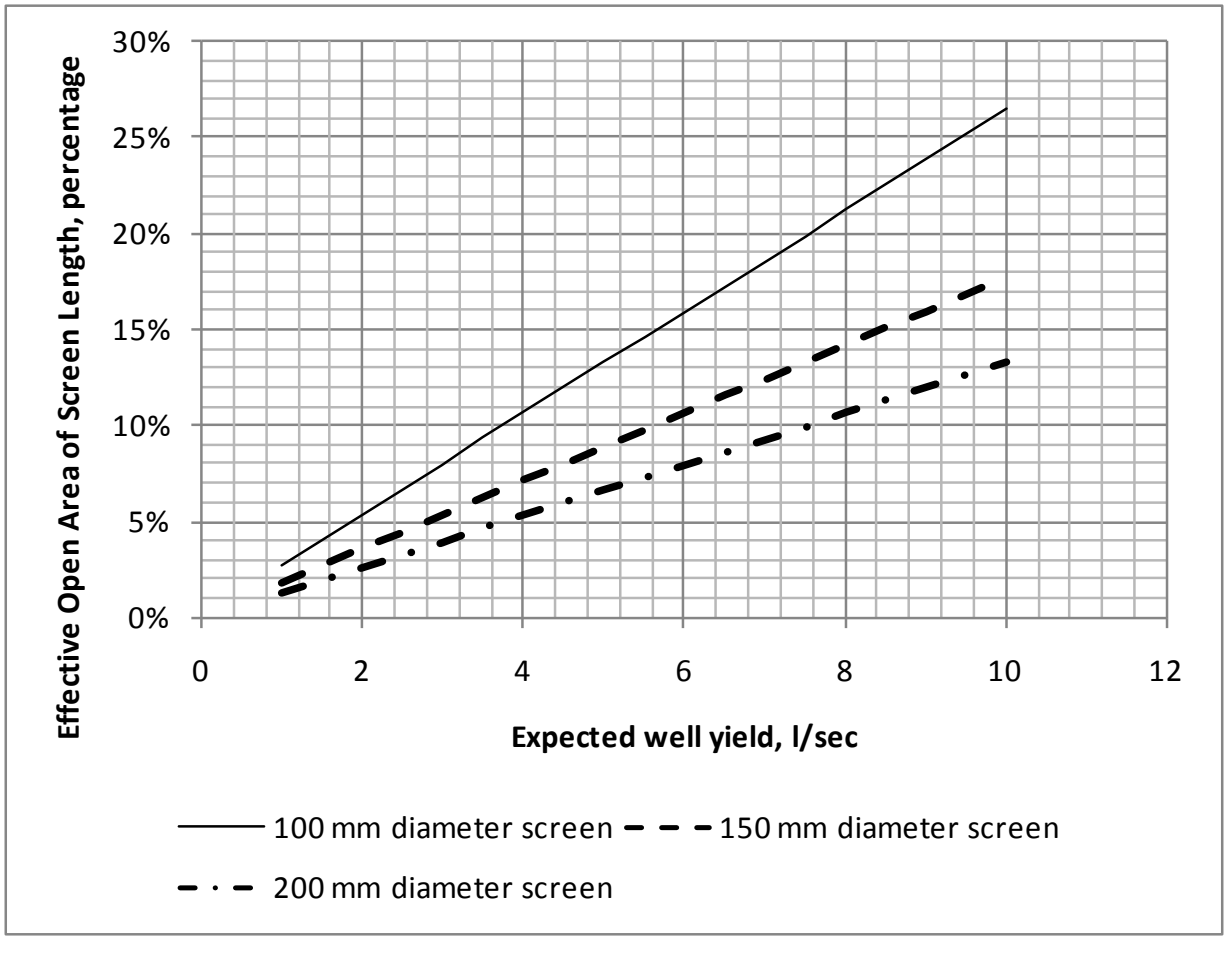
(h) Calculations supporting screen slot size

(i) Photo of screen slot pattern

TABLE 2 Typical Effective Open Area for Slotted Well Screens

Nominal Well Screen Diameter	Screen Slot Size		Steel Continuous Slot		PVC Continuous Slot		PVC Slotted Pipe	
mm	No	mm	cm2/m	%	cm2/m	%	cm2/m	%
100	20	0.508	931	25	-	-	-	-
100	60	1.524	1,905	52	1,100	30	381	11
150	30	0.762	1,693	25	1,206	18	550	8
150	60	1.524	2,857	41	1,968	29	995	14
150	95	2.413	3,492	51	-	-	-	-
200	30	0.762	1,629	16	-	-	-	-
200	60	1.524	2,857	28	-	-	-	-
200	95	2.413	3,851	38	-	-	-	-
200	125	3.175	4,529	45	-	-	-	-

Figure 1 Minimum Effective Open Area versus Well Yield



6.0 WELL TYPES

6.1 NATURALLY DEVELOPED WELL

In water bearing coarse grained material defined as gravel, cobbles, and conglomerate more water can be obtained by natural development of the permeable zone around the well screen than using a gravel pack. After setting the surface casing, a naturally developed well shall be initially drilled by reaming the test hole from the ground surface to the lower level of the water-bearing strata. The well casing and screen shall not be less than 150mm in diameter. The hole shall be of sufficient size to leave a concentric annular space of not less than 50mm between the outside of the casing and the walls of the hole. A grouting basket shall be used to keep the grout from invading the zone around the well screen. The space around the casing shall be filled with cement grout as hereinbefore specified. After grouting is completed, drilling operations shall not be resumed for at least 24 hours to allow proper setting of the grout. After the grout has set, the well shall be developed by pumping or bailing until the fine grained material is removed.

6.2 GRAVEL-PACKED WELL

6.2.1 GENERAL

Gravel-packed wells shall be constructed in fine grained aquifer materials consisting of silty sand, sand and fine gravel (less than material 3 mm in size). The space around the well screen shall be filled with gravel pack material as described hereafter; either gradation A or gradation B shall be used based on a mechanical sieve size analysis of the natural water bearing sediments. The term “gravel-packed well” does not actually mean that the filter material is always gravel sized material. The filter material may be sand sized. The gradation depends upon the size of the aquifer material that yields the water. Coarse grain aquifers shall be considered to be any water bearing stratum that has more than 50 percent of the individual particle sizes greater than 6 mm in diameter. Aquifers with greater proportion of fine sand and silt (less than 6 mm size) shall be considered fine grained aquifers. After setting the surface casing, a gravel-packed well shall be initially drilled by reaming the test hole from the ground surface to the lower-most level of the water bearing strata. The casing and screen shall be as herein specified and shall be not less than 150mm in diameter. The hole shall be of sufficient size to leave a concentric annular space of not less than 50mm between the outside of the screen and casing and the walls of the hole. The gravel pack shall be 1.5 times the length of the screen section it surrounds. For example if the screen is 3 meters in length the gravel pack shall be 4.5 meters in length straddling the screen on the top and bottom. The space around the screen shall be filled with filter material as hereinafter specified. The well casing shall be grouted in with cement grout as hereinafter specified. After grouting is completed, drilling operations shall not be resumed for at least 24 hours to allow proper setting of the grout. The hole below the outer casing shall penetrate the water bearing strata a sufficient depth to install the well screen and produce the required yield without causing excessive velocities through the well screen. The casing shall be connected directly to the top of the well screen and extend up to one foot above the ground surface.

6.2.2 GRAVEL-PACK

After the screen and casing have been installed, filter material shall be installed around the screen by filling the entire space between the screen and the walls of the hole with filter material. The filter shall have a wall thickness of 50mm, measured from the outer edge of the screen to the wall of the hole. Filter material shall be of properly sized, graded, well-rounded natural sand and gravel suitable for the strata encountered. Angular aggregate shall not be used. The filter material shall be of such size as will allow the maximum flow of water into the well and prevent the infiltration of sand. It shall be washed siliceous material, reasonably smooth and round and free of flat or elongated pieces as well as dirt, vegetable matter or other foreign matter. The Driller shall demonstrate to the COR that the filter material is suitable for the conditions prior to placement and submit a gradation of the material for approval. In no case will improperly sized filter material be added around the screen.

Gravel pack gradations shall be based on the aquifer particle size. Silty sand particle size aquifers shall use Gradation A (Table 3), and gravel and larger particle size material shall use Gradation B (Table 4).

TABLE 3 Gradation A Gravel Pack Materials

Material for gravel pack in silty sand aquifers - Shall consist of stone containing rounded shapes and surfaces with no flat surfaces having the following gradation:

Sieve Size	% Total Wt. Passing
12.5 mm	100
4.75 mm	75 +/- 13
1.18 mm	25 +/- 15
0.425mm	8 +/- 4

TABLE 4 Gradation B Gravel Pack Materials

Material for gravel pack in sandy gravel including cobbles - Shall consist of stone containing rounded shapes and surfaces with no flat surfaces have the following gradation:

Sieve Size	% Total Wt. Passing
40 mm	100
32 mm	75 +/- 13
16 mm	25 +/- 15
9 mm	8 +/- 4

All aggregate shall contain less than 5% by weight rock powder, silt, clay, shale, clay lumps, coal, lignite, soft stone, or other deleterious materials.

6.2.3 PIPE OR CONDUCTOR FOR FILTER PLACEMENT

If possible, a pipe or conductor having an inside nominal diameter of not less than 25mm shall be lowered to the bottom of the well between the drilled hole and the screen. It shall be so arranged and connected at the surface of the ground to water pumping and filter placement equipment so that water and filter material, fed at uniform rates, are discharged through it as the filter fills the hole from the bottom up. The filter sand and water conductor shall be raised at the rate that will keep the bottom of the pipe approximately at the filter material level in the hole.

6.3 ROCK WELL

A rock well shall be initially drilled from the ground surface to a point at least 3 meters below the top of consolidated material (bedrock), but not less than 10 meters below the top of the rock and the bottom of the casing shall be set at this bottom elevation. The finished internal diameter of the casing shall be not less than 150mm, and the hole shall be drilled to a sufficient diameter so as to leave a concentric annular space not less than 37mm (1 1/2-inches) between the casing and the walls of the hole. A temporary casing may be used to prevent caving of the hole walls, but the temporary casing must be removed when the grouting of the permanent casing is performed. This space shall be filled with cement grout in a manner as previously specified. After the grouting is completed, drilling operations shall not be resumed for at least 24 hours to allow proper setting of the grout. Drilling into the water-bearing rock strata shall be resumed after the grout has set. A hole at least 150mm in diameter, concentric with the casing above,

shall be drilled into the water-bearing rock a sufficient depth to produce the required amount of water without causing excessive velocities of the water through the rock.

7.0 WELL DEVELOPMENT

After completion, the well shall be thoroughly developed. The developing equipment shall be of sufficient capacity to remove all drilling fluids, sand, rock cuttings or any other foreign matter. The wells shall be thoroughly cleaned from top to bottom before beginning the well tests. The recommended type of development for stainless steel screened wells is hydro-jetting; however surge blocks, air-development or other development techniques are permissible at the option of the Contractor. The well shall be disinfected before removing the test pump and collecting samples for determining the water quality (see

8.0 WELL PUMPS

8.1 PERMANENT WELL PUMP

The pump and motor diameter shall be at least 25 mm smaller than the inside diameter of the well screen or casing, whichever is smaller, in order to allow it to be removed for servicing after the buildup of scale on the outside of the pump and inside of the screen and casing. Before any installation of the permanent pump in the well, a pump design approval shall be submitted to the COR for approval.

8.2 SUBMITTAL

Pump design approval submittals shall include the following information:

(a) Capacity of well from test well submittal.

(b) Size of permanent well diameter and depth.

(c) Depth of static water level below top of well.

(d) Power source and pump motor electrical power requirements.

(e) Expected well draw down.

(f) Pump discharge piping diameter, length, fittings and appurtenances (valves, meters, etc.)

(g) System total dynamic head required by well pump.

(h) Pump design discharge rate.

(i) Selected pump curve and duty point from manufacturer.

(j) Pump/motor product material specifications from manufacturer.

8.3 HAND PUMP

A standard hand pump with seal and air gap shall be installed on the permanent well discharge piping to discharge at concrete pad around well if required by the contract technical requirements. The hand pump shall be capable of pumping at a minimum pressure head of 138 Kpa (20 psi) in the event there is either a loss of power supply or a pump failure in the water well system.

9.0 WELL HEAD COMPLETION

A stainless steel check valve (if not on the pump discharge piping), isolation valve, sampling port, and well discharge tantalizer shall be installed on the pump discharge pipe line, as minimum appurtenances, between the well head and the water storage treatment tanks. A water level measurement port shall be provided at the well head. Additional appurtenances such as flow meter, air release valve, and chlorination treatment piping may be required depending upon the contract technical requirements. Any structure built over the well must have a large door in the roof that allows for easy removal of the pump piping and pump using an overhead crane. Examples of well head construction are shown in Append B.

10.0 TESTS

10.1 TEST FOR QUALITY OF WATER

During the testing of the test well and again during the yield and drawdown test in the permanent well, the Contractor shall schedule to obtain a preliminary sample of the water in suitable containers and of sufficient quantity to have bacterial, physical and chemical analyses made in accordance with the following Table 5 to determine if the water is potable. The word "potable" for purpose of this contract is further defined as water that is suitable for drinking by the public, i.e., good, clear water free from objectionable amounts of harmful bacteria and chemical and physical properties, as defined by References 1.1 or 1.2. Sampling shall be performed by qualified personnel who must obtain sampling kits and schedule their site visit to obtain the samples as directed. The coordination with the COR for the sampling and analysis should begin at the beginning of the contractual period.

TABLE 5

WATER QUALITY ANALYSIS TABLE

<u>Physical Characteristics</u>	
Color	Temperature
Threshold odor number	pH value
Turbidity	
<u>Chemical Characteristics (Expressed as mg/L)</u>	
Arsenic	Total Hardness as $Ca(CO)_3$
Barium	Cadmium
Chromium	Copper
Lead	Mercury
Selenium	Silver
Zinc	Sulphates as SO_4
Fluoride as F	Chlorides as Cl
Manganese as Mn (dissolved and total)	Conductivity

Iron as Fe (dissolved and total)	Nitrites as NO_2
Total Dissolved Solids	Nitrates as NO_3
total coliform/fecal coliform (bacteria)	

10.2 TEST FOR SAND

After the final well is fully developed and while the development pump is still installed, a test for sand shall be conducted. The well shall be allowed to rest for at least one hour, then pumping shall begin at the full design well yield. Driller shall pump or bail the well until the water pumped from the well shall be substantially free from clay, silt, and sand (< 8.0 mg/l) as measured by an Imhoff cone container or until the water is clear (which shall be determined by the COR. The Imhoff cone container results shall be based on the average results of five samples taken from a large barrel or other tank during the pump test. The samples shall be taken 15 minutes after the start of the pump test, after 25%, 50% and 75% of the pump test duration, and near the end of the pump test. After completion of the well, unless the permanent pump is immediately installed, the Contractor shall cap the well to prevent contamination of the well.

10.3 TEST FOR YIELD AND DRAWDOWN

Upon completion of the permanent production well, the Driller shall install the pump with discharge piping of sufficient size and length to conduct the water being pumped to a point of safe discharge and all equipment necessary for measuring the rate of flow and the water level in the well. A continuous 6-hour pumping test shall be conducted with the pumping rate and drawdown recorded approximately at the intervals indicated in Table 6.

TABLE 6 Time Intervals for Drawdown and Recovery Readings

½ minute
1 minute
2 minutes
3 minutes
4 minutes
5 minutes
7 minutes
10 minutes
15 minutes
20 minutes
30 minutes
60 minutes
120 minutes
240 minutes
360 minutes

Repeat the reading schedule above for duration of recovery portion of the test after the pump is shut off. Monitor the recovery of the well 24-hours after the pump is shut down.

11.0 DISINFECTION

Immediately after the well is completed, unless the permanent pump is ready for installation, the well shall be sterilized by adding chlorine or hypochlorite in such volume and strength and shall be so applied that a concentration of at least 50 ppm shall be obtained in all parts of the well. The chlorine or hypochlorite shall be prepared and introduced into the well in a manner approved by the COR and shall remain in the well for a period of at least twelve hours. Section A1-10 of AWWA Specification A100 describes acceptable methods of sterilization of a well. After the contact period, the well shall be pumped until the residual chlorine content of the well water removed is 1 ppm or less.

12.0 ABANDONMENT OF WELLS

In the event that the Contractor fails to construct a well of the required capacity, or should he abandon the well because of loss of tools or for any other cause, the Contractor shall fill the entire hole with grout and remove the casing to the satisfaction of the site COR.

13.0 CLEAN-UP

Upon completion of the well and other incidentals, all debris and surplus materials resulting from the work shall be removed from the job site. The drilling fluid shall be pumped out and properly disposed of and the excavation for the sump backfilled suitable to the site COR.

14.0 QUALITY CONTROL

The Driller shall establish and maintain quality control for operations under this section to assure compliance with specification requirements and maintain records of his quality control for all materials, equipment, and construction operations, including but not limited to the following:

(a) Protection of existing facilities.

(b) Protection of quality of water.

(c) Drilling, logging, and testing of test holes.

(d) Drilling operations for well.

(e) Setting of casings, screens, and grouting of casings.

(f) Placement of filter material, if used.

(g) Well development.

(h) All testing of finished well.

(i) Well disinfection.

(j) Filling abandoned test hole or well if required.

A copy of these records and tests, as well as records of corrective action taken, shall be maintained by the Driller for review if requested by the COR.

15.0 SUBMITTALS

Attachment A contains a flow chart of the drilling process and identifies the points where submittals are expected. The following summarizes the submittals previously listed in greater detail in this specification.

(a) Test well approval.

(b) Well screen approval and gravel pack gradation.

(c) Pump design approval.

(d) Drilling operations for well.

16.0 PAYMENT

Payment for the production water well, except test hole (and test well), will be included in the contract lump sum price for "Water Well" which payment shall constitute full compensation for all costs in connection therewith. Payment for test hole and test well, if permanent well cannot be developed, as determined by the Contracting Officer, will be made at the contract unit price per linear foot for "Test Hole (and Test Well)", which payment shall include all labor, mobilization and demobilization of equipment, drilling, testing, materials, and other incidentals necessary for the test hole and (test well). Payment for test hole and (test well) used in the permanent well will be included in the contract lump sum price for "Water Well." Payment for linear footage of hole drilled not resulting in a permanent well will be paid for as linear footage of "Test Hole (and Test Well)."

Attachment A - Water Well Construction Process

	Preliminary Well Development		Production Well Development		Well Pump and Wellhead Completion
1	Pre Construction Meeting	10	Well Installation	19	***Submit*** Test Data and Pump Selection for Approval
2	Site Preparation	11	Install Screen and Final Casing	20	Final Well Pump and Discharge Piping Installation
3	Surface Casing Installation	12	Install Gravel Pack, Bentonite Seal and Surface Grout Seal	21	Install pump Water Level Sensor, Piping, etc
4	Pilot Test Hole Drilling	13	Well Development	22	Disinfect Well
5	Geologic Logging	14	Swab and Airlift	23	Well Head Completion Including Building
6	Groundwater Sampling	15	Pump Performance and Sand Tests	24	***Submit*** Well Completion Report
7	Conslutant Determines Well Design	16	Well Completion and Testing	24	***Submit*** Contents of previous Submittals
8	*** Submit*** Well Design	17	Aquifer Test Using Test Pump	26	***Submit*** Pump Equipment and Other material and Product Warranties
9	Contractor Reams Pilot Borehole to Final Diameter	18	Analyze Pump Test Data and Calculate Pump duty Point (TDH, Q)		

Attachment B

Examples of Unacceptable and Acceptable Well Construction

(Source: Statement of work Military Water Well Construction, Testing and Completion, United States Forces, Afghanistan, 2009)

Figure 4. Example of unacceptable, locally fabricated well screen. Slots were cut into the steel casing using a cutoff wheel or grinder (yellow ovals), creating 0.05% openings per foot. Folded knife is 4-in long.

Figure 5. Example of approved stainless steel slotted screen with threaded connections (folded knife is 4-in long). Minimum opening area is 15% of the screen surface per foot.

Figure 6. PVC Schedule 40 slotted well screens

Figure 6. Unacceptable wellhead completion examples. Upper panel - casing sticks up <1-inch above the floor, has an inadequate sanitary seal, and has inadequate well pad slope and floor drainage. Lower panel – no sanitary seal or well house over well, and the well pad has no slope Both examples are missing totalizers, water sampling ports, water level measuring ports.

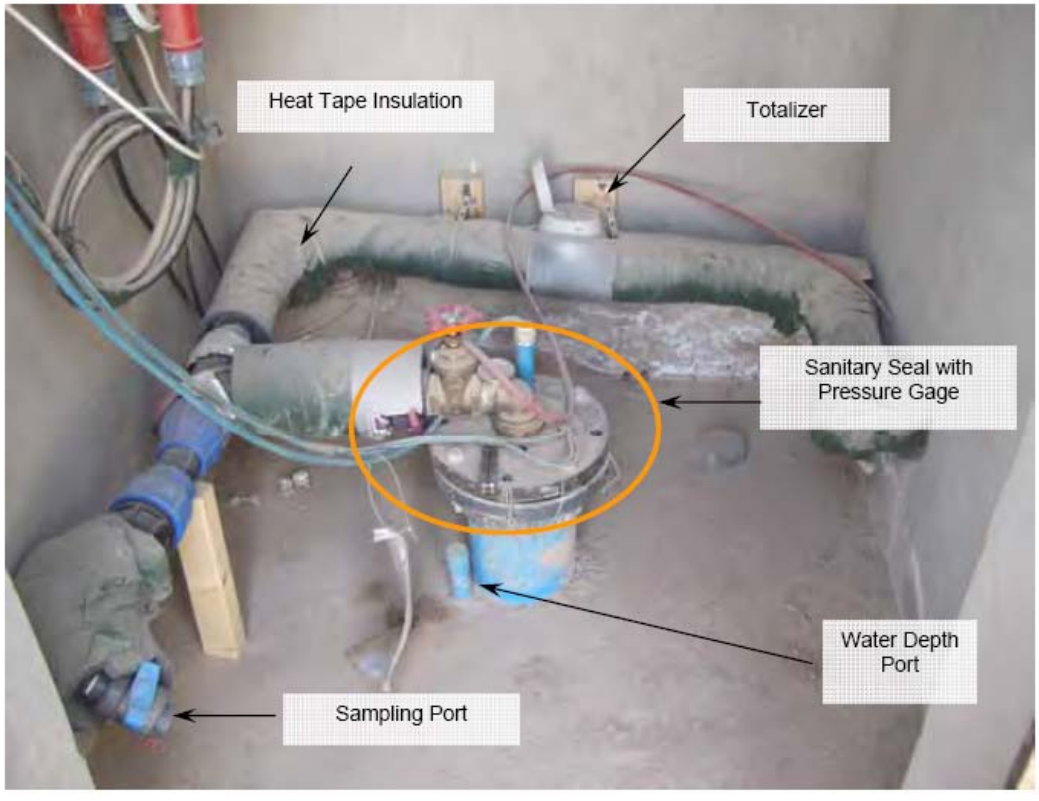


Figure 7. Acceptable wellhead completion, with totalizer (rear center), raw water sample port (lower left), sanitary port for measuring water level (left of 8-in blue well casing), and adequate sanitary seal on well casing (center).