US Army Corps
of Engineers
Afghanistan Engineer District

AED Design Requirements: Sanitary Sewer & Septic System

Various Locations,
Afghanistan

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TABLE OF CONTENTS

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

Section	**Page**
1. General	**5**
2. Field Investigations	**5**
a) Site Survey	5
b) Percolation testing	5
3. Sanitary Sewer System	**8**
a) Sanitary Sewer System Layout	8
b) Protection of Water Supplies	8
c) Quantity of Wastewater	9
d) Gravity Sewers	9
e) Pipe, Fittings and Connections	12
f) Manholes	12
g) Building Connections and Service Lines	12
h) Cleanouts	12
i) Grease Interceptors	12
j) Oil Water Separators	13
k) Field Tests and Inspections	13
4. Septic System	**13**
a) General	13
b) Septic Tank	13
c) Absorption Field	15
d) Pressure Dosing of Leach Fields	17
e) Leaching Chambers	29
f) Seepage Pits	30
5. Design Submittal Information	**32**
6. As-Builts	**33**

Tables **Page**

Table 1. Minimum Slopes for Sewers 11

Table 2. Soil Treatment Areas in Square Meters 17

Table A-1. Orifice Diameter 24

Table A-2. Friction loss in plastic pipe 26

Table A-3. Friction losses through plastic fittings 26

Table A-4. Void Volume for various diameter pipes 27

Table 7-6. Sidewall Areas of Circular Seepage Pits 32

Figures **Page**

Figure 1. Percolation Testing 7

Figure 2. Schematic Pipe Definitions 11

Figure 3. Pressure Distribution System 19

Figure 4. Typical Pressurized Flow Distribution Device 20

Figure 5. Typical Flow Distribution Valve 20

Figure 6. Orifice Spacing 23

Figure 7. Example of a Dosing Tank 28

Figure 8. Example of Leaching Chamber Trench Installation 29

Figure 9. Example of Leaching Chamber Trench Installation 30

Figure 10. Seepage Pit Cross Section 31

Examples **Page**

Example 1: Calculating Percolation Rates 6

Example 2: Size of a Septic Tank 14

Example 3: Size of an Absorption Field 16

Example 4: Size a Pressure Distribution Network 22

Example 5: Size of a Seepage Pit 32

Appendix

Appendix A - Example Gravity Sewer Calculation

Appendix B – Drawing Details

Septic Tank Details

Absorption Bed and Trench Details

Dosing system Layout

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

Leaching Chambers Option 2

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:

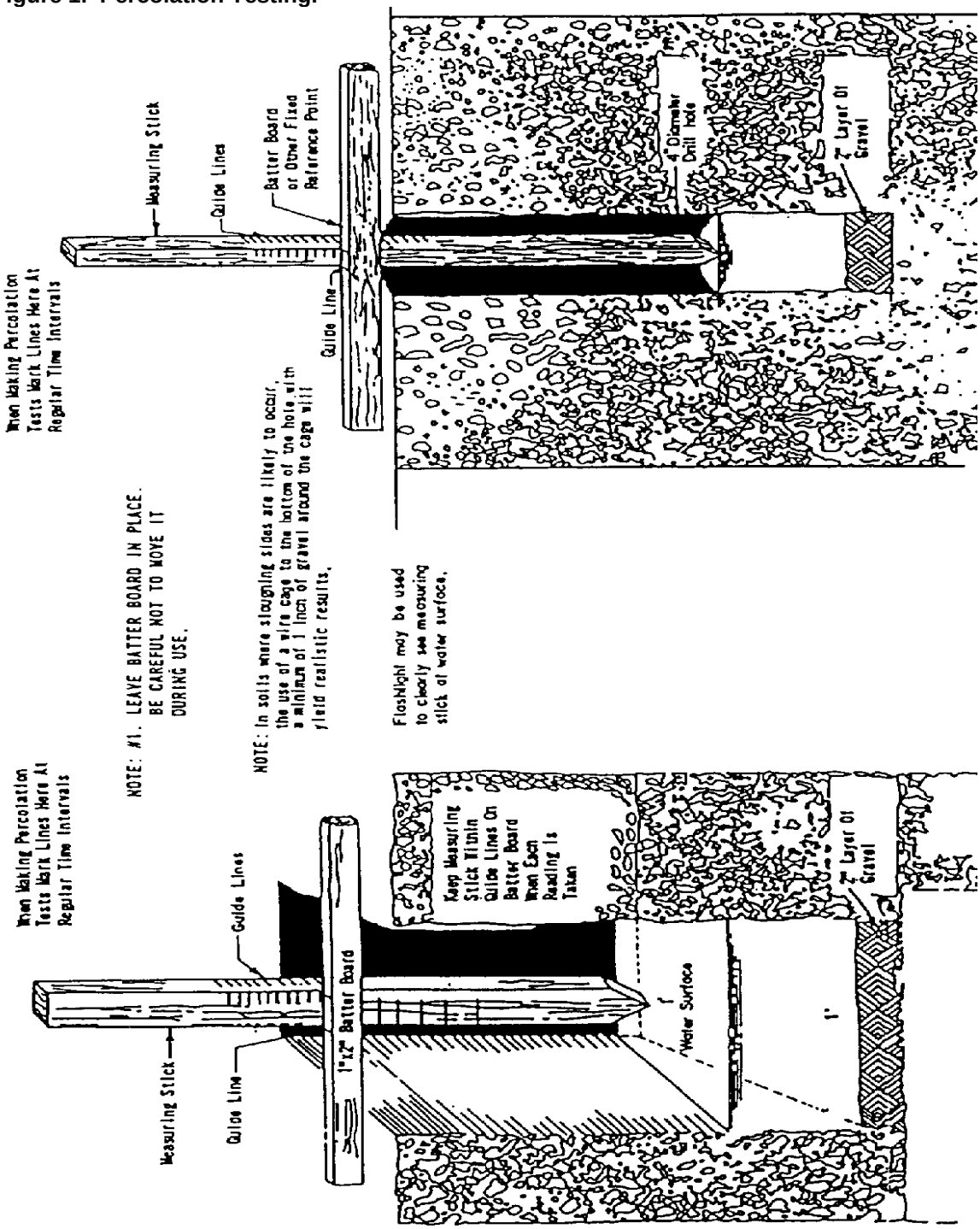
Example 1: Calculating Percolation Rates - In 30 minutes, the measured drop in the water level is 15 mm.

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.

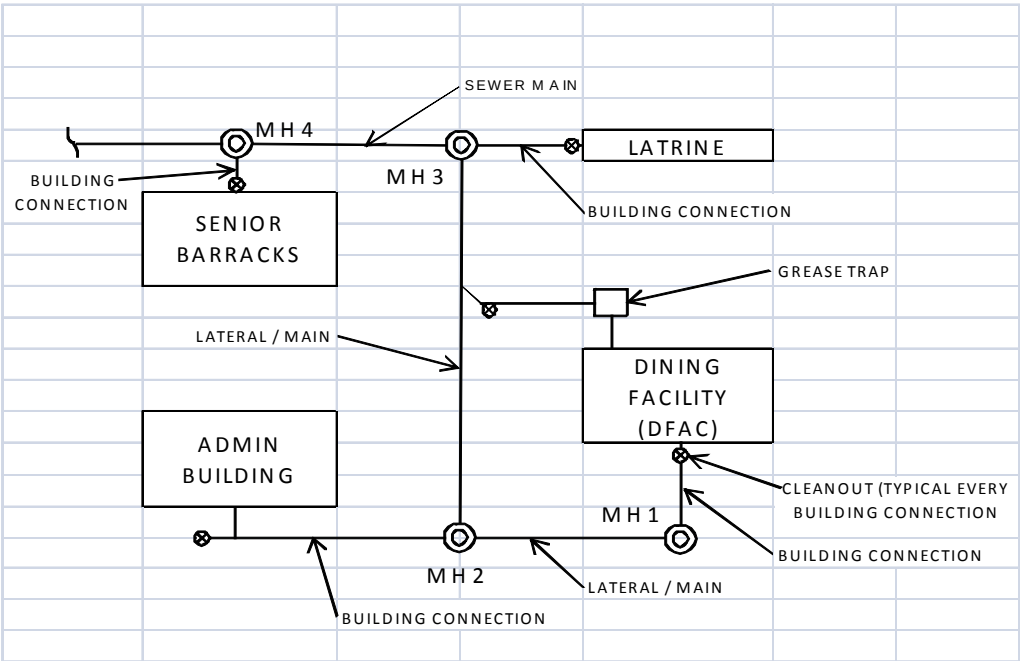


Figure 2. Schematic Pipe Definitions

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.

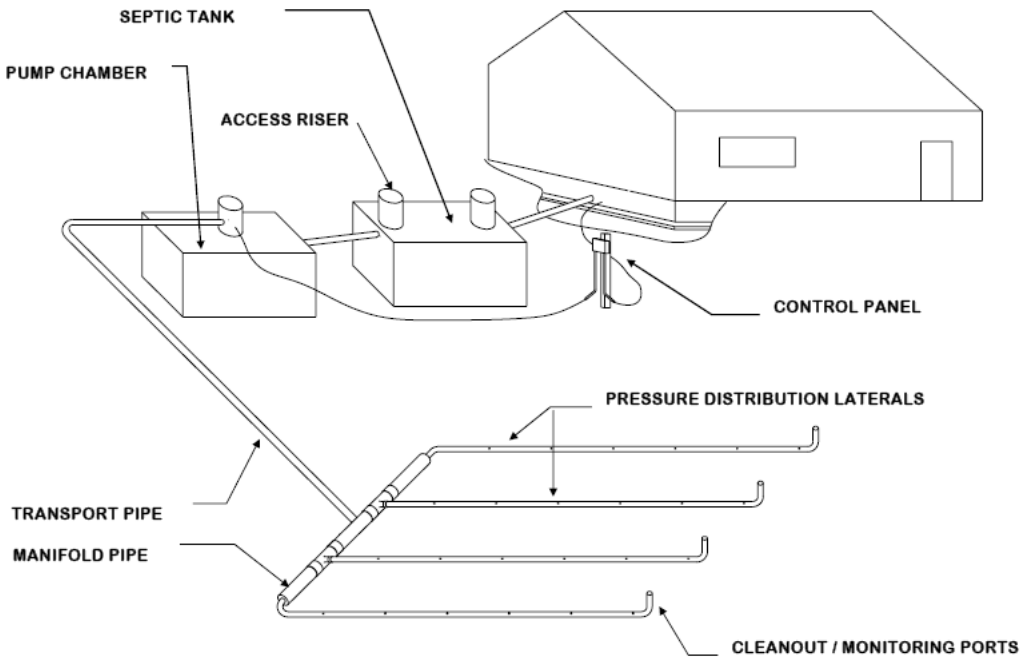
7) Manifold Pipe - The manifold pipe is located between the transport pipe and the laterals in the leach field. This pipe is sized so that there is no more than a 15% variation in the rates of discharge between the first and last orifices in the network. The PVC pipe shall conform to ASTM D2241 PVC Pressure-Rated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

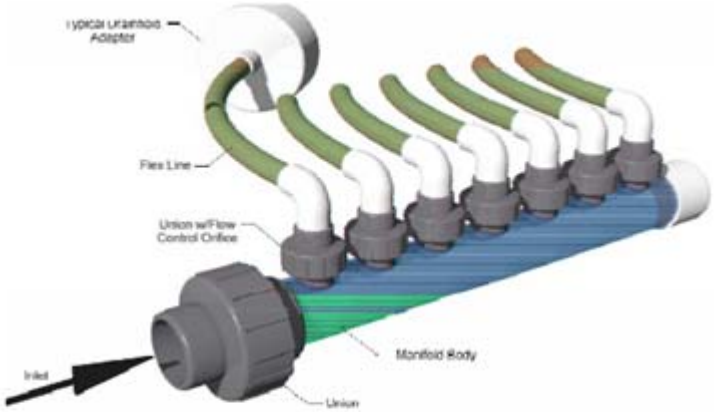
   Clean outs - Clean outs shall be placed at the ends of the laterals, a minimum of 0.5 meter above ground. They should have treaded caps at their ends to allow for inspection and cleaning of the laterals. In cold climates they should be insulated to prevent the laterals from freezing.

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

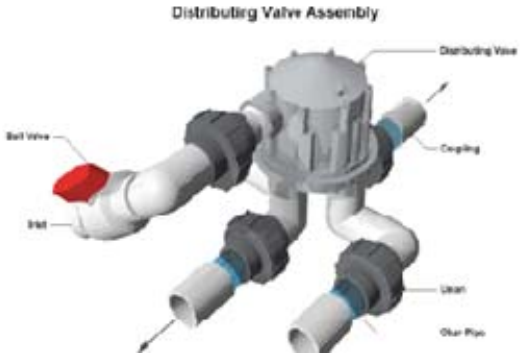
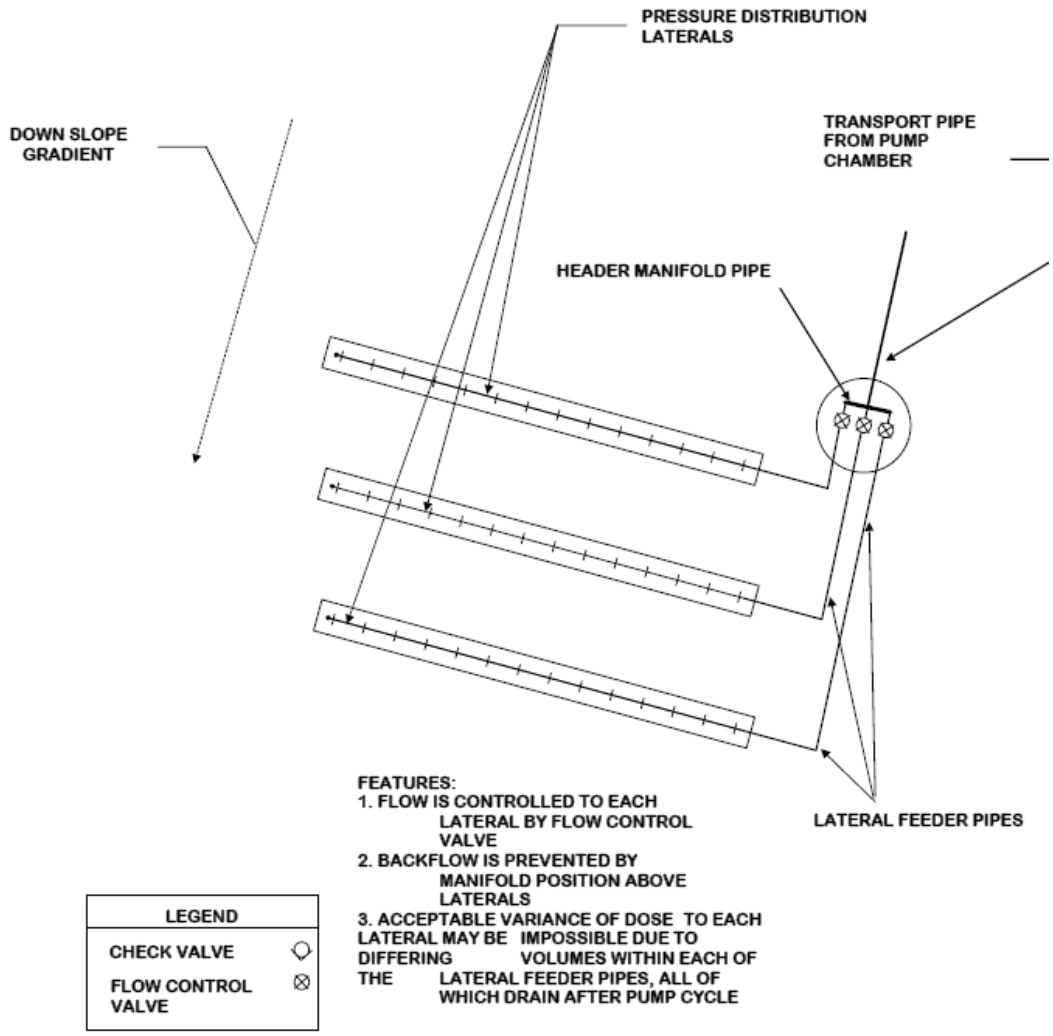


Figure 5. Typical Flow Distribution Valve

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.

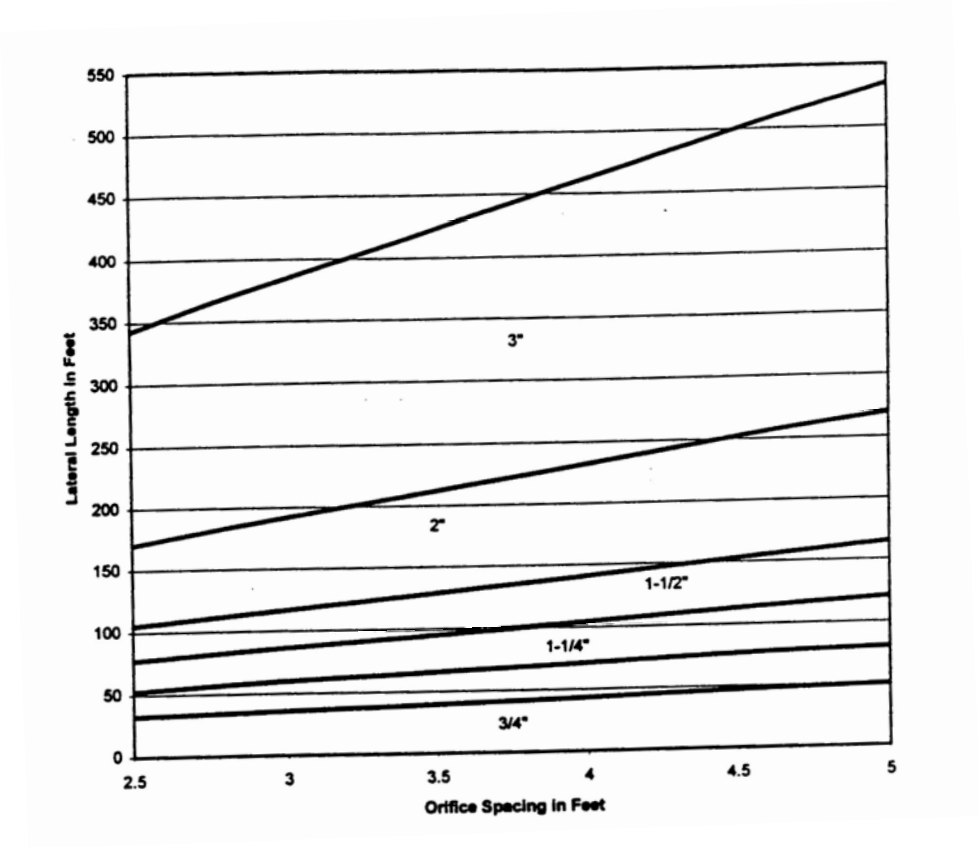
Spacing = (area/orifice x number of laterals / (absorption area width).

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.

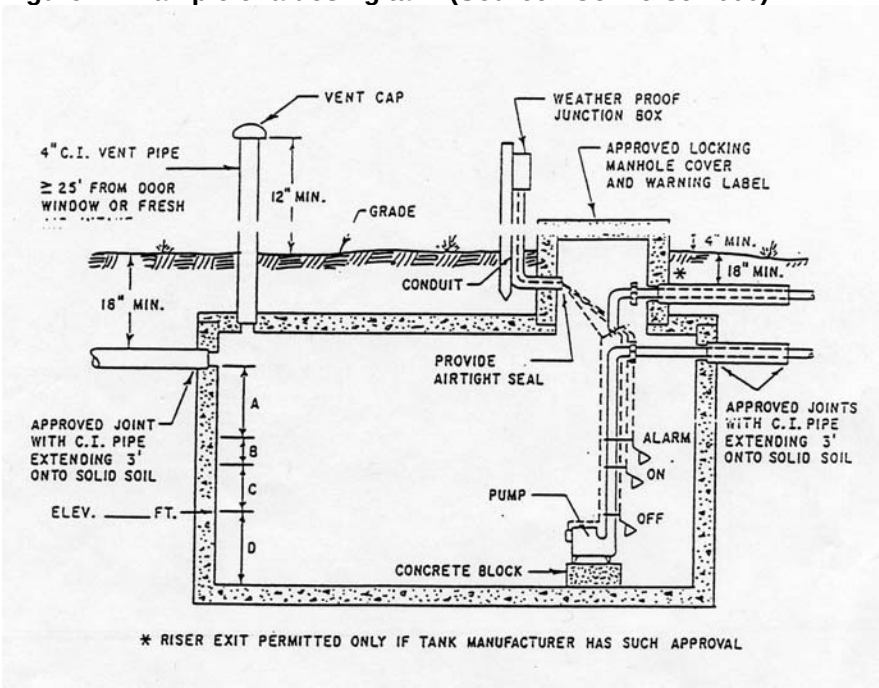
Step 4. **Size the dose chamber.**

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.

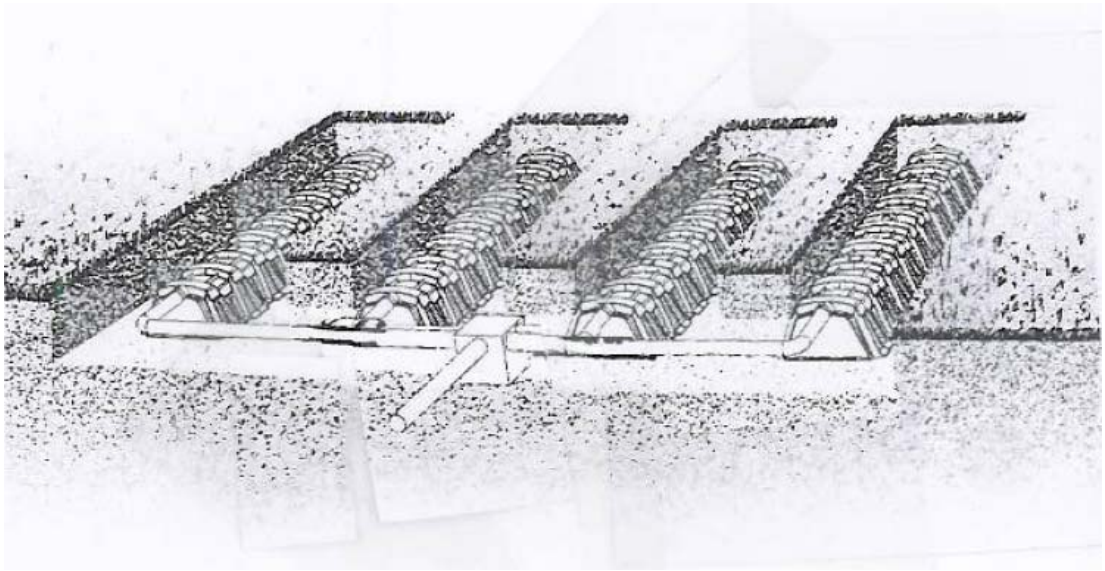


Figure 8. Example of Leaching Chamber Trench Installation

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.

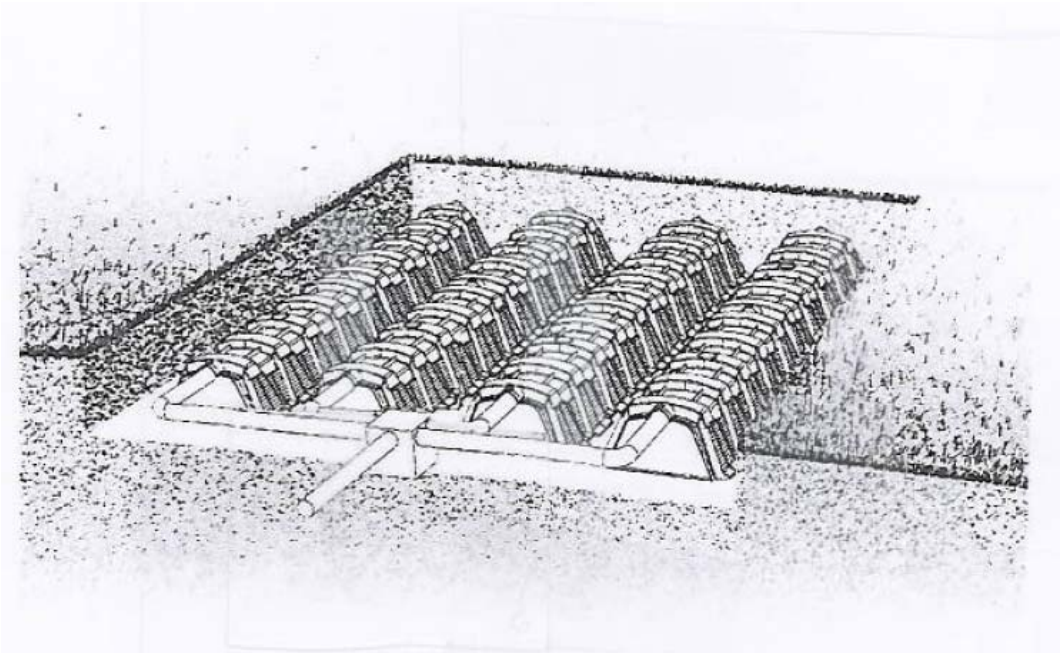


Figure 9. Example of Leaching Chamber Bed Installation

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.

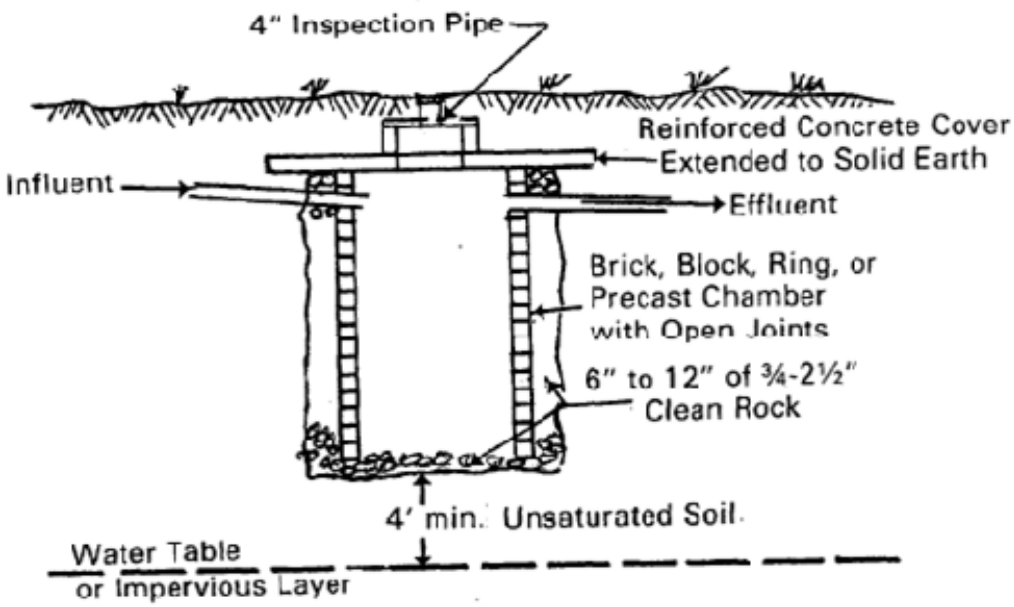


Figure 10. Seepage Pit Cross Section

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

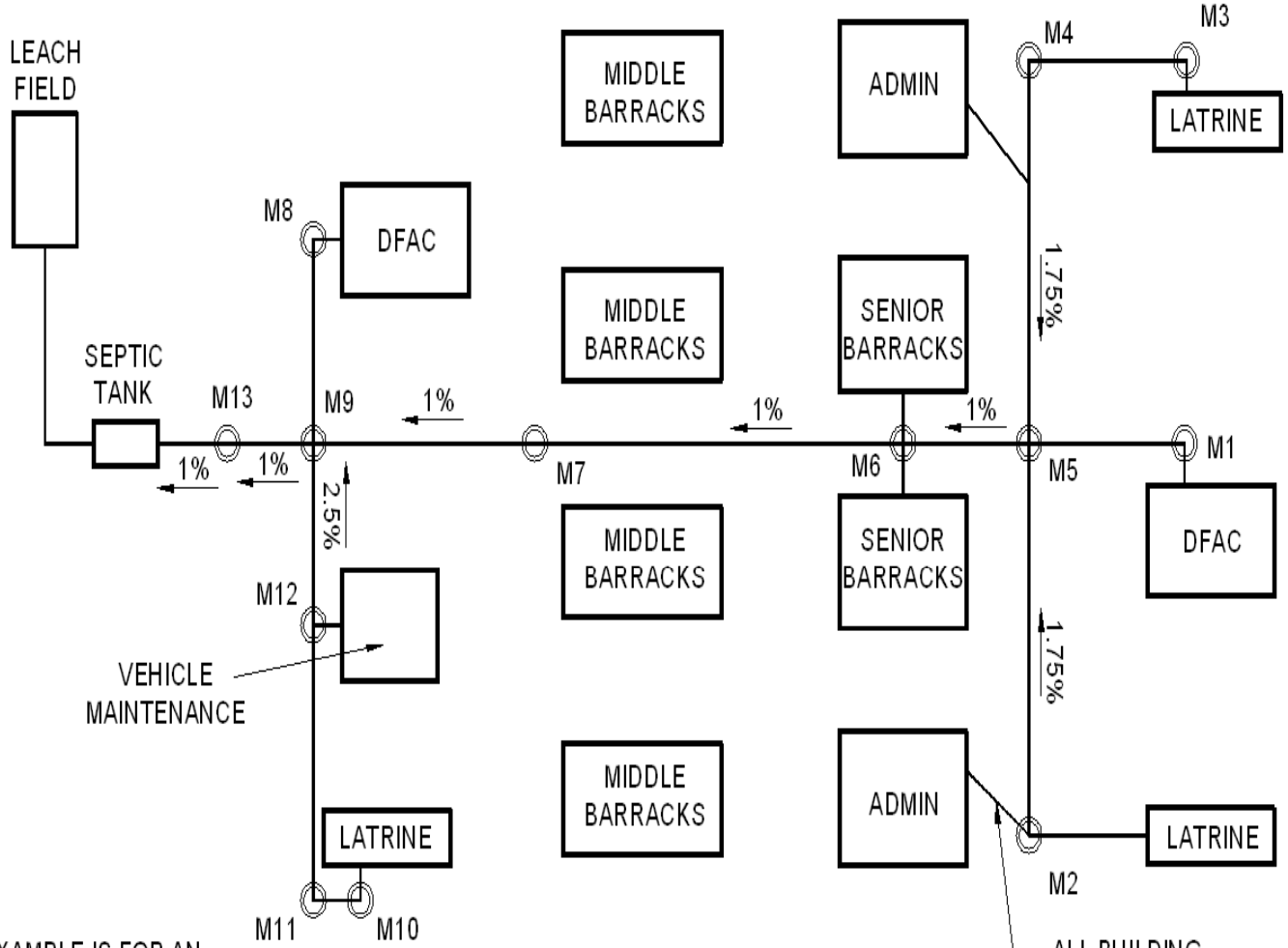
The objectives of the analysis for the system include:

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

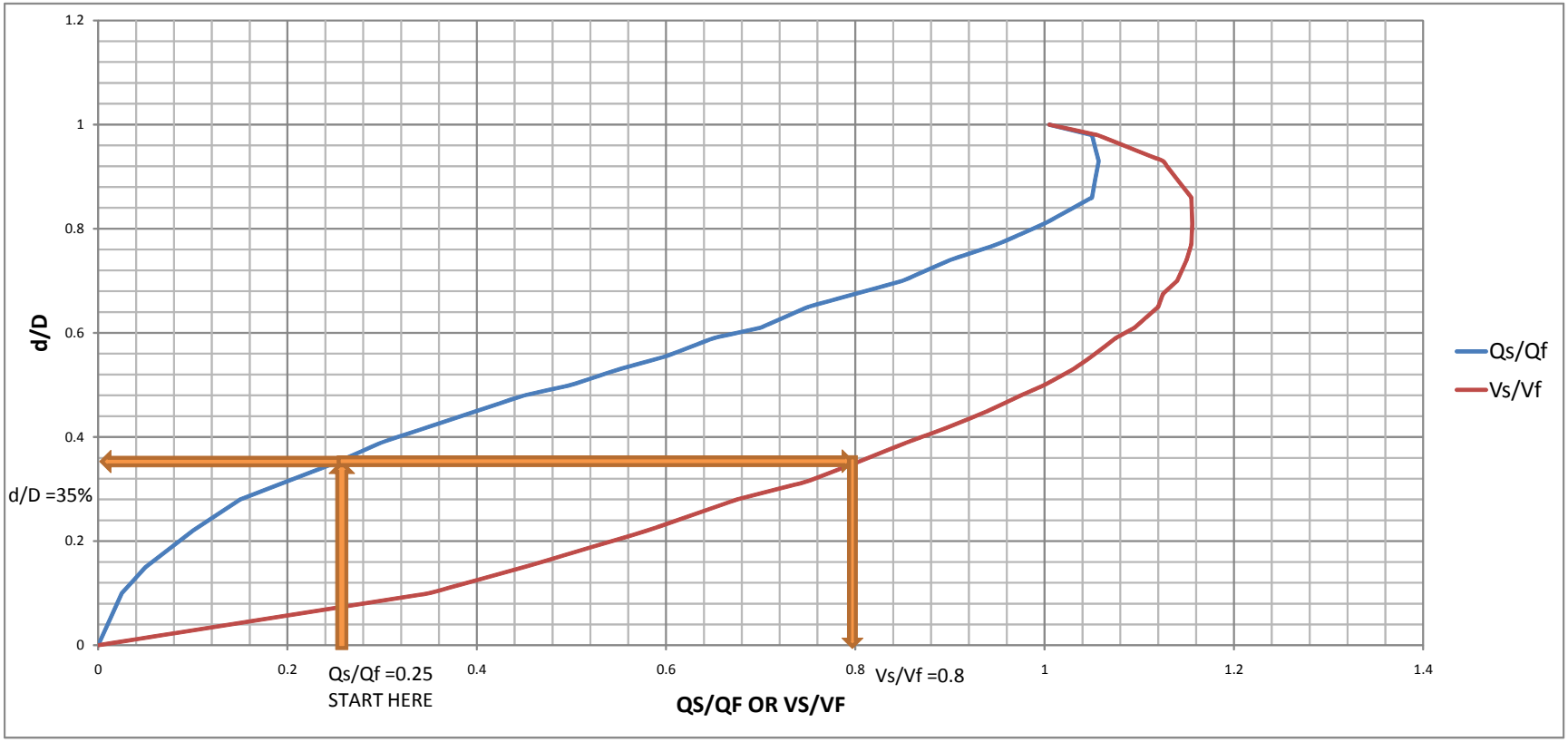
M5-M6 PEAK DIURNAL FLOW IS CALCULATED:
(8.3%*2+21.5%*2+6.0%)*5.6 Lps = 3.68 Lps
USE THE SAME METHOD TO CALCULATE THE ADF:
(8.3%*2+21.5%*2+6.0%)*1.72 Lps = 1.13 Lps

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

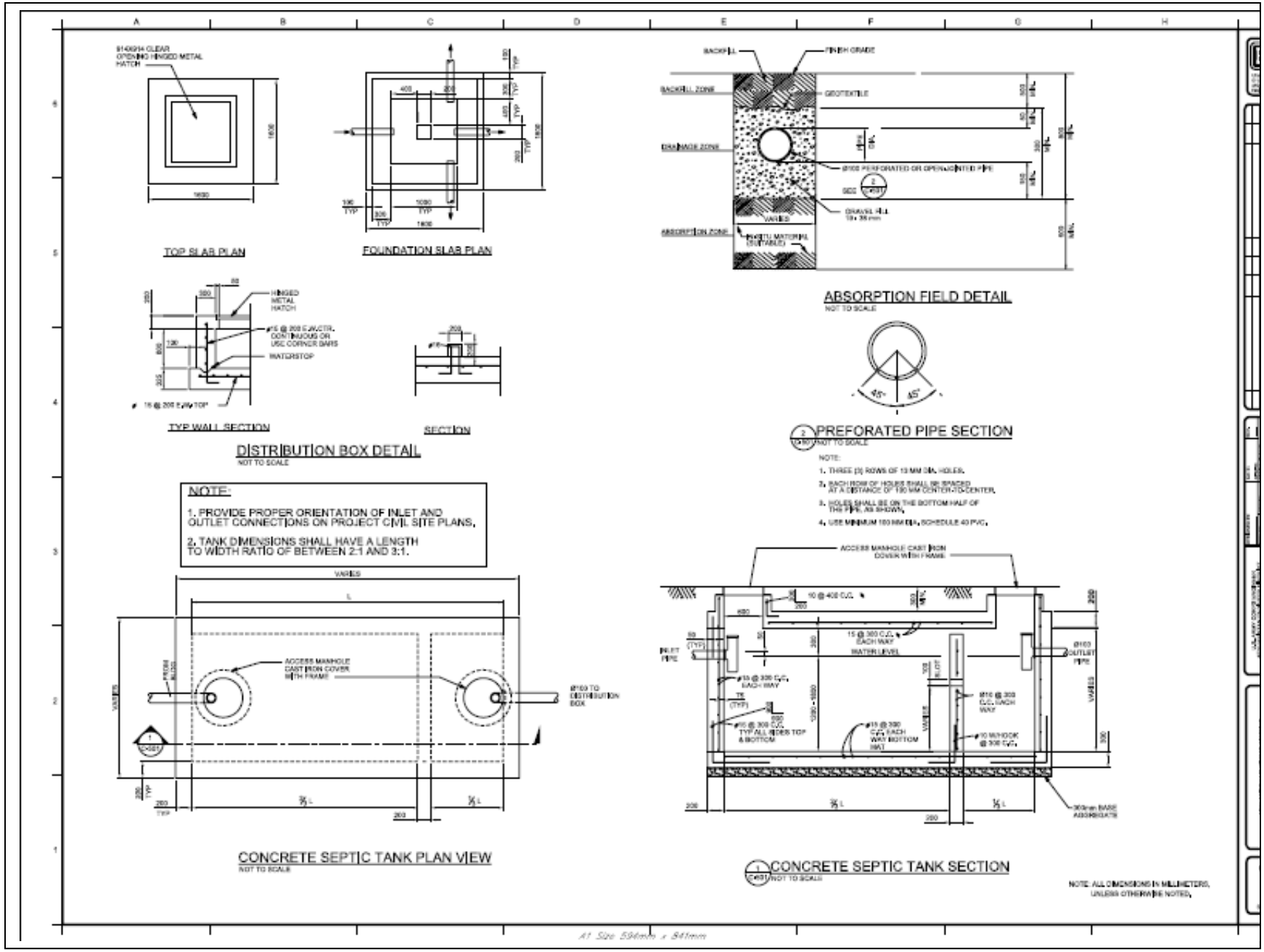
Absorption Bed and Trench Details

Dosing system Layout

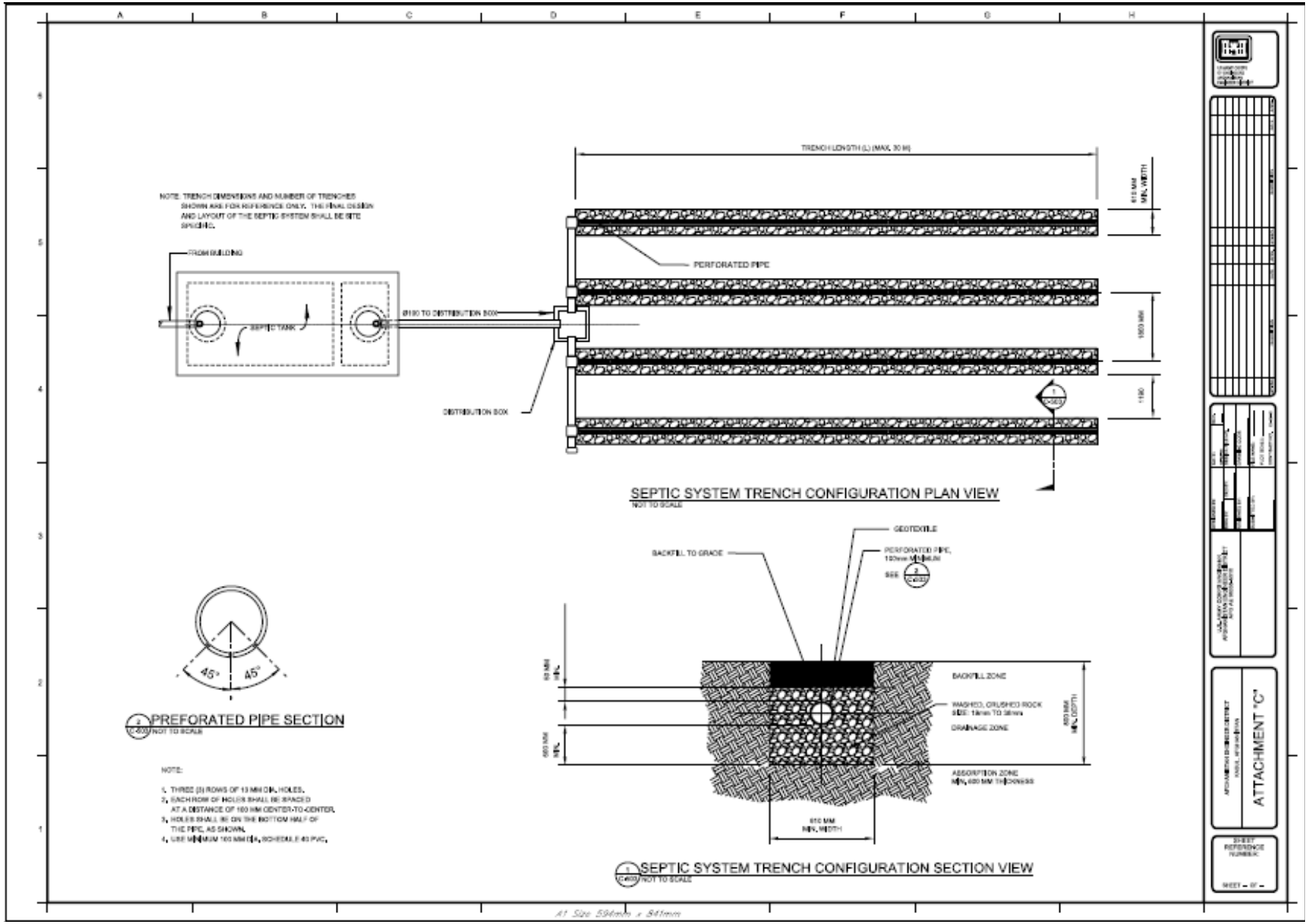
Leaching Chambers Option 1A and 1B (Traffic Loading)

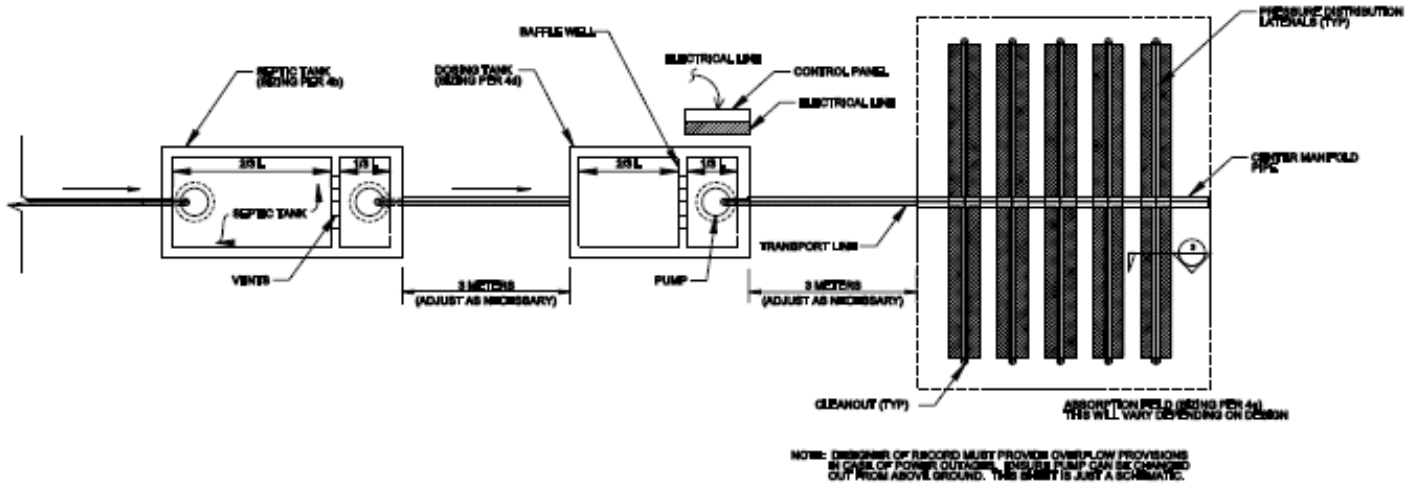
Leaching Chambers Option 2

AED Design Requirements
Sanitary Sewer & Septic Systems



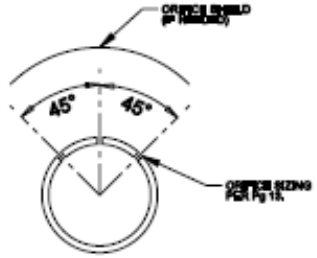
AED Design Requirements
Sanitary Sewer & Septic Systems



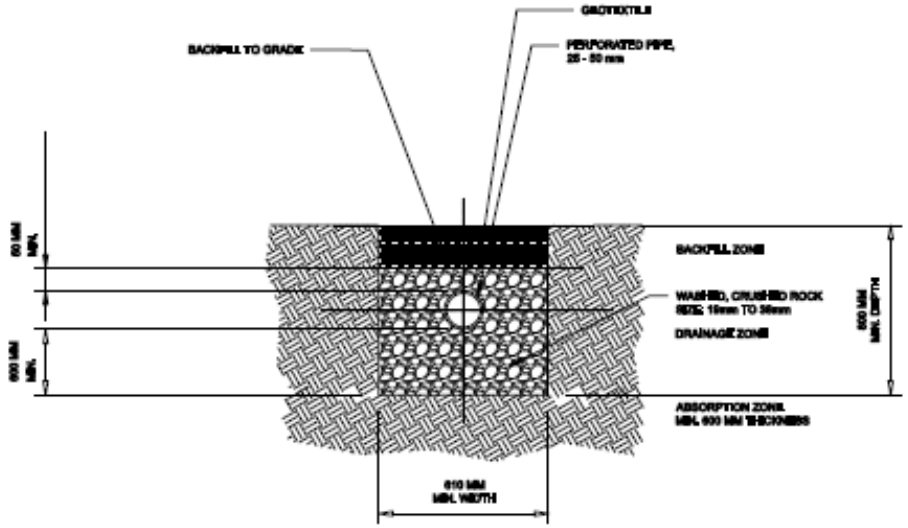


NOTE: DESIGNER OF RECORD MUST PROVIDE OVERFLOW PROVISIONS IN CASE OF POWER OUTAGES. ENSURE PUMP CAN BE CHANGED OUT FROM ABOVE GROUND. THIS SHEET IS JUST A SCHEMATIC.

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



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

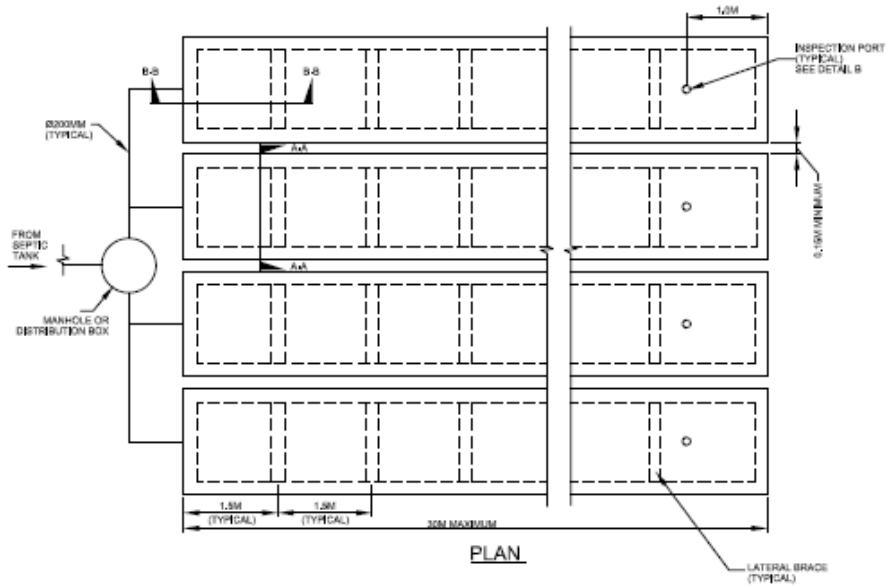


3 **TRENCH CONFIGURATION SECTION VIEW**
NOT TO SCALE

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.

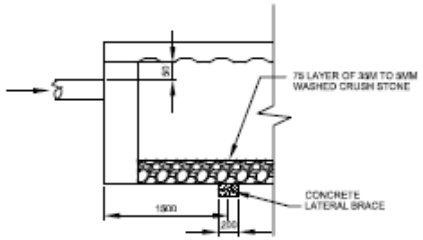
(6) LAY DOWN STEEL DECK, CHAIR REINFORCING IN CENTER OF 100MM SLAB, FORM EDGES, AND POUR SLAB.

(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.

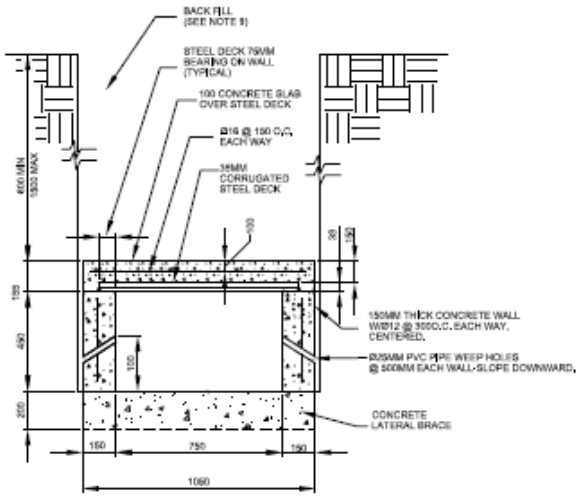
(8) USE A WELL-GRADED SOIL AS BACKFILL. A WELL-GRADED SOIL CONTAINS AN EVEN DISTRIBUTION OF PARTICLE SIZES, RANGING FROM SILT THROUGH SAND TO GRAVEL, WITH A MAXIMUM OF 10 PERCENT FINES (SOIL PASSING #200 SIEVE). COMPACT BACKFILL IN LIFTS NOT EXCEEDING 300MM TO A MINIMUM OF 95 PERCENT OF THE MODIFIED 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

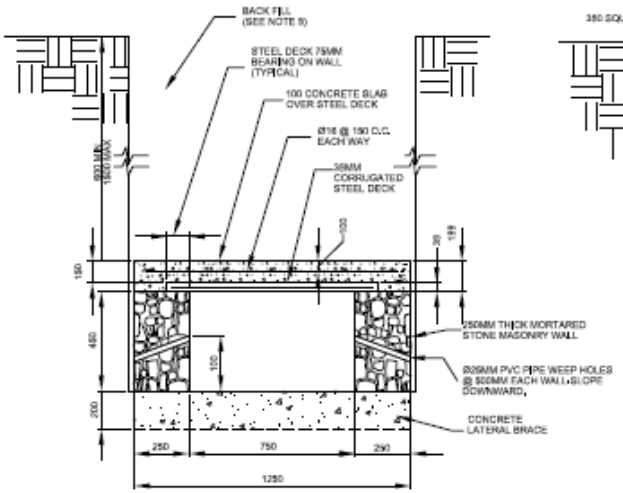
(10) LEACHING CHAMBERS HAVE BEEN DESIGNED FOR A MAXIMUM TRUCK AXLE LOAD OF 12,000 KG.



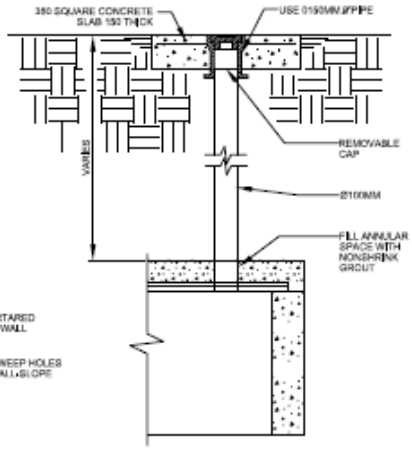
CROSS SECTION B-B



CROSS SECTION A-A
CONCRETE WALL OPTION 1A



CROSS SECTION A-A
STONE WALL OPTION 1B



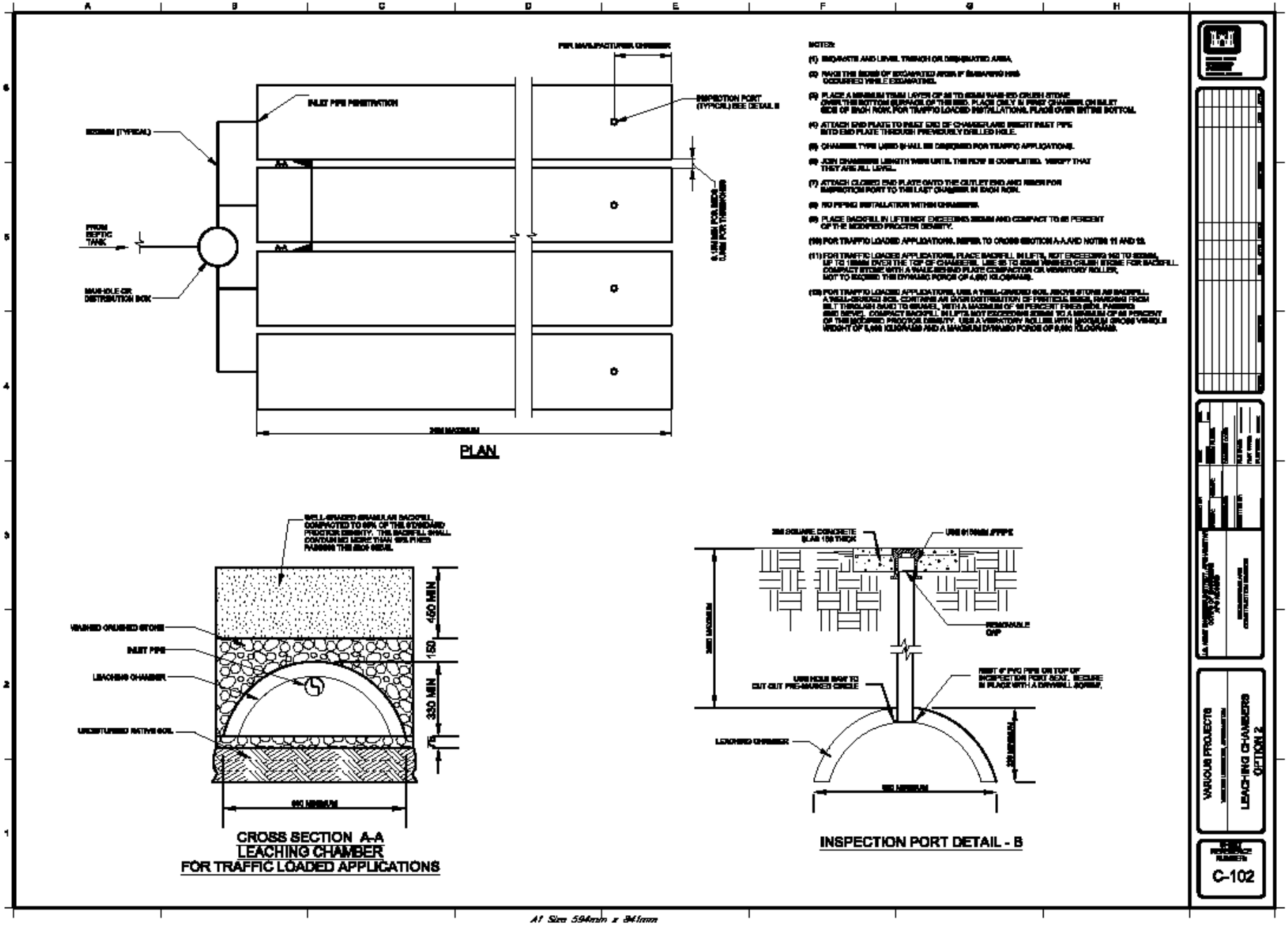
INSPECTION PORT DETAIL - B

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

TABLE OF CONTENTS

AED DESIGN REQUIREMENTS
FOR
SITE LAYOUT GUIDANCE
VARIOUS LOCATIONS,
AFGHANISTAN

Section	Page
1. General	**1**
2. Site Layout	**1**
3. Building Location	**1**
a) Dimensional Factors	1
b) Environmental Factors	1
c) Orientation Factors	1
d) Other Siting Factors	1
4. Vehicular Circulation	**1**
a) Access Intersections	2
b) Controlled Entrances	3
c) Sight Distances	3
5. Parking	**4**
6. Pedestrian Circulation	**4**
7. Utility System Design	**5**
a) Utility Separation	5
8. Physical Security	**6**
a) Standard 1. Standoff Distance	6
b) Standard 2. Unobstructed Space	11
c) Standard 3. Drive-Up/Drop-Off Areas	11
d) Standard 4. Access Roads	12
e) Standard 5. Parking Beneath Buildings or on Roofs	12
11. As-Builts	**12**

Figures

Figure 1. Standoff Distance – Controlled Perimeter 8

Figure 2. Standoff Distance – No Controlled Perimeter 8

Figure 3. Parking and Roadway Control for Existing Buildings – Controlled Perimeter 9

Figure 4. Parking and Roadway Control for Existing Buildings - No Controlled Perimeter 9

Tables

Table 1. Dimensions for Design Vehicles 2

Table 2. Minimum Turning Radii for Design Vehicles 2

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

Table 4. Minimum Sight Distance 3

Table 5. Standoff Distances for New and Existing Buildings 7

1. General

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

2. Site Layout

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

3. Building Location

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

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

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

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

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

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

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

4. Vehicular Circulation

Circulation should promote safe and efficient movement of vehicles and pedestrians. Maintaining maximum separation of vehicles and pedestrians helps promote safety. Safe roadway circulation systems have a perceivable hierarchy of movement, lead to a clear destination and do not interrupt other activities. The road system should be planned to keep groupings of related functions reasonably close to each other and the interrelating land-use areas for maximum efficiency. Additionally, the road system should minimize on site travel and permit the optimum circulation of traffic originating both outside and

within the site. The American Association of State Highway and Transportation Officials (AASHTO) places vehicles into two general classes: passenger cars and trucks. The passenger car class includes passenger cars, and light delivery trucks such as vans and pickups. The truck class includes single-unit trucks, recreation vehicles, buses, trucks semi-tractor trailer combinations, and trucks or truck tractors with semi-trailers in combination with full trailers. Roadway layout to provide the maneuverability and traffic safety required by the vehicles that utilize the roadways is necessary. Table 1 lists dimensions for some of the more common vehicles. Table 2 lists minimum turning radii for the same vehicles.

Table 1. Dimensions for Design Vehicles

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

Table 2. Minimum Turning Radii for Design Vehicles

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

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

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

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

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

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

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

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

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

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

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

Table 4. Minimum Sight Distance

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

NOTE: Sight distances are based on the following assumptions:

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

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

3. Turns are 90-degree.

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

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

5. Parking

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

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

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

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

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

6. Pedestrian Circulation

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

A grid path system is composed of straight lines and right angles and tends to provide the most direct access between locations. The grid system is appropriate in formal landscapes and in areas with strong

architectural definition. A curvilinear path system is less formal and should be used to encourage pedestrian interaction with the landscape where direct access to facilities is not critical. Organic sidewalk systems are unique in that the sidewalk patterns are defined by the space outside of the sidewalk and therefore vary in width. Because of this, organic sidewalks are less formal and often respond to natural elements in the surrounding landscape.

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

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

7. Utility System Design

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

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

8. Physical Security

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

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

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

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

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

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

Table 5. Standoff Distances for New and Existing Buildings

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

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

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

Figure 1. Standoff Distance – Controlled Perimeter

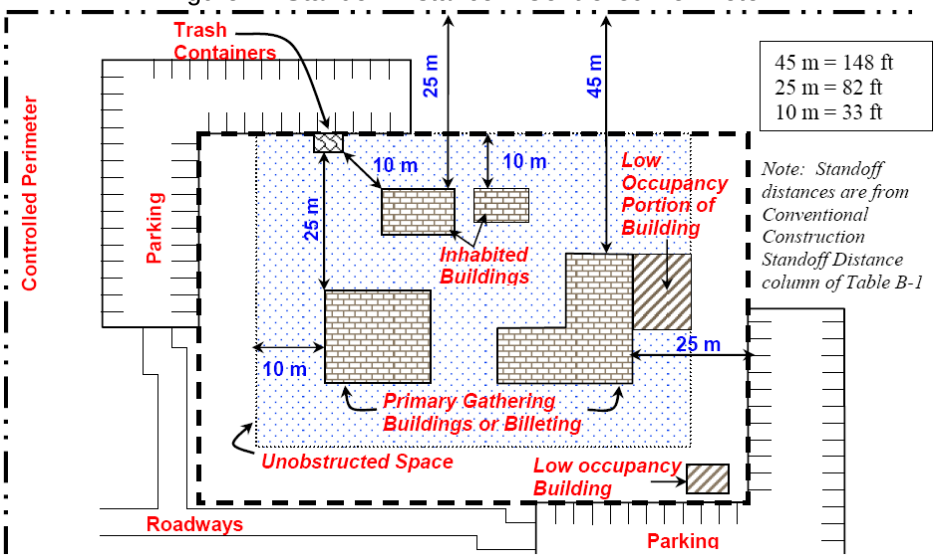


Figure 2. Standoff Distance – No Controlled Perimeter

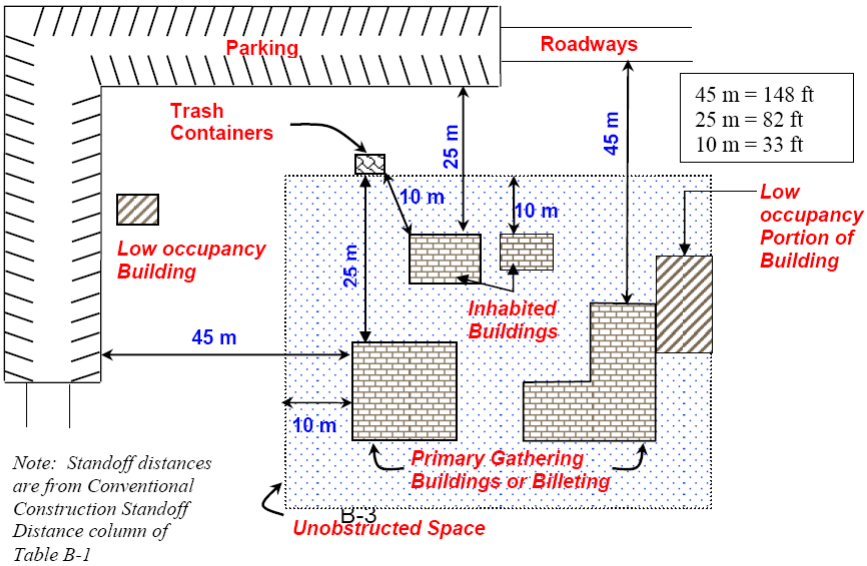


Figure 3. Parking and Roadway Control for Existing Buildings - Controlled Perimeter

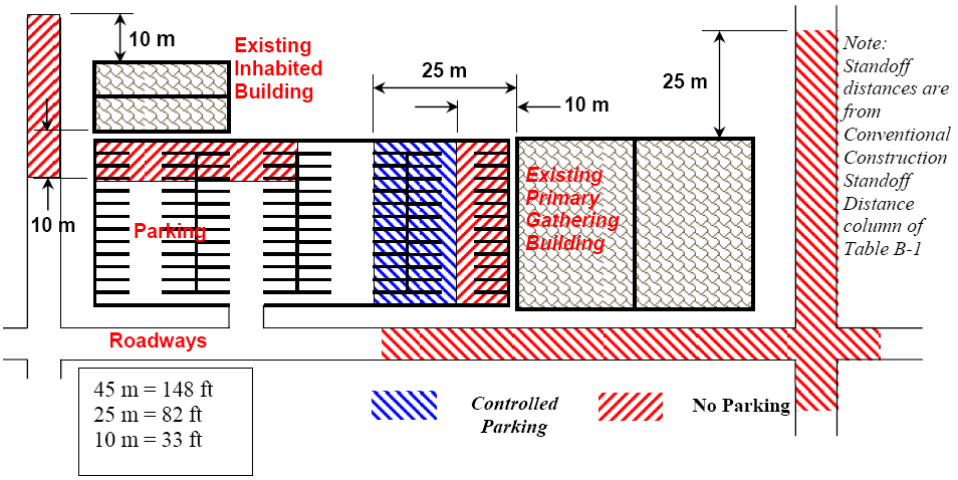
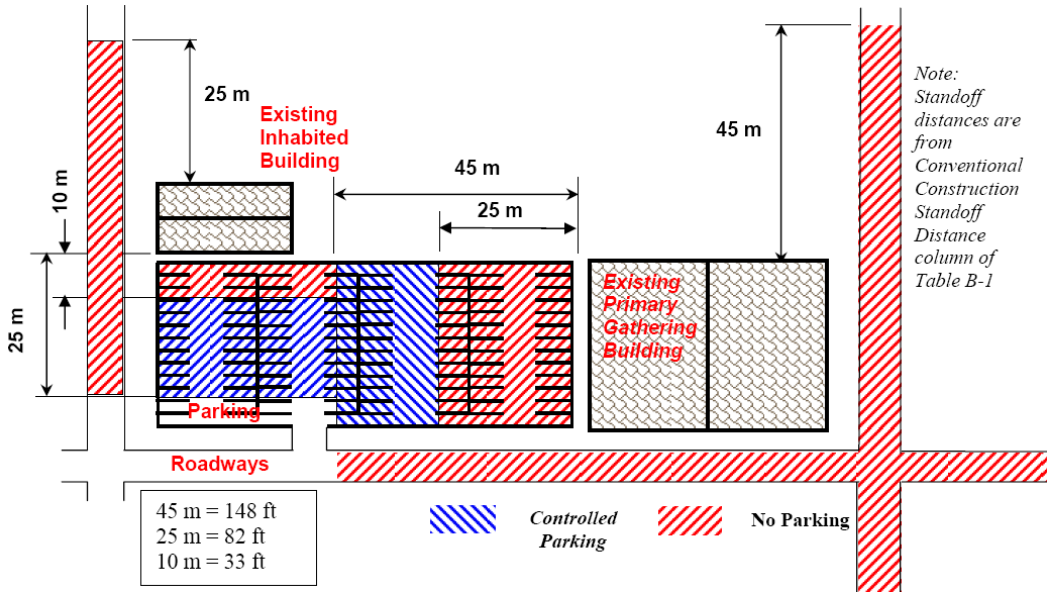


Figure 4. Parking and Roadway Control for Existing Buildings - No Controlled Perimeter



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

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

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

subject to the following:

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

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

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

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

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

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

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

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

10 meters from the existing building.

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

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

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

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

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

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

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

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

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

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

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

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

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

11. As-Builts

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

US Army Corps
of Engineers
Afghanistan Engineer District

AED Design Requirements:
Vertical Curve Design

Various Locations,
Afghanistan

MARCH 2009

TABLE OF CONTENTS

AED DESIGN REQUIREMENTS
FOR
VERTICAL CURVE DESIGN
VARIOUS LOCATIONS,
AFGHANISTAN

Section	Page
1. General	**1**
2. Vertical Curves	**1**
3. Crest Vertical Curve Stopping Sight Distance	**1**
4. Crest Vertical Curve Passing Sight Distance	**3**
5. Sag Vertical Curves	**5**
6. Design Considerations	**7**
7. As-Builts	**7**

Exhibits

Exhibit 1. Types of Vertical Curves	1

Tables

Table 1. Stopping Sight Distance	1
Table 2. Design Control for Stopping Sight Distance and for Crest Vertical Curves	2
Table 3. Passing Sight Distance for Design of Two-Lane Highways	3
Table 4. Design Controls for Crest Vertical Curves Based on Passing Sight Distance	4
Table 5. Design Control for Sag Vertical Curves	6

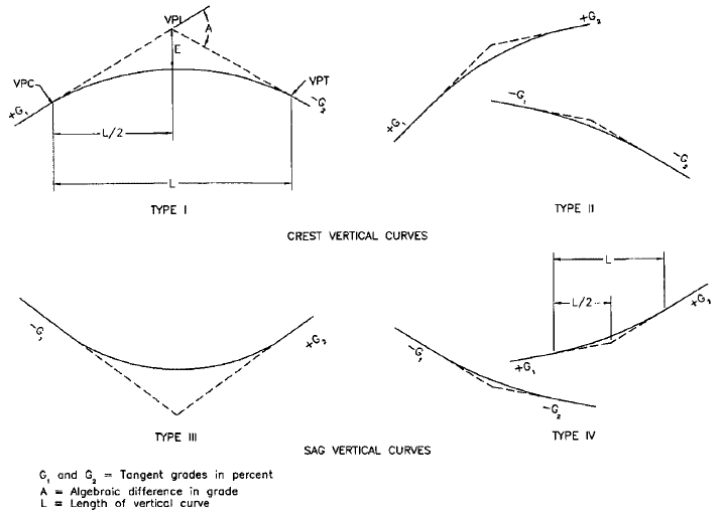
1. General

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

TABLE OF CONTENTS

AED DESIGN REQUIREMENTS
FOR
WATER TANKS
VARIOUS LOCATIONS,
AFGHANISTAN

Section	Page
1. General	**1**
2. Water Tanks	**1**
3. Distribution System Requirements	**1**
4. Types of Storage	**5**
5. Water Requirements	**7**
6. Capacity of Water Supply System	**8**
7. Storage Requirements	**8**
8. Amount of Water Available Under Emergency Conditions	**9**
9. Elevated Storage Capacity	**9**
10. Water Distribution System Capacity Analysis	**10**
11. Shop Submittals As-Builts	**11**
12. References	**12**

Figures

Figure 1. Effects of Elevated Storage	**4**
Figure 2. Conventional Elevated Storage Tank	**6**
Figure 3. Single Pedestal Elevated Storage Tank	**6**

Tables

Table 1. Domestic Water Allowance **7**

Table 2. Capacity Factors **8**

Appendices

Appendix A Example Water Distribution Analysis **13**

Appendix B Procedures for Leakage and Pressure Testing **17**

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.

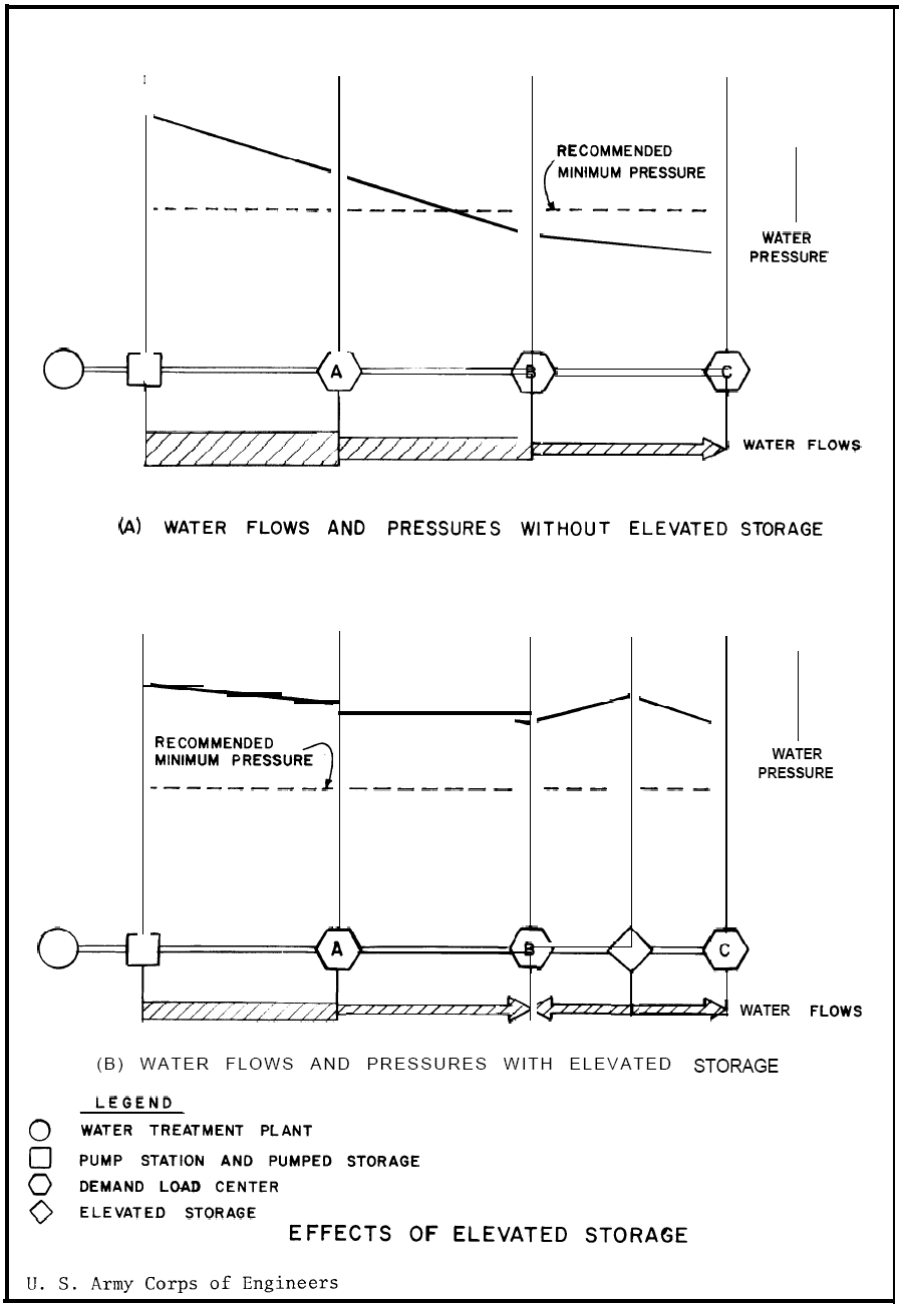
c) Pressure Provisions.

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

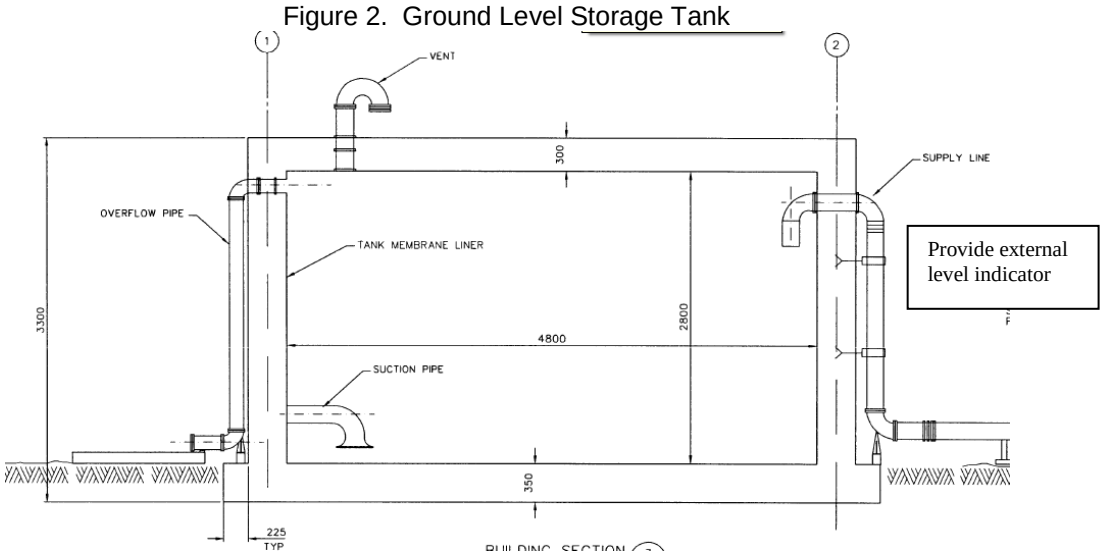
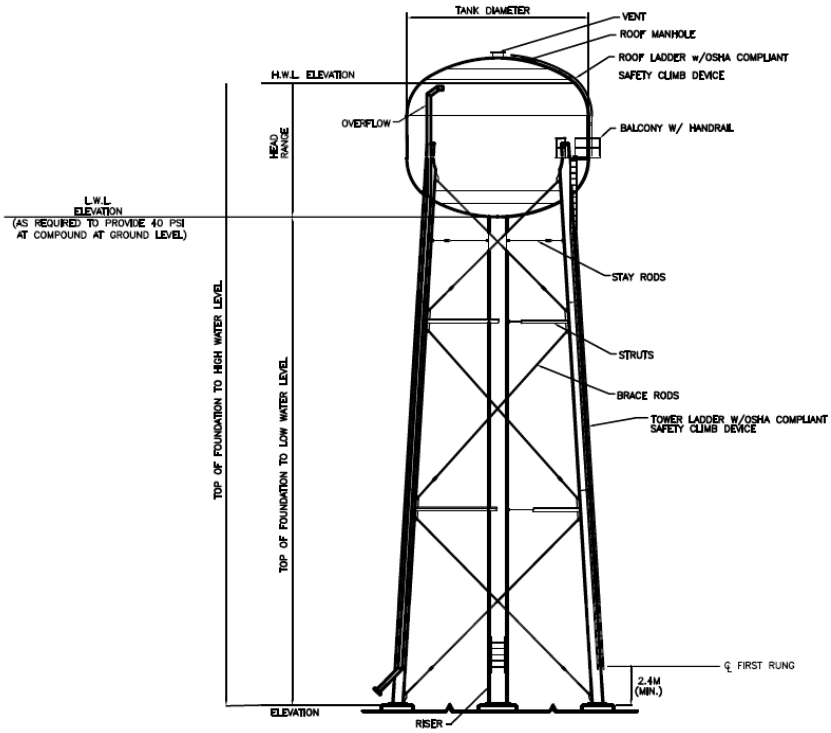


Figure 3. Standard Elevated Storage Tank



ELEVATION
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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:

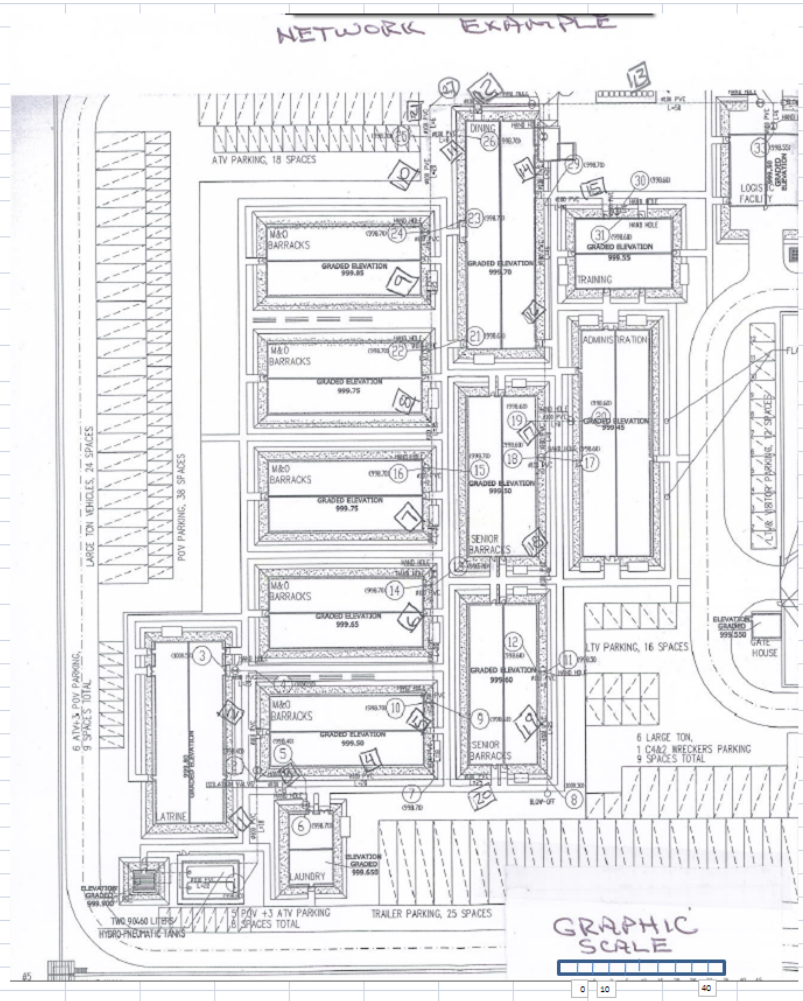
Water Distribution Hydraulic Analysis Example Network Description Table

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:

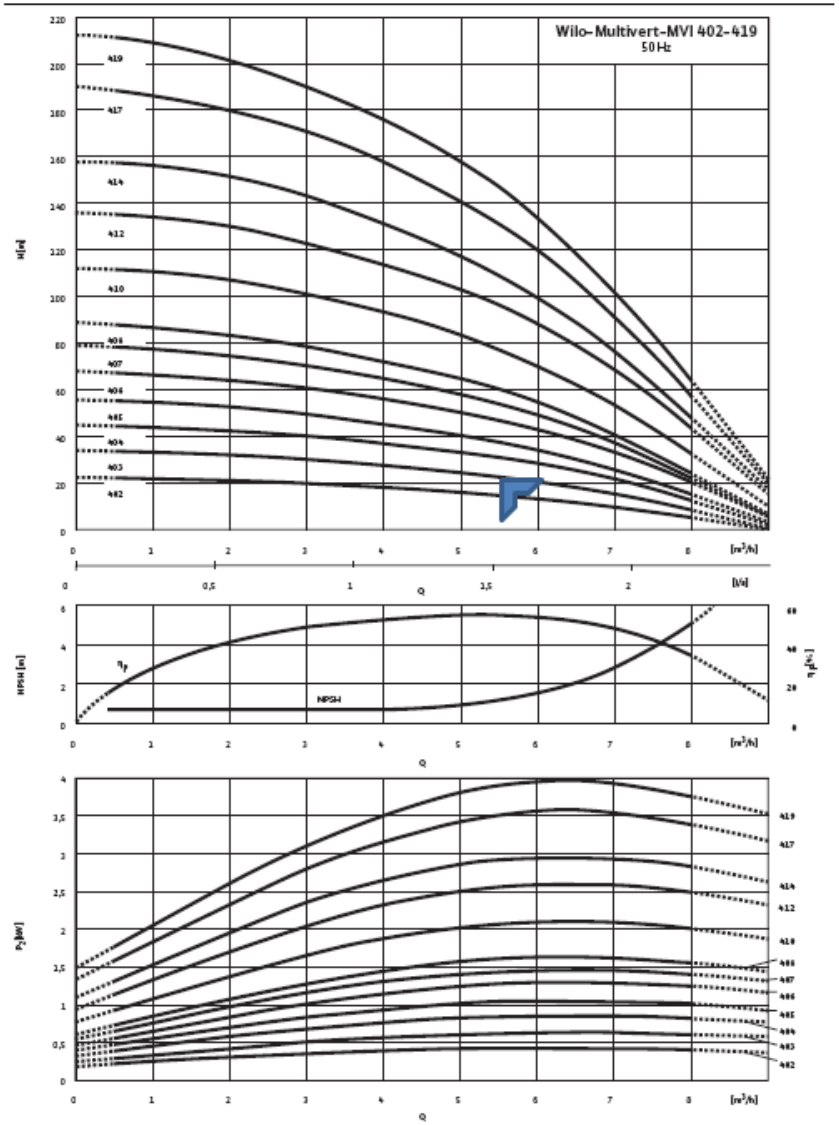
1. Verifying the minimum diameter for the distribution piping is large enough to minimize head loss throughout the system such that the technical criterion for minimum pressure is achieved. This is to be done with the minimum amount of elevated tank or booster pump energy requirements to minimize operation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

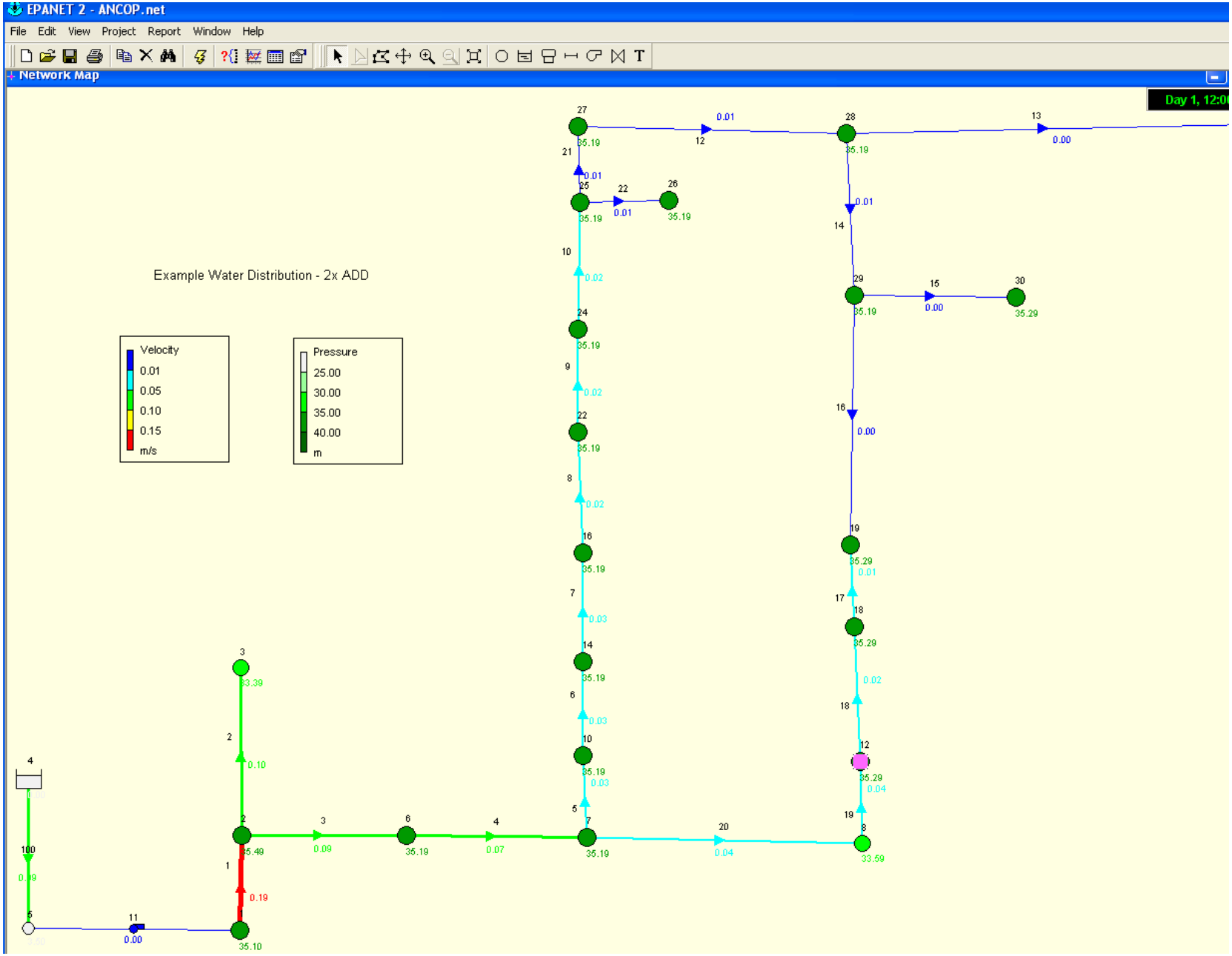
1. Set the design units to SI and defaults and labels
    a. Under project pull down tab edit the following
        i. Hydraulics – set head loss form to Hazen Williams (H-W) and flow units to liter per second (L/S)
        ii. Properties – set pipe length to 25 (meters) and roughness to 145 (conservative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

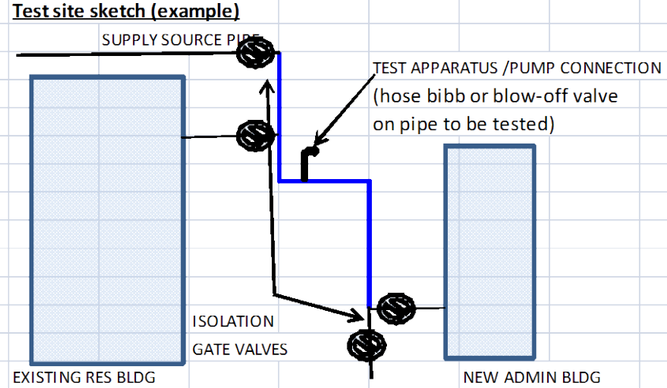
Pressure test data

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.

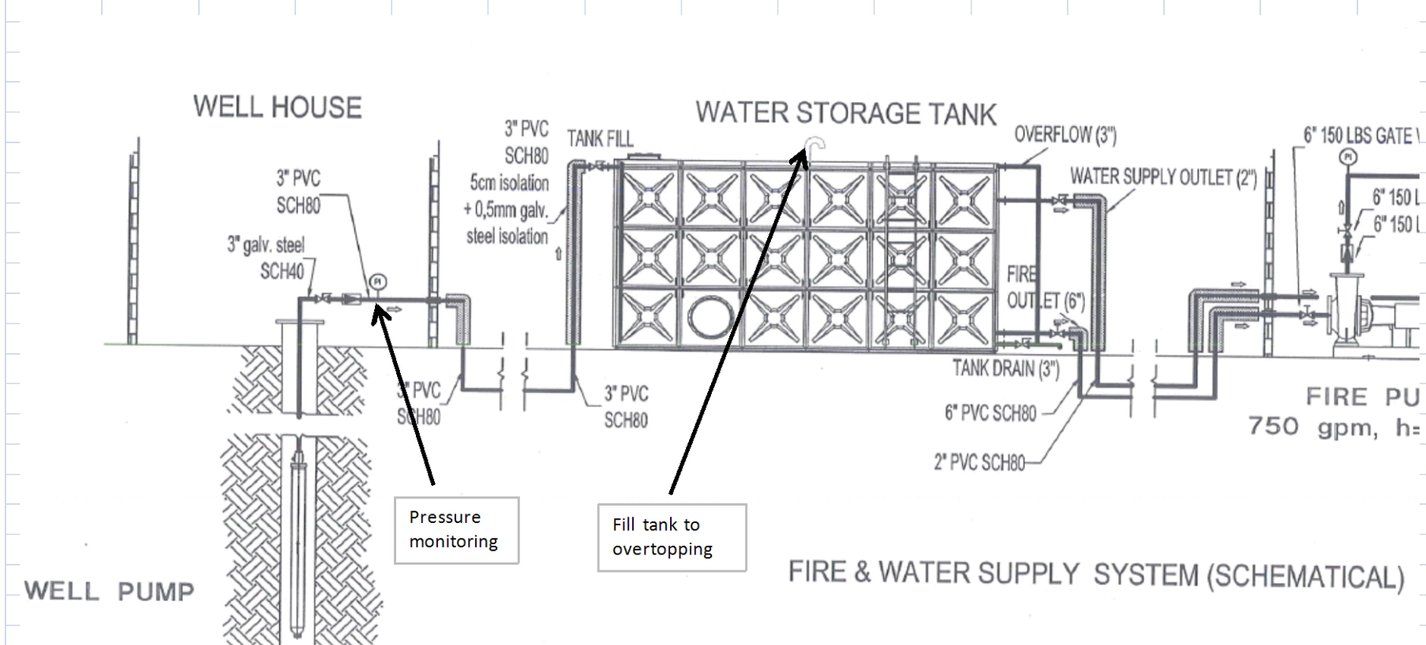
Low Pressure Hydrostatic Test Option

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

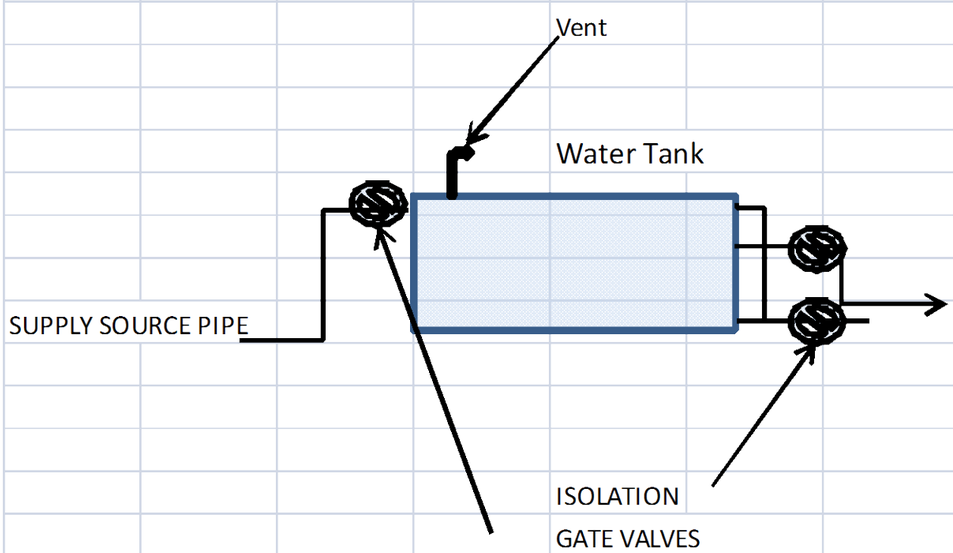
Test Information (shall be printed including names)

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

TABLE OF CONTENTS

AED DESIGN REQUIREMENTS
FOR
WELL PUMPS & WELL DESIGN
VARIOUS LOCATIONS,
AFGHANISTAN

Section	Page
1. General	**1**
2. Wells	**1**
3. Types of Wells	**2**
4. Water Quality Evaluation	**3**
a) Selection of a Well Site	3
b) Sampling and Analysis	5
5. Well Hydraulics	**5**
6. Well Design and Construction	**7**
a) Diameter	7
b) Depth	9
c) Casing	9
d) Screens	10
e) Filter Packing	12
f) Grouting and Sealing	13
g) Accessibility	14
h) Details Relating to Water Quality	14
7. Well Pumps	**15**
a) Pump Type	15
b) Pump Capacity	16
8. Development and Disinfection	**17**
a) Well Development	17
b) Disinfection of Completed Well	18
c) Disinfection of Flowing Artesian Wells	18

9. As-Builts **19**

10. References **19**

Appendix A. Guide Specifications for Drinking Water Wells **20**
Appendix B. Water Well construction Process **34**
Appendix C. Examples of Unacceptable and Acceptable Well Construction **35**
Appendix D. Well Pump Hydraulic Sizing Example **39**

Tables

Table 1. Types and Methods of Well Installation 3

Table 2. Minimum Distance from Pollution Sources 4

Table 3. Well Diameter vs. Anticipated Yield 8

Table 4. Change in Yield for Variation in Well Diameter 9

Table 5. Minimum PVC SCH 80 pipe casing wall thicknesses by well diameter 10

Table 6. Minimum steel pipe casing wall thicknesses by well diameter 10

Table 7. Characteristics of Pumps Used in Water Supply Systems 16

Figures

Figure 1. Diagrammatic Section of Gravel-Packed Well 2

Figure 2. Diagram of Water Table Well 6

Figure 3. Well in Rock Formation 7

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:

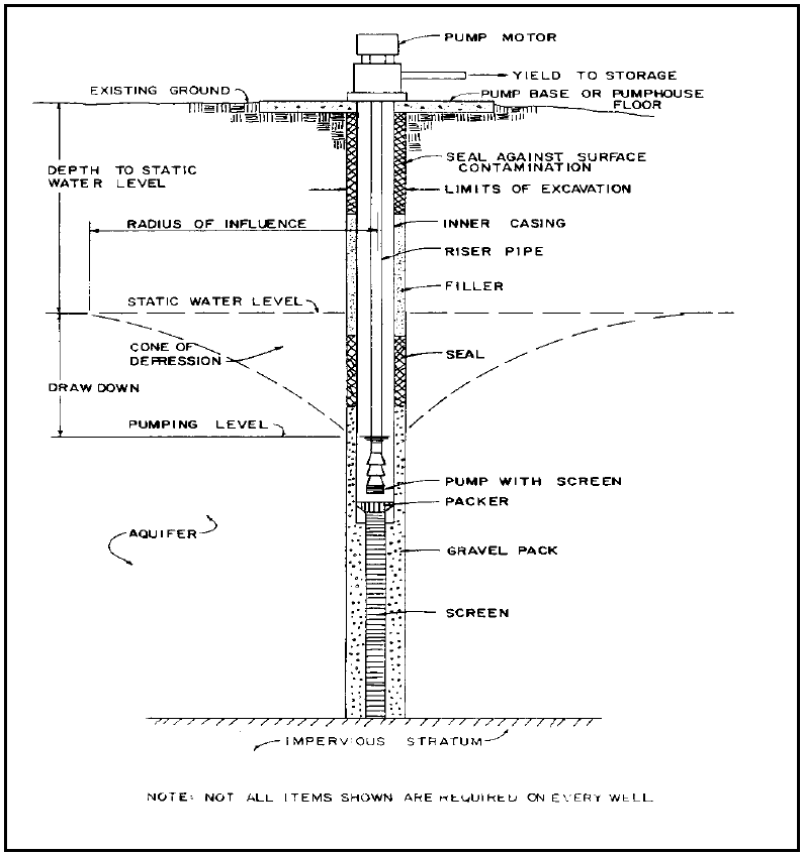
- Well casing diameter
- Casing materials
- Well and casing depth
- Well screen length and diameter
- Well screen slot openings
- Aquifer and well development and gravel pack material
- Collection and analysis of pump test and water quality data
- Selection of well pump

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.

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

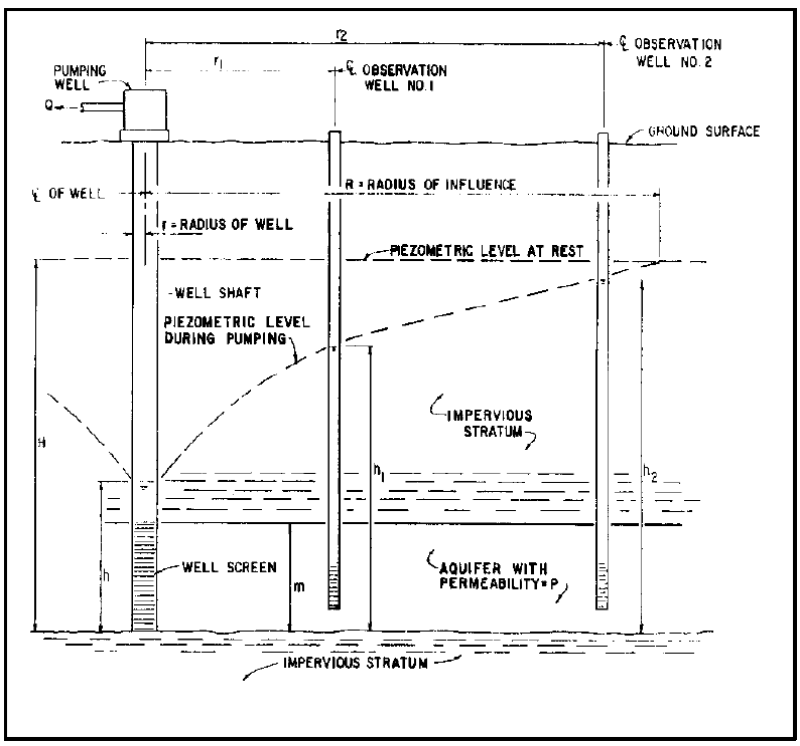
Where:

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

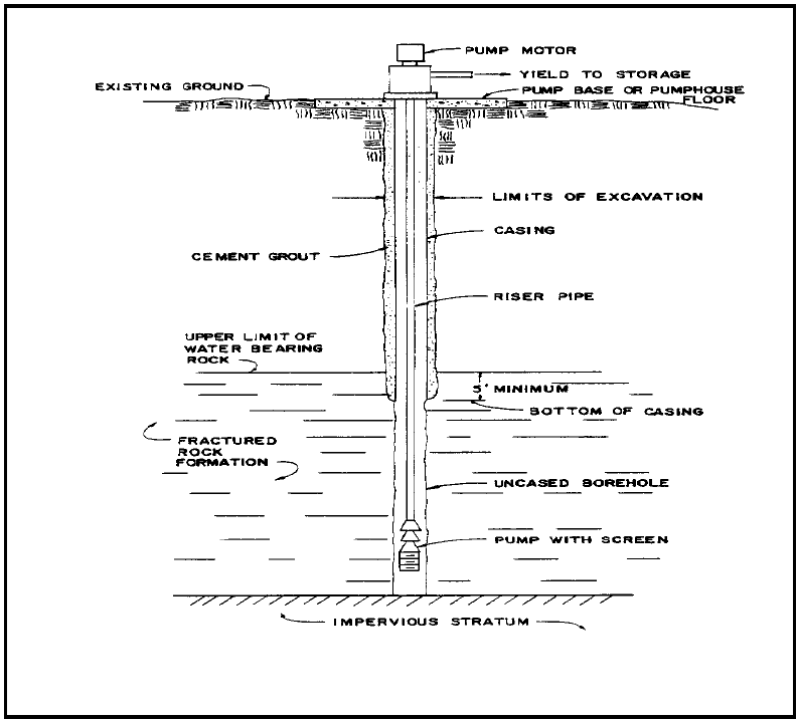


6. Well Design and Construction

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

INDEX

1. Applicable Publications
2. Location and Depth of Well
3. Local Conditions
4. Protection of Existing Facilities
5. Protection of Quality of Water
6. Test Well Development
7. Test Well Pumping
8. Approval of Test Well
9. Well Construction
10. Well Development
11. Well Pumps
12. Tests
13. Disinfection
14. Abandonment of Well
15. Clean-up
16. Quality Control
17. Submittals
18. Payment
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

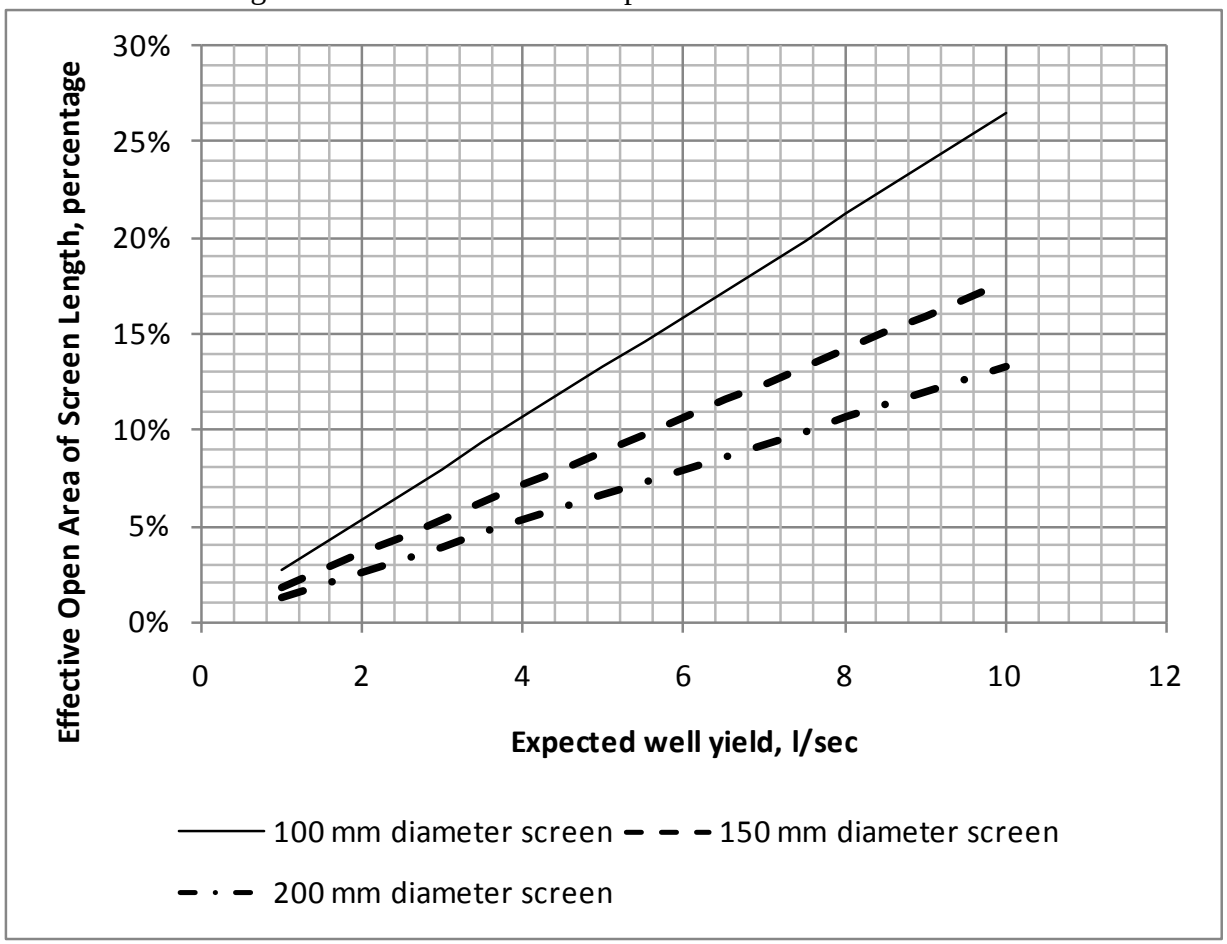
9.5 WELL SCREEN: The well screen and attached fittings shall be constructed entirely of corrosion resistant stainless steel unless specifically authorized by AED. Steel screens shall be made from Type 304 stainless steel meeting the requirements of AWWA A100. High chloride concentrations will quickly 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.

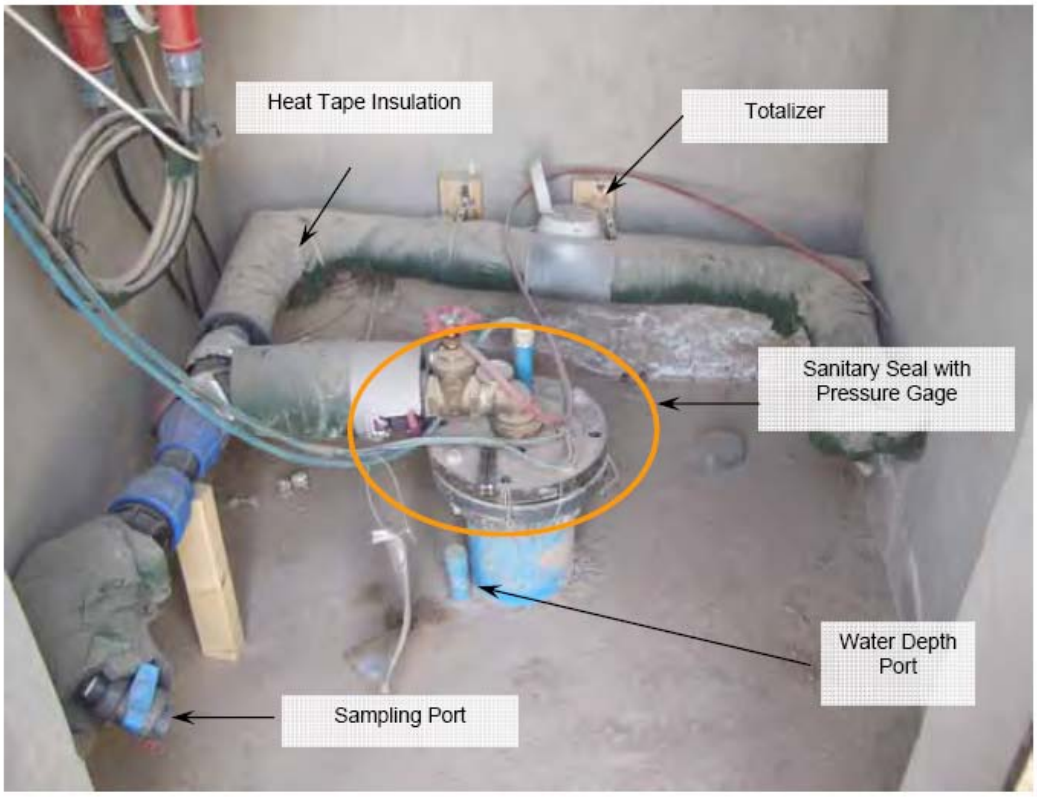


Figure 7. Acceptable wellhead completion, with totalizer (rear center), raw water sample port (lower left), sanitary port for measuring water level (left of 8-in blue well casing), and adequate sanitary seal on well casing (center).

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

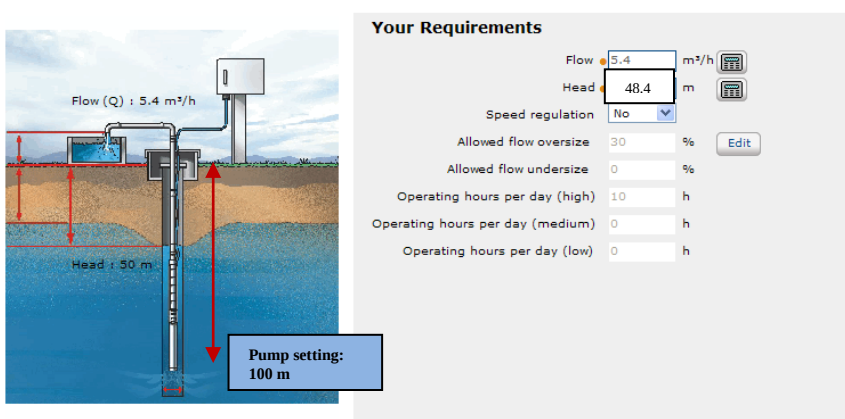
a) System Head Curve. The first step in the pump selection process is to determine the region of operating head curve for the pump piping system. The head curve is determined from the total dynamic head (TDH) required in the piping system at the design flow rate. The TDH is the total discharge head (h_d) 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:

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

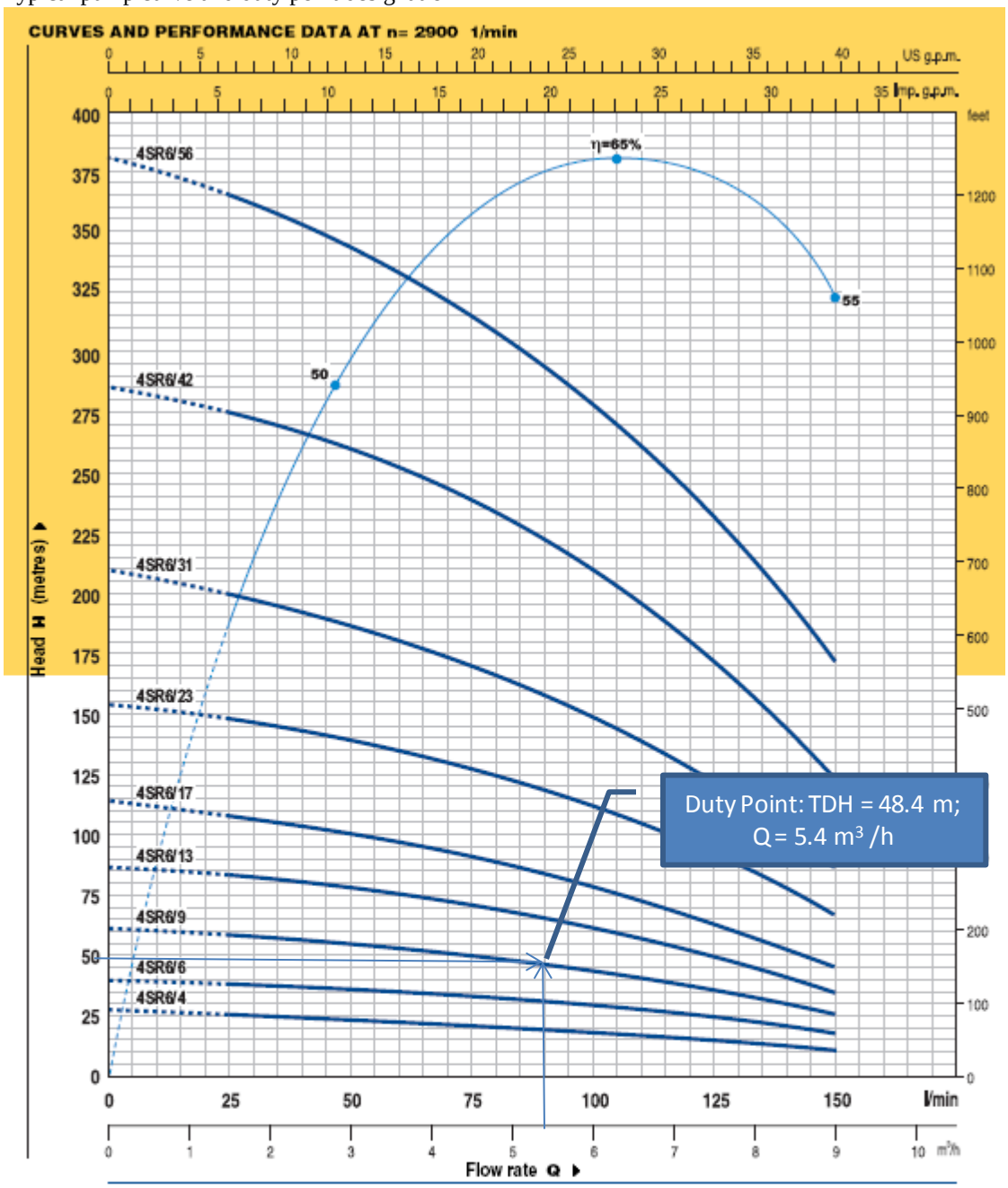
Where:

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

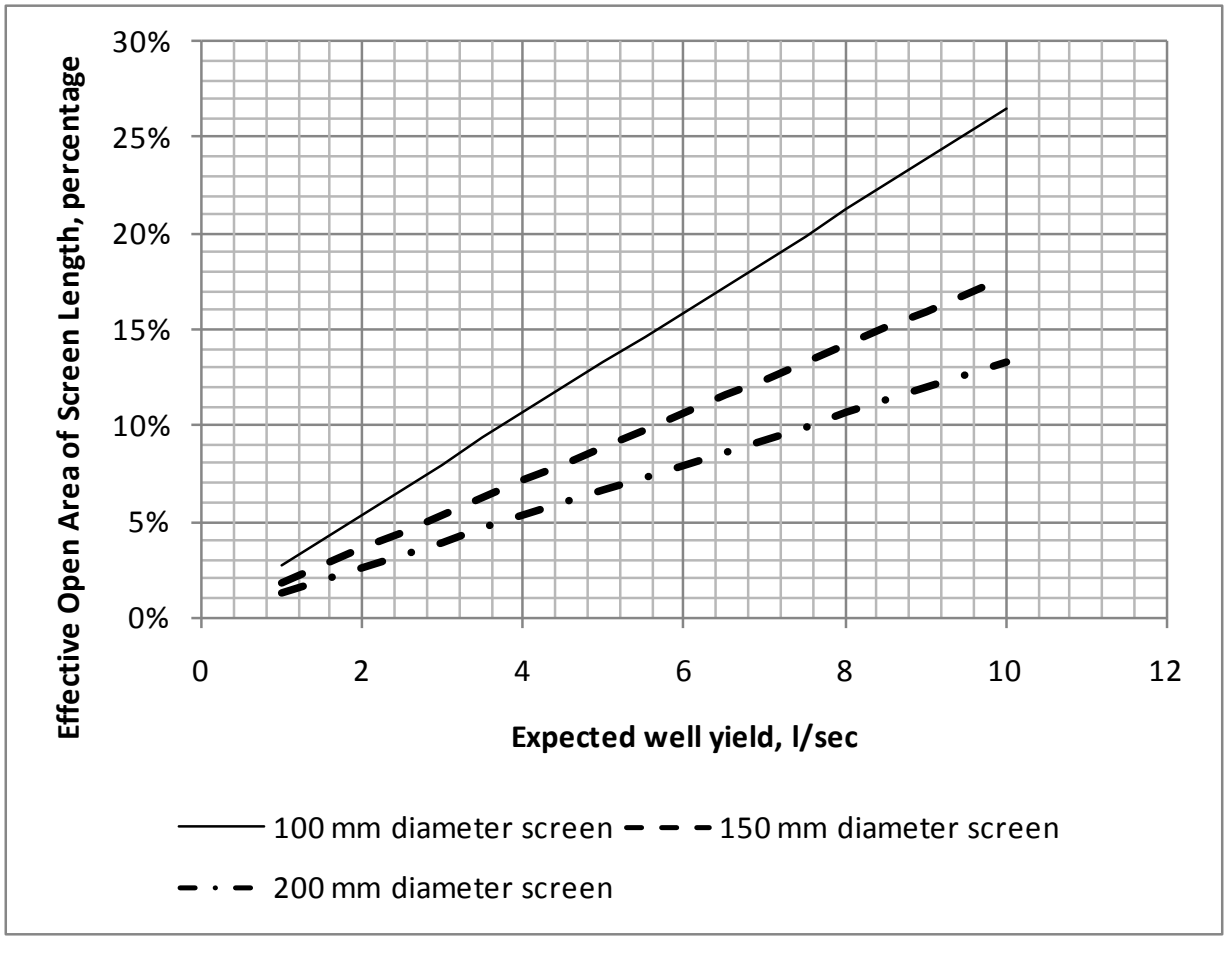
(h) Calculations supporting screen slot size

(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.

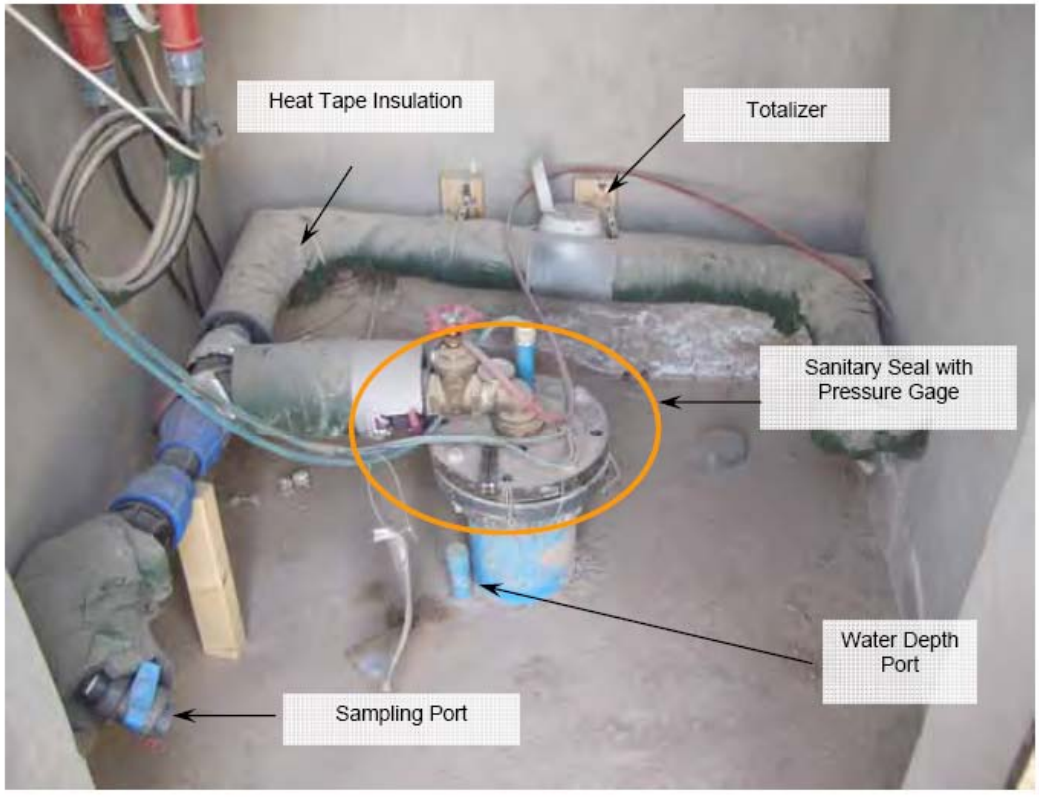


Figure 7. Acceptable wellhead completion, with totalizer (rear center), raw water sample port (lower left), sanitary port for measuring water level (left of 8-in blue well casing), and adequate sanitary seal on well casing (center).