Executive Summary

As government officials and donors already know, vertical structures and the associated utilities are often the most easily recognized infrastructure support to rural areas. This manual combines codes from all over the world into an Afghanistan model. The use of this manual is intended to serve as a temporary standard during the solidification of National Standards for buildings.

This first version is still very rough and the errors are numerous, in fact, no country has yet to come up with a perfect manual that can foresee technologies, law changes and economic changes. You will see a greater emphasis on traditional buildings in rural areas – administrative buildings, health care, community centers and similar facilities. Less of an emphasis is laid upon fire protection, geotechnical engineering and advanced/foreign materials.

This manual is intended to be a work in progress, so please review and provide ideas, comments, suggestions and other feedback to either Eng. Mirkhel arif.mirkhel@mrrd.gov.af or members of the Engineering Services Directorate afkohistani1@gmail.com.

General Note:
The provision of this manual shall not be deemed to nullify any provision of national law and codes.
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Acronyms

Afs Afghani
AAC Afghan Architectural Code
ANDS Afghanistan National Development Strategy
AREDP Afghanistan Rural Enterprise Development Program
AWP Annual Work Plan
BOQ Bill of Quantity
CDC Community Development Council
CTA Chief Technical Advisor
DABS De Afghan Breshna Shirakat
DDA District Development Council
DDP District Development Plan
DPM Deputy Program Manager
ERDA Energy for Rural Development in Afghanistan
FP Facilitating Partner
GIS Geographic Information System
GOA Government of Afghanistan
ICT Information Communication Technology
kW Kilo Watt
LIDD Local Institution Development Department
M&E Monitoring and Evaluation
MD Man Day
MDG Millennium Development Goal
MERD Monitoring, Evaluation and Reporting Department
MIS Management Information System
MEW Ministry of Energy and Water
MOU Memorandum of Understanding
MRRD Ministry of Rural Rehabilitation and Development
NABDP National Area Based Development Program
NGO Non-Governmental Organization
NPP National Priority Program
NSP National Solidarity Program
O & M Operation and Management
PDP Provincial Development Plan
PM Program Manager
PRRD Provincial Rural Rehabilitation and Development
REnD Rural Energy Department
RESC Rural Energy Service Center
RET Rural Energy Technology
ToR Terms of Reference
TRC Technical Review Committee
UNDP United Nations Development Program
1 General Rural Building Classifications

Architecture has the complex task of meeting human needs with durable materials at an affordable price. This very dynamic process called architectural design needs to meet the following parameters:

i. Aesthetic
ii. Useful
iii. Sustainable (Economic, social and environmental)

The primary form of communication between the end user, funding source and the construction team are the architectural drawings. Though other drawings are possible, for rural Afghanistan, the primary examples of drawings will probably be limited to:

i. Plans (foundation plan, floors plans, master plan and roof plan)
ii. Elevations (Front elevation and side elevations)
iii. Sections (Cross section and longitudinal sections)

A building and construction drawings contains the following parts:

i. Architecture
ii. Survey
iii. Structure
iv. Water Supply
v. Electrical

Selection of constructional materials:

Two kinds of constructional materials are predominantly used in Afghanistan:

i. Local materials which are found where the buildings are built (e.g., stone, gravel and wood).
ii. Fabric materials which are manufactured locally and internationally by equipment like steel bar, cement, brick and marble with other kinds of stone.

Before architectural design of a building following parameters shall be observed:

i. Climate of the region
ii. Local constructional materials
iii. Orientation
iv. Use and Occupancy Classification

1.1 Occupancy Introduction

Structures or portions of structures shall be classified with respect to occupancy in one or more of the groups listed below. A room or space that is intended to be occupied at different times for different purposes shall comply with all of the requirements that are applicable to each of the purposes for which the room or space will be occupied. Structures with multiple occupancies or uses shall comply with Section 508 IBC. Where a structure is proposed for a purpose that is not specifically provided for in this code, such structure shall be classified in the group that the occupancy most nearly resembles, according to the fire safety and relative hazard involved.
2. Business Group (B)
3. Educational Group (E)
4. Factory and Industrial Groups (F-1 and F-2)
5. High Hazard Groups (H-1, H-2, H-3, H-4 and H-5)
6. Institutional Groups (1-1, 1-2, 1-3 and 1-4)
7. Mercantile Group (M)
8. Residential Groups (R-1, R-2, R-3 and R-4)
9. Storage Groups S-1 and S-2
10. Utility and Miscellaneous (Group U)

1.2 **Assembly Group A**

Assembly Group (A) occupancy includes, among others, the use of a building or structure, or a portion thereof, for the gathering of persons for purposes such as civic, social or religious functions; recreation, food or drink consumption or awaiting transportation.

Assembly occupancies shall include the following:

**A-1:** Assembly uses, usually with fixed seating, intended for the production and viewing of the performing arts or motion pictures including, but not limited to:

- Motion picture theaters
- Symphony and concert halls
- Television and radio studios
- Admitting and Audience Theaters

**A-2:** Assembly uses intended for food and drink consumption including, but not limited to:

- Banquet halls
- Night clubs
- Restaurants
- Taverns and bars

**A-3:** Assembly uses intended for worship, recreation or amusement and other assembly uses not classified elsewhere in Group (A) including, but not limited to:

- Amusement arcades
- Art galleries
- Bowling alleys
- Community halls
- Courtrooms
Dance halls (not including food or drink consumption)

Exhibition halls

Funeral parlors

Gymnasiums (without spectator seating)

Indoor swimming pools (without spectator seating)

Indoor tennis courts (without spectator seating) Lecture halls

Libraries

Museums

Places of religious worship

Pool and billiard parlors

Waiting areas in transportation terminals

A-4: Assembly uses intended for viewing of indoor sporting events and activities with spectator seating including, but not limited to:

Arenas

Skating rinks

Swimming pools

Tennis courts

A-5: Assembly uses intended for participation in or viewing outdoor activities including, but not limited to:

Amusement park structures

Bleachers

Grandstands

Stadiums

1.3 **Business Group B**

Business Group B occupancy includes, among others, the use of a building or structure, or a portion thereof, for office, professional or service-type transactions, including storage of records and accounts. Business occupancies shall include, but not be limited to, the following:

Airport traffic control towers

Ambulatory health care facilities
Animal hospitals, kennels and pounds

Banks

Barber and beauty shops

Car wash

Civic administration

Clinic-outpatient

Dry cleaning and laundries: pick-up and delivery stations and self-service

Educational occupancies for students above the 12th grade Electronic data processing

Laboratories: testing and research

Motor vehicle showrooms

Post offices

Print shops

Professional services (architects, attorneys, dentists, physicians, engineers, etc.)

Radio and television stations

Telephone

Exchanges

Training and skill development not within a school or academic program

1.4 Educational Group E

Educational Group E occupancy includes, among others, the use of a building or structure, or apportion thereof, by six or more persons at any one time for educational purposes through the 12th grade. Religious educational rooms and religious auditoriums, which are accessory to places of religious worship in accordance with Section 303.1 IBC and have occupant loads of less than 100, shall be classified as A-3 occupancies.

The following occupancy or buildings are related to MRRD projects:

1.5 Administration Buildings (Office Buildings)

General Introduction

Office building is the most tangible reflection of a profound change in employment patterns that has occurred over the last one hundred years. In present-day America, northern Europe, and Japan, at least 50 percent of the working population is employed in office settings as compared to 5 percent of the population at the beginning of the 20th century.
Interestingly, the life-cycle cost distribution for a typical service organization is about 3 to 4 percent for the facility, 4 percent for operations, 1 percent for furniture, and 90 to 91 percent for salaries. As such, if the office structure can leverage the 3 to 4 percent expenditure on facilities to improve the productivity of the workplace, it can have a very dramatic effect on personnel contributions representing the 90 to 91 percent of the service organization's costs.

To accomplish this impact, the buildings must benefit from an integrated design approach that focuses on meeting a list of objectives. Through integrated design, a new generation of high-performance office buildings is beginning to emerge that offers owners and users increased worker satisfaction and productivity, improved health, greater flexibility, and enhanced energy and environmental performance. Typically, these projects apply life-cycle analysis to optimize initial investments in architectural design, systems selection, and building construction.

Building Attributes:

An office building must have flexible and technologically advanced working environments that are safe, healthy, comfortable, durable, aesthetically-pleasing, and accessible. It must be able to accommodate the specific space and equipment needs of the tenant. Special attention should be made to the selection of interior finishes and art installations, particularly in entry spaces, conference rooms and other areas with public access.

Types of Spaces:

An office building incorporates a number of space types to meet the needs of staff and visitors.

These may include:

Offices: May be private or semi-private.

Conference Rooms

Employee/Visitor Support Spaces:

Lobby: Central location for building directory, schedules, and general information

Atria or Common Space: Informal, multi-purpose recreation and social gathering space

Cafeteria or Dining Hall

Kindergarten

Toilets or Restrooms

Parking Areas

Administrative Support Spaces:

Administrative Offices: May be private or semi-private

Operation and Maintenance Spaces
General Storage: For items such as stationery, equipment, and instructional materials.

Food Preparation Area or Kitchen

Computer/Information Technology (IT) Closets

Maintenance Closets

Table 2 fully closed offices

<table>
<thead>
<tr>
<th>Description Tenant Occupiable Areas</th>
<th>Qty.</th>
<th>Each</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Office Spaces</strong></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Enclosed Executive Offices</td>
<td>225</td>
<td>21</td>
</tr>
<tr>
<td>Enclosed Large Offices</td>
<td>150</td>
<td>14</td>
</tr>
<tr>
<td>Enclosed Small Offices</td>
<td>120</td>
<td>11.2</td>
</tr>
<tr>
<td>Open Workstations</td>
<td>80</td>
<td>7.4</td>
</tr>
<tr>
<td>Reception Desk</td>
<td>80</td>
<td>7.4</td>
</tr>
<tr>
<td><strong>Support Spaces</strong></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Reception Seating</td>
<td>200</td>
<td>18.5</td>
</tr>
<tr>
<td>&quot;Unimproved&quot; Conference Large</td>
<td>600</td>
<td>56</td>
</tr>
<tr>
<td>Conference Small</td>
<td>150</td>
<td>14</td>
</tr>
<tr>
<td>Informal Breakout Centers</td>
<td>80</td>
<td>7.4</td>
</tr>
<tr>
<td>Printer/Copier/Fax Center</td>
<td>60</td>
<td>5.6</td>
</tr>
<tr>
<td>Break Room Service Unit</td>
<td>340</td>
<td>31.6</td>
</tr>
<tr>
<td>Information Reference Centers</td>
<td>150</td>
<td>14</td>
</tr>
<tr>
<td>Supply Room</td>
<td>40</td>
<td>3.8</td>
</tr>
<tr>
<td>Work Room</td>
<td>200</td>
<td>18.6</td>
</tr>
</tbody>
</table>
Corridors:

The corridors shall meet the requirements of section 1017 IBC.

Toilets:

Table 2.1 number of toilets and lavatories for men and women

<table>
<thead>
<tr>
<th>Water Closets (Urinals see section 419.2 of the International Plumping Code)</th>
<th>Lavatories</th>
<th>Drinking Fountains(Urinals see section 419.2 of the International Plumping Code)</th>
<th>Other</th>
</tr>
</thead>
<tbody>
<tr>
<td>Male</td>
<td>Female</td>
<td>Male</td>
<td>Female</td>
</tr>
<tr>
<td>1 Per 25 for the first 50 and 1 per 50 for the remainder exceeding 50</td>
<td>1 per 40 for the first 80 and 1 per 80 for the remainder exceeding 80</td>
<td>1 per 100</td>
<td>1 Service sink</td>
</tr>
</tbody>
</table>

Also shall meet the other requirements of Afghan Architectural Code (AAC Chapter 19 Section 1901)

School Classifications:

Schools are classifying according to grade as the following in Afghanistan education system:

Primary schools (Grade 1-6)
Secondary schools (Grade 7-9)

High school (10-12)

According to the function and educational system in Afghanistan schools are classified to general and vocational schools. Radius for school access in urban norms for residential areas is 400-500m and number of student in a class is usually 40 students but it can be decreased according to grades from 1-6 grades up to 36 students, form 9-12 grade 30 and for nursery classes up to 20.

Type of spaces:

Administrative Offices

Art facility

Classroom: Day lighting is most important in classrooms, where most teaching and learning occurs.

Common areas/courtyards

Gymnasium

Health Services

Kindergarten

Lobby: Schools often show case team trophies in the foyer or feature a colorful display at child’s eye level.

Media Center: Schools are changing traditional libraries into media centers, adapting to new technology, as well as to other issues such as comfort, flexibility and maximum use of space.

Multipurpose Rooms

Restrooms

Science Facility

Area and space norms for schools:

For each student 2-2.2 m²

Classroom 65-70 m²

Practical class for chemistry, physics and biology 70-80 m²

Theory and practice class for each 70-75 m²

Class for theoretical and practical lessons of physics 70-75 m²

Class for theoretical and practical lessons of chemistry 70-75 m²

Practical class 30-35m²
Visit room (For staff and teachers) 80-85m²
Principal office 20-25m²
Vice principal 20-25m²
Store 30-35m²

A 15 x 27 m gymnasium is necessary for 10 to 12 class school.

Notes:
The corridors width shall be 2.4-4 m and the other requirements for corridors shall meet section 1018 IBC.
The minimum width of stairs shall be 1.25 m; it should not be exceeded more than 2.5 m and the travel distance from the furthest point should not be more than 25 m to stairs. Other requirements for stairs shall meet section 1009 IBC.

Toilet:

Table 3 Number of toilets, lavatories and drinking fountains for males.

<table>
<thead>
<tr>
<th></th>
<th>Male</th>
<th>Female</th>
<th>Male</th>
<th>Female</th>
</tr>
</thead>
<tbody>
<tr>
<td>Water Closets (Urinals see section 419.2 of the International Plumping Code)</td>
<td>1 Per 50</td>
<td></td>
<td>1 per 50</td>
<td></td>
</tr>
<tr>
<td>Drinking Fountains (Urinals see section 419.2 of the International Plumping Code)</td>
<td></td>
<td>1 per 100</td>
<td></td>
<td>1 Service sink</td>
</tr>
</tbody>
</table>

Also shall meet the other requirements of Afghan Architectural Code (AAC Chapter 19 Section 1901)

Notes:
Drinking fountains shall not be placed in toilet rooms.
The entrance hall area shall be 0.25 m² per student.

Health Care Center (Clinic Unit)

1.6 Health Care Facilities
For many years the health care system has experienced a continuing decline in the number of beds required for inpatients. As inpatient care is reduced, there is a corresponding trend toward increased...
outpatient health care. An outpatient clinic is less expensive to build and operate than a hospital. Space need not be devoted to "hotel functions" (the typical nursing units of hospitals), and the extensive dietetic and housekeeping areas that accompany them. Fire code requirements are considerably less demanding and mechanical and electrical systems can be simpler.

By definition, all outpatient facilities are alike in having no overnight patients. Otherwise, they can range from simple physicians' offices that provide primary care, to large, independent "hospitals without beds." Outpatient surgical facilities are now a common facility type, as the majority of surgical procedures may not require overnight hospitalization. An increasing number of community-level outpatient clinics are satellites of larger medical centers or systems, and are thus part of a complex that can emphasize continuity of care.

Almost all hospitals already include some outpatient diagnostic and treatment spaces. Many outpatient construction projects are responses to hospitals' increased outpatient workloads. Existing outpatient facilities within hospitals are expanded, overhauled, and updated. Such a renovation can serve a number of important functions in addition to that of giving the hospital a new outpatient focus. It may create improved circulation patterns or it may replace obsolete clinical areas with state of the art services for use by inpatients as well as outpatients.

Building Attributes:

Although outpatient facilities may vary greatly in size and in services offered, all should have certain common attributes:

1. Efficiency and Cost-Effectiveness

The layout of the clinic should promote staff efficiency by minimizing distance of necessary travel between frequently used spaces

2. Flexibility and expandability

3. Cleanliness and sanitation:

Both sanitation and the appearance of it are important goals for outpatient facilities. They are promoted by:

Appropriate durable finishes for each functional space. Antimicrobial surfaces should be considered for appropriate locations.

Proper detailing of such features as doorframes, casework, and finish transitions to avoid dirt-catching and hard-to-clean crevices and joints.

4. Easy Visibility

To encourage its use, the facility should be:

Easy to find, clearly visible from the approach road, with good directional signage from nearby major roads

Easy to recognize, with a welcoming image and clear, appropriately located directional signage
5. Accessibility

All areas, both inside and out, should:

Comply with the minimum requirements of the disabilities. Be easy to use by the many patients with temporary or permanent handicaps.

Ensuring grades are flat enough to allow easy movement, and sidewalks and corridors are wide enough for two wheelchairs to pass easily.

Ensuring entrance areas are designed to accommodate patients with slower adaptation rates to dark and light; marking glass walls and doors to make their presence obvious.

The minimum width of corridors shall be 1.5 m, the corridors which are used for stretchers to carry the patients shall be 2.25 m and the other requirements for corridors shall meet the section 1018 IBC.

The width of doors shall be 2.1-2.2 m for ordinary doors, 2.5 for equipment cross doors and the other requirements for door shall meet section 1008 IBC.

6. Therapeutic Environment

Although the needs of outpatients are less intense than those of hospital inpatients, an individual's visit may still be very stressful. Every effort should be made to make the outpatient visit as unthreatening and comfortable as possible, and to make the patient's experience more like going to a doctor's office than to a hospital. This can be accomplished by:

Using familiar and non-institutional materials with cheerful and varied colors and textures is emotionally beneficial.

Opening up an inwardly directed environment with views of landscaped courtyards and other outdoor spaces, particularly from waiting spaces provides emotional benefits. Nature scenes may be provided if outdoor view is unavailable.

Using cheerful and varied colors and textures, keeping in mind that some colors are inappropriate and can interfere with provider assessments of patient's pallor and skin tones, disorient older or impaired patients, or agitate patients, staff, and particularly some psychiatric patients.

Admitting ample natural light wherever feasible and using color-corrected lighting in interior spaces, which closely approximates natural daylight.

Designer should provide quiet areas for meditation, such as, in larger facilities, quiet rooms, and meditation gardens.

7. Aesthetics

Aesthetics is closely related to creating a therapeutic environment (homelike, attractive). Also, aesthetics is important to the clinic's public image and is thus an important marketing tool, both for patients and staff. Aesthetic considerations include:
Increased use of natural light, natural materials, and textures

Use of artwork

Attention to detail, proportions, color, and scale.

Homelike and intimate scale in patient rooms and offices

Signage that promotes optimal way-finding, satisfies the orientation needs of the first-time patient, allows easy navigation, and provides highly visible reference points immediately adjacent to each major entrance.

Use mechanical door openers to assist in entering and leaving the facility.

8. Security and Safety

In addition to general safety concerns of all buildings, clinics have several particular security concerns:

Protection of clinic property and assets, including drugs

Protection of patients, including incapacitated patients and staff

Violent or unstable patients need to be controlled safely

Recommend a walled compound whenever cover feasible

9. Sustainability

Locally available materials

Low maintenance choice, if high up first costs
<table>
<thead>
<tr>
<th>No.</th>
<th>Type of space</th>
<th>Quantity</th>
<th>Area m²</th>
<th>Total area m²</th>
<th>Comments</th>
</tr>
</thead>
</table>

Table 4: Space program for clinic
<table>
<thead>
<tr>
<th>No.</th>
<th>Room Type</th>
<th>No. of Rooms</th>
<th>Width</th>
<th>Depth</th>
<th>Area</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Exam Room</td>
<td>6</td>
<td>9.3</td>
<td></td>
<td>55.8</td>
</tr>
<tr>
<td>2</td>
<td>Physician Office</td>
<td>3</td>
<td>9.3</td>
<td></td>
<td>27.9</td>
</tr>
<tr>
<td>3</td>
<td>Medical Director</td>
<td>1</td>
<td>9.3</td>
<td>9.3</td>
<td>9.3</td>
</tr>
<tr>
<td>4</td>
<td>Clinical Manager</td>
<td>1</td>
<td>9.3</td>
<td>9.3</td>
<td>9.3</td>
</tr>
<tr>
<td>5</td>
<td>Charge Nurse/Office Supervisor</td>
<td>1</td>
<td>13</td>
<td>13</td>
<td>2</td>
</tr>
<tr>
<td>6</td>
<td>Social Work</td>
<td>1</td>
<td>9.3</td>
<td>9.3</td>
<td>2</td>
</tr>
<tr>
<td>7</td>
<td>Consultation Room</td>
<td>1</td>
<td>9.3</td>
<td>9.3</td>
<td>2</td>
</tr>
<tr>
<td>8</td>
<td>Treatment Room</td>
<td>1</td>
<td>18.6</td>
<td>18.6</td>
<td>33.9</td>
</tr>
<tr>
<td>9</td>
<td>Nurse Station</td>
<td>1</td>
<td>25.1</td>
<td>25.1</td>
<td>63.1</td>
</tr>
<tr>
<td>10</td>
<td>Clean Utility/Medication Room</td>
<td>1</td>
<td>11.2</td>
<td>11.2</td>
<td>12.5</td>
</tr>
<tr>
<td>11</td>
<td>Soiled Utility</td>
<td>1</td>
<td>11.2</td>
<td>11.2</td>
<td>12.5</td>
</tr>
<tr>
<td>12</td>
<td>Housekeeping</td>
<td>1</td>
<td>7</td>
<td>7</td>
<td>7</td>
</tr>
<tr>
<td>13</td>
<td>Patient Toilet</td>
<td>2</td>
<td>4.6</td>
<td>9.2</td>
<td>18.5</td>
</tr>
<tr>
<td>14</td>
<td>Staff Toilet</td>
<td>1</td>
<td>8</td>
<td>8</td>
<td>8</td>
</tr>
<tr>
<td>15</td>
<td>Employee Lounge</td>
<td>1</td>
<td>19.4</td>
<td>19.4</td>
<td>38.8</td>
</tr>
<tr>
<td>16</td>
<td>Conference Room</td>
<td>1</td>
<td>21.1</td>
<td>21.1</td>
<td>44.3</td>
</tr>
<tr>
<td>17</td>
<td>Wheelchair Storage</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>18</td>
<td>Public Women's Toilet</td>
<td>1</td>
<td>5.5</td>
<td>5.5</td>
<td>27.5</td>
</tr>
<tr>
<td>19</td>
<td>Public Men's Toilet</td>
<td>1</td>
<td>5.5</td>
<td>5.5</td>
<td>27.5</td>
</tr>
<tr>
<td>20</td>
<td>Reception/Registration</td>
<td>1</td>
<td>31.5</td>
<td>31.5</td>
<td>39</td>
</tr>
<tr>
<td>21</td>
<td>Waiting Area</td>
<td>1</td>
<td>82.1</td>
<td>82.1</td>
<td>680</td>
</tr>
<tr>
<td>22</td>
<td>Entrance Vestibule</td>
<td>1</td>
<td>18.3</td>
<td>18.3</td>
<td>33.6</td>
</tr>
<tr>
<td>23</td>
<td>IT Room</td>
<td>1</td>
<td>11</td>
<td>11</td>
<td>11</td>
</tr>
<tr>
<td>24</td>
<td>Mechanical Room</td>
<td>1</td>
<td>16.4</td>
<td>16.4</td>
<td>26.5</td>
</tr>
<tr>
<td>25</td>
<td>Electrical Room</td>
<td>1</td>
<td>7.6</td>
<td>7.6</td>
<td>7.6</td>
</tr>
<tr>
<td>26</td>
<td>Optional Space Program</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>27</td>
<td>Blood Draw/Lab &amp; Toilet Room</td>
<td>1</td>
<td>30.3</td>
<td>30.3</td>
<td>30.3</td>
</tr>
<tr>
<td>28</td>
<td>Radiography Room</td>
<td>1</td>
<td>32.3</td>
<td>32.3</td>
<td>32.3</td>
</tr>
<tr>
<td>29</td>
<td>Pharmacy</td>
<td>1</td>
<td>26.7</td>
<td>26.7</td>
<td>26.7</td>
</tr>
<tr>
<td>30</td>
<td>Anteroom</td>
<td>1</td>
<td>3.3</td>
<td>3.3</td>
<td>3.3</td>
</tr>
<tr>
<td>31</td>
<td>Pharmacy Toilet</td>
<td>1</td>
<td>5.2</td>
<td>5.2</td>
<td>5.2</td>
</tr>
</tbody>
</table>

Minimum floor area of 80 ft² (7.4 m²) required. UNMH standards require 100 ft² (9.3 m²) min. 2'-8" (0.85 m) clearance at each side and at the foot of exam table. Provide sink and counter for writing.

4'-0"(1.2 m) wide door. 3'-0" (0.91 m) clearance at each side and at the foot of treatment table. Minimum room dimension shall be 10'-0" (3 m).

Pyxis supply & medication, Secure access, Linen Cart. HVAC to accommodate heat generated by Pyxis machine.

Bio-hazard waste disposal, Trash, Soiled Linen Hamper, Hand wash sink, Countertop, Clinical Sink.

Mop sink, utility shelf, exhaust fan, cart, supplies, and 1 vacuum.

1 patient toilet for every 6 exam rooms.

Unisex, confirm plumbing fixture count per code.

Lockers, refrigerator, sink, microwave, coffee and table with seating for 8.

Seating for 12-16

Part of Waiting Area. Out of direct line of traffic.

Confirm plumbing count per code. Toilets for public use shall be conveniently accessible from waiting.

Confirm plumbing count per code. Toilets for public use shall be conveniently accessible from waiting.

2 Registration Workstations, 2 Discharge Workstations,

2.5 seats per 1 exam room, 30 seats provided 20 ft² (1.9 m²) per seat, typical. Provide provisions for drinking water.

Provide automatic sliding doors

Refer to mechanical needs

Refer to electrical needs

Refer to mechanical needs

Provide provisions for drinking water.
1.7 Community Centers

General Introduction:

Community centres are public locations where members of a community tend to gather for group activities, social support, public information, and other purposes. They may sometimes be open for the whole community or for a specialised group within the greater community. Examples of community centres for specific groups include: Islamic community centres, youth clubs etc.

By their nature community buildings must serve a variety of functions among which are:

Meetings
Parties and receptions
Exhibitions
Adult education
Child care and education

Table 5 minimum floor areas for various activities

<table>
<thead>
<tr>
<th>Space Type</th>
<th>Area per space m²</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Main hall:</strong></td>
<td></td>
</tr>
<tr>
<td>Closely seated occupant 0.46 m² (based on movable seats, usually armless, 450 mm center to center; with fixed seating and 500 mm center to center will increase to 0.6 m²)</td>
<td></td>
</tr>
<tr>
<td>Children 5–8 years (out of 2.5 school and holiday schemes, open access projects)</td>
<td></td>
</tr>
<tr>
<td><strong>Dining:</strong></td>
<td>0.9 to 1.1 m²</td>
</tr>
<tr>
<td><strong>Meeting rooms:</strong></td>
<td>2.25 up to 4 people</td>
</tr>
<tr>
<td>2 m²</td>
<td>6 people</td>
</tr>
<tr>
<td>1.55 m²</td>
<td>8–12 people</td>
</tr>
<tr>
<td>1.25 m²</td>
<td>20 people</td>
</tr>
</tbody>
</table>

Elements of the plan and spaces:

**Offices:** As necessary

**Library:** As necessary
Entrance: This should be large enough to accommodate an influx of people, such as prior to a meeting. Sign posting should be clear as many will be unfamiliar with the building. Unless there is a separate goods entrance, it should allow for bulk delivery of food and drink, display material and equipment. Consider the arrangement of the doors, the durability of surfaces and easy accesses to both the kitchen and the hall maximum area floor per occupant 10 Ft² (0.9 m²). [IBC Table 1004.1.1]

Hall: A rectangular shape is likely to be suitable for a wider range of uses than a square or any other shape. If black-out is required, pay special attention to size and location of windows; mechanical ventilation may be needed.

Meeting rooms: If more than one, make them different sizes. Alternatively, have one space that can be divided using sliding folding doors; although some of these do not provide adequate sound insulation. At least one meeting room should have direct access to the hall.

Toilets: Separate toilets will be needed for men, women and disabled people.

Table 5.2 Number of toilet and lavatories for men and women

<table>
<thead>
<tr>
<th>Water Closets (Urinals see section 419.2 of the International Plumping Code)</th>
<th>Lavatories</th>
<th>Drinking Fountains(Urinals see section 419.2 of the International Plumping Code)</th>
<th>Other</th>
</tr>
</thead>
<tbody>
<tr>
<td>Male</td>
<td>Female</td>
<td>Male</td>
<td>Female</td>
</tr>
<tr>
<td>1 Per 125</td>
<td>1 Per 65</td>
<td>1 Per 200</td>
<td></td>
</tr>
</tbody>
</table>

Also shall meet the other requirements of Afghan Architectural Code (AAC Chapter 19 Section 1901)

Kitchen: Kitchen space requirements for various functions:

Preparation

Service

Staff facilities

Cooking

Washing up

Storage (total)

A separate store should be provided for each main use:

- Kitchen (Dry, cold, freeze)
• Seating and other furniture

The kitchen store should be directly accessible from the kitchen, the others from the hall. Storage space should be as generous as space and budget will allow.

General Requirements and Definitions

Height and Areas:

The height and area of buildings shall meet the requirements of section 502, 503 and 504 IBC.

Unlimited Area Buildings:

The unlimited area buildings shall meet the requirements of section 504.4, 507.5, 507.6 and 507.9 IBC.

Interior Finishes:

The interior finishes shall meet the requirements of section 801, 802, 803, 804 and 805 IBC.

Decorative Material and Trim:

The decorative material and trim shall meet the section 806 IBC.

Fire Protection System:

The fire protection system shall meet the section 901 and 902 IBC.

Automatic Sprinkler Systems:

The automatic sprinkler systems shall meet the requirements of section 903 IBC.

Fire Alarm and Detection Systems:

The fire alarm and detection systems shall meet the requirements of section 907 IBC.

Means of Egress:

The means of egress shall meet the requirements of section 1001 and 1002 IBC.

General Means of Egress:

The general means of egress shall meet the requirements of section 1003 IBC.

Occupant Load:

The occupant load shall meet the requirements of section 1004 IBC.

Egress width:
The egress width shall meet the requirements of section 1005 IBC.

**Means of egress Illumination:**

The means of egress illumination shall meet the requirements of section 1006 IBC.

**Accessible means of egress:**

The accessible means of egress shall meet the requirements of section 1007 IBC.

**Doors, gates and Turnstiles:**

The doors, gates and turnstiles shall meet the requirements of section 1008 IBC.

**Stairways:**

The stairways shall meet the requirements of section 1009 IBC.

**Ramps:**

The ramps shall meet the requirements of section 1010 IBC.

**Exit Signs:**

The exit signs shall meet the requirements of section 1011 IBC.

**Handrails:**

The handrails shall meet the requirements of section 1012 IBC.

**Exit Access:**

The exit access shall meet the requirements of section 1014 IBC.

**Exit and Exit Access Doorways:**

The exit and exit access doorway shall meet the requirements of section 1015 IBC.

**Exit Access Travel distance:**

The exit access travel distance shall meet the requirements of section 1016 IBC.

**Corridors:**

The corridors shall meet the requirements of section 1017 IBC.

**Exits:**

The exits shall meet the requirements of section 1020 IBC.
Number of exits and Continuity:

The number of exits and continuity shall meet the requirement of section 1021.

Exit Passageway:

The exit passageway shall meet the requirements of section 1023 IBC.

1.8 Assembly Group Buildings

The assembly group buildings also shall meet the requirement of section 1028 IBC.

Toilet for the disabled:

The toilet for the disabled shall be minimum of (1.5m x1.5m). Door should open to outside to provide adequate space for a wheelchair and shall have handrails throughout.

Seismic, settlement and thermal joints:

The seismic, settlement and thermal joint are determined according to the length of building and it is usually required when length of building is more than 30-46. It is better to locate these joint in the same place. The joint with shall be 5 to 10 cm.

Distance between buildings:

The distance between two 2-3 floors building shall be twice of its height. In 1-6 floor buildings it shall be one and half of its height and more than that it shall be twice of its height.
2 Structural Concrete Requirements

2.1 Introduction

A building manual states only the minimum requirements necessary to provide for public health and safety. The manual is based on this principle. For any structure, the owner or the structural designer may require the quality of materials and construction to be higher than the minimum requirements necessary to protect the public as stated in the manual. However, lower standards are not permitted.

The manual has no legal status unless it is adopted by the government bodies having the police power to regulate building design and construction. Where the manual has not been adopted, it may serve as a reference to good practice even though it has no legal status.

For structural concrete, $f'_c$ shall not be less than 2500 psi. No maximum value of $f'_c$ shall apply unless restricted by a specific code provision.

Drawings and Specifications

(A). Copies of design drawings, typical details, and specifications for all structural concrete construction shall bear the seal of a registered engineer or architect. These drawings, details, and specifications shall show:

1) Name and date of issue of code and supplement to which design conforms;
2) Live load and other loads used in design;
3) Specified compressive strength of concrete at stated ages or stages of construction for which each part of structure is designed;
4) Specified strength or grade of reinforcement;
5) Size and location of all structural elements, reinforcement, and anchors;
6) Provision for dimensional changes resulting from creep, shrinkage, and temperature;
7) Anchorage length of reinforcement and location and length of lap splices;
8) Type and location of mechanical and welded splices of reinforcement;
9) Details and location of all contraction or isolation joints specified for plain concrete

(B). Calculations pertinent to design shall be filed with the drawings when required by the building official. Analyses and designs using computer programs shall be permitted provided design assumptions, user input, and computer-generated output are submitted. Model analysis shall be permitted to supplement calculations.

2.2 Inspection
Concrete construction shall be inspected as required by the legally adopted general building code. In the absence of such inspection requirements, concrete construction shall be inspected throughout the various work stages by or under the supervision of a registered design professional or by a qualified inspector.

The inspector shall require compliance with design drawings and specifications. Unless specified otherwise in the legally adopted general building code, inspection records shall include:

a) Quality and proportions of concrete materials and strength of concrete;
b) Construction and removal of forms and reshoring;
c) Placing of reinforcement and anchors;
d) Mixing, placing, and curing of concrete;
e) Sequence of erection and connection of precast members;
f) Any significant construction loadings on completed floors, members, or walls;
g) General progress of work.

When the ambient temperature falls below 40 F or rises above 95 F, a record shall be kept of concrete temperatures and of protection given to concrete during placement and curing.

Records of inspection required and shall be preserved by the inspecting engineer or architect for 2 years after completion of the project.

2.3 Materials

2.3.1 Tests of Materials
The building official shall have the right to order testing of any materials used in concrete construction to determine if materials are of quality specified.

Tests of materials and of concrete shall be made in accordance with standards

A complete record of tests of materials and of concrete shall be retained by the inspector for 2 years after completion of the project, and made available for inspection during the progress of the work.

Cement

Cement shall conform to one of the following specifications:

a) “Standard Specification for Portland Cement” (ASTM C 150);
b) “Standard Specification for Blended Hydraulic Cements” (ASTM C 595), excluding Types S and SA which are not intended as principal cementing constituents of structural concrete;
c) “Standard Specification for Expansive Hydraulic Cement” (ASTM C 845);

Cement used in the work shall correspond to that on which selection of concrete proportions was based.

**Aggregates**

Concrete aggregates shall conform to one of the following specifications:

a) “Standard Specification for Concrete Aggregates” (ASTM C 33);
b) “Standard Specification for Lightweight Aggregates for Structural Concrete” (ASTM C 330)

(Exception: Aggregates that have been shown by special test or actual service to produce concrete of adequate strength and durability and approved by the building official.

Nominal maximum size of coarse aggregate shall be not larger than:

a) 1/5 the narrowest dimension between sides of forms, nor
b) 1/3 the depth of slabs, nor
c) 3/4 the minimum clear spacing between individual reinforcing bars or wires, bundles of bars, individual tendons, bundled tendons, or ducts.

These limitations shall not apply if, in the judgment of the engineer, workability and methods of consolidation are such that concrete can be placed without honeycombs or voids.

**Water**

Water used in mixing concrete shall be clean and free from injurious amounts of oils, acids, alkalis, salts, organic materials, or other substances deleterious to concrete or reinforcement.

Nonpotable water shall not be used in concrete unless the following are satisfied:

Selection of concrete proportions shall be based on concrete mixes using water from the same source.

Mortar test cubes made with nonpotable mixing water shall have 7-day and 28-day strengths equal to at least 90 percent of strengths of similar specimens made with potable water. Strength test comparison shall be made on mortars, identical except for the mixing water, prepared and tested in accordance with “Standard Test Method for Compressive Strength of Hydraulic Cement Mortars (Using 2-in. or [50-mm] Cube Specimens)” (ASTM C 109).

**Steel Reinforcement**

Reinforcement shall be deformed reinforcement.
Welding of reinforcing bars shall conform to “Structural Welding Code — Reinforcing Steel,” ANSI/AWS D1.4 of the American Welding Society. Type and location of welded splices and other required welding of reinforcing bars shall be indicated on the design drawings or in the project specifications. ASTM reinforcing bar specifications, except for ASTM A 706, shall be supplemented to require a report of material properties necessary to conform to the requirements in ANSI/AWS D1.4.

Deformed Reinforcement

Deformed reinforcing bars shall conform to the requirements for deformed bars in one of the following specifications:

a) “Standard Specification for Deformed and Plain Carbon-Steel Bars for Concrete Reinforcement” (ASTM A 615);
b) “Standard Specification for Low-Alloy Steel Deformed and Plain Bars for Concrete Reinforcement” (ASTM A 706);
c) “Standard Specification for Rail-Steel and Axle-Steel Deformed Bars for Concrete Reinforcement” (ASTM A 996). Bars from rail-steel shall be Type R.

2.3.2 Plain Reinforcement

Plain bars for spiral reinforcement shall conform to the specification listed in 2.5.1(a) or (b).

Table 2.1 Designations, Diameters, Areas, and Weights of Standard Bars

<table>
<thead>
<tr>
<th>Bar No.</th>
<th>Cross-Sectional Diameter (in)</th>
<th>Cross-Sectional Area (in²)</th>
<th>Nominal Weight lb/ft</th>
</tr>
</thead>
<tbody>
<tr>
<td>Inch-Pound SI</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>3</td>
<td>0.375</td>
<td>0.11</td>
<td>0.376</td>
</tr>
<tr>
<td>4</td>
<td>0.500</td>
<td>0.2</td>
<td>0.668</td>
</tr>
<tr>
<td>5</td>
<td>0.625</td>
<td>0.31</td>
<td>1.043</td>
</tr>
<tr>
<td>6</td>
<td>0.750</td>
<td>0.44</td>
<td>1.502</td>
</tr>
<tr>
<td>7</td>
<td>0.875</td>
<td>0.60</td>
<td>2.044</td>
</tr>
<tr>
<td>8</td>
<td>1.000</td>
<td>0.79</td>
<td>2.670</td>
</tr>
<tr>
<td>9</td>
<td>1.128</td>
<td>1.00</td>
<td>3.400</td>
</tr>
<tr>
<td>10</td>
<td>1.270</td>
<td>1.27</td>
<td>4.303</td>
</tr>
<tr>
<td>11</td>
<td>1.410</td>
<td>1.56</td>
<td>5.313</td>
</tr>
<tr>
<td>14</td>
<td>1.693</td>
<td>2.25</td>
<td>7.650</td>
</tr>
<tr>
<td>18</td>
<td>2.257</td>
<td>4.00</td>
<td>13.600</td>
</tr>
</tbody>
</table>
### Table 2.2 ASTM Specifications – Grade and Min. Yield Strength

<table>
<thead>
<tr>
<th>ASTM Specification</th>
<th>Grade/Minimum Yield Strength</th>
<th>Inch-Pound (psi)</th>
<th>Metric (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>A 615 and A 615M</td>
<td>40/40000</td>
<td></td>
<td>280/280</td>
</tr>
<tr>
<td></td>
<td>60/60000</td>
<td></td>
<td>420/420</td>
</tr>
<tr>
<td></td>
<td>75/75000</td>
<td></td>
<td>520/520</td>
</tr>
<tr>
<td>A 996 and 996M</td>
<td>40/40000</td>
<td></td>
<td>280/280</td>
</tr>
<tr>
<td></td>
<td>50/50000</td>
<td></td>
<td>350/350</td>
</tr>
<tr>
<td></td>
<td>60/60000</td>
<td></td>
<td>420/420</td>
</tr>
<tr>
<td>A 706/A 706M</td>
<td>60/60000</td>
<td></td>
<td>420/420</td>
</tr>
</tbody>
</table>

### DETAILS OF REINFORCEMENT

**Standard Hooks**

The term standard hook as used in this code shall mean one of the following:

- 180-deg bend plus 4dₖ extension, but not less than 2-1/2 in. at free end of bar
- 90-deg bend plus 12dₖ extension at free end of bar

For stirrup and tie hooks

- No. 5 bar and smaller, 90-deg bend plus 6dₖ extension at free end of bar; or
- No. 6, No. 7, and No. 8 bar, 90-deg bend plus 12dₖ extension at free end of bar; or
- No. 8 bar and smaller, 135-deg bend plus 6dₖ extension at free end of bar
**Fig. 3.1 Standard Hooks for Stirrups and Tie Reinforcements**

**Fig. 3.2 Standard Hooks for Primary Reinforcement**

**Minimum Bend Diameters**

Diameter of bend measured on the inside of the bar, other than for stirrups and ties in sizes No. 3 through No. 5, shall not be less than the values in Table 3.1.
Inside diameter of bend for stirrups and ties shall not be less than $4d_b$ for No. 5 bar and smaller. For bars larger than No. 5, diameter of bend shall be in accordance with Table 3.1.

Inside diameter of bend in welded wire reinforcement for stirrups and ties shall not be less than $4d_b$ for deformed wire larger than D6 and $2d_b$ for all other wires. Bends with inside diameter of less than $8d_b$ shall not be less than $4d_b$ from nearest welded intersection.

Table 3.1 Minimum Diameters of Bend

<table>
<thead>
<tr>
<th>Bar Size</th>
<th>Minimum Diameter</th>
</tr>
</thead>
<tbody>
<tr>
<td>No. 3 through No. 8</td>
<td>$6d_b$</td>
</tr>
<tr>
<td>No. 9, No. 10, and No. 11</td>
<td>$8d_b$</td>
</tr>
<tr>
<td>No. 14 and No. 18</td>
<td>$10d_b$</td>
</tr>
</tbody>
</table>

**Bending**

All reinforcement shall be bent cold, unless otherwise permitted by the engineer.

Reinforcement partially embedded in concrete shall not be field bent, except as shown on the design drawings or permitted by the engineer.

**Surface Conditions of Reinforcement**

At the time concrete is placed, reinforcement shall be free from mud, oil, or other nonmetallic coatings that decrease bond.

**Placing Reinforcement**

Reinforcement shall be accurately placed and adequately supported before concrete is placed, and shall be secured against displacement within tolerances permitted in 3.5.

Unless otherwise specified by the registered design professional, reinforcement, including tendons, and post-tensioning ducts shall be placed within the following tolerances

Tolerance for $d$ and minimum concrete cover in flexural members, walls, and compression members shall be as follows:

<table>
<thead>
<tr>
<th>$d$ ≤ 8 in.</th>
<th>$d$ &gt; 8 in.</th>
</tr>
</thead>
<tbody>
<tr>
<td>+3/8 in.</td>
<td>±1/2 in.</td>
</tr>
<tr>
<td>−3/8 in.</td>
<td>−1/2 in.</td>
</tr>
</tbody>
</table>
Except that tolerance for the clear distance to formed soffits shall be minus 1/4 in. and tolerance for cover shall not exceed minus 1/3 the minimum concrete cover required in the design drawings and specifications.

Tolerance for longitudinal location of bends and ends of reinforcement shall be ± 2 in. Except the tolerance shall be ± 1/2 in. at the discontinuous ends of brackets and corbels, and ± 1 in. at the discontinuous ends of other members. The tolerance for minimum concrete cover of above shall also apply at discontinuous ends of members.

**Spacing Limits for Reinforcement**

The minimum clear spacing between parallel bars in a layer shall be \( d_b \), but not less than 1 in.

Where parallel reinforcement is placed in two or more layers, bars in the upper layers shall be placed directly above bars in the bottom layer with clear distance between layers not less than 1 in.

In spirally reinforced or tied reinforced compression members, clear distance between longitudinal bars shall not be less than 1.5\( d_b \) nor less than 1-1/2 in.

Clear distance limitation between bars shall apply also to the clear distance between a contact lap splice and adjacent splices or bars.

In walls and slabs other than concrete joist construction, primary flexural reinforcement shall not be spaced farther apart than three times the wall or slab thickness, nor farther apart than 18 in.

**Concrete Protection for Reinforcement**

Cast-in-place concrete (non-prestressed)

The following minimum concrete cover shall be provided for reinforcement.

<table>
<thead>
<tr>
<th>Type of Exposure</th>
<th>Minimum Cover, in</th>
</tr>
</thead>
<tbody>
<tr>
<td>a. Concrete cast against and permanently exposed to earth</td>
<td>3</td>
</tr>
<tr>
<td>b. Concrete exposed to earth or weather:</td>
<td></td>
</tr>
<tr>
<td>No. 6 through No. 18 bars</td>
<td>2</td>
</tr>
<tr>
<td>No. 5 bar, W31 or D31 wire, and smaller</td>
<td>1.5</td>
</tr>
<tr>
<td>c. Concrete not exposed to weather or in contact with ground:</td>
<td></td>
</tr>
<tr>
<td>Slabs, walls, joists:</td>
<td></td>
</tr>
<tr>
<td>No. 14 and No. 18 bars</td>
<td>1.5</td>
</tr>
<tr>
<td>No. 11 bar and smaller</td>
<td>3/4</td>
</tr>
</tbody>
</table>
Beams, columns:

Primary reinforcement, ties, stirrups, spirals 1.5

Shells, folded plate members:

No. 6 bar and larger 3/4
No. 5 bar, W31 or D31 wire, and smaller 1/5

2.3.3 Lateral Reinforcement for Compression Members

Spiral Reinforcement

Spiral reinforcement for compression members shall conform to the following:

Spirals shall consist of evenly spaced continuous bar or wire of such size and so assembled to permit handling and placing without distortion from designed dimensions.

For cast-in-place construction, size of spirals shall not be less than 3/8 in. diameter.

Clear spacing between spirals shall not exceed 3 in., nor be less than 1 in.

Anchorage of spiral reinforcement shall be provided by 1-1/2 extra turns of spiral bar or wire at each end of a spiral unit.

Spiral reinforcement shall be spliced, if needed, by any one of the following methods:

Lap splices not less than the larger of 12 in. and the length indicated below:

Deformed uncoated bar or wire........... 48d₀
Plain uncoated bar or wire................. 72d₀
epoxy-coated deformed bar or wire ... 72d₀

Spirals shall extend from top of footing or slab in any story to level of lowest horizontal reinforcement in members supported above.

Ties

Tie reinforcement for compression members shall conform to the following:

All non-prestressed bars shall be enclosed by lateral ties, at least No. 3 in size for longitudinal bars No. 10 or smaller, and at least No. 4 in size for No. 11, No. 14, No. 18, and bundled longitudinal bars. Deformed wire or welded wire reinforcement of equivalent area shall be permitted.

Vertical spacing of ties shall not exceed 16 longitudinal bar diameters, 48 tie bar or wire diameters, or least dimension of the compression member.
Ties shall be arranged such that every corner and alternate longitudinal bar shall have lateral support provided by the corner of a tie with an included angle of not more than 135 deg and no bar shall be farther than 6 in. clear on each side along the tie from such a laterally supported bar. Where longitudinal bars are located around the perimeter of a circle, a complete circular tie shall be permitted.

Ties shall be located vertically not more than one-half a tie spacing above the top of footing or slab in any story, and shall be spaced as provided herein to not more than one-half a tie spacing below the lowest horizontal reinforcement in slab or drop panel above.

**Shrinkage and Temperature Reinforcement**

Reinforcement for shrinkage and temperature stresses normal to flexural reinforcement shall be provided in structural slabs where the flexural reinforcement extends in one direction only.

Shrinkage and temperature reinforcement shall be provided in accordance with the following.

Deformed reinforcement conforming to 102.2.5.1 used for shrinkage and temperature reinforcement shall be provided in accordance with the following:

Area of shrinkage and temperature reinforcement shall provide at least the following ratios of reinforcement area to gross concrete area, but not less than 0.0014:

- Slabs where Grade 40 or 50 deformed bars are used: 0.0020
- Slabs where Grade 60 deformed bars or welded wire reinforcement are used: 0.0018
- Slabs where reinforcement with yield stress exceeding 60,000 psi measured at a yield strain of 0.35 percent is used: \((0.0018 \times 60,000)/f_y\)

Shrinkage and temperature reinforcement shall be spaced not farther apart than five times the slab thickness, nor farther apart than 18 in.

**2.3.4 Requirements for Structural Stability**

In the detailing of reinforcement and connections, members of a structure shall be effectively tied together to improve integrity of the overall structure.

For cast-in-place construction, the following shall constitute minimum requirements:

Beams along the perimeter of the structure shall have continuous reinforcement consisting of:

- At least one-sixth of the tension reinforcement required for negative moment at the support, but not less than two bars; and

- At least one-quarter of the tension reinforcement required for positive moment at mid span, but not less than two bars.

Where splices are needed to provide the required continuity, the top reinforcement shall be spliced at or near midspan and bottom reinforcement shall be spliced at or near the support. Splices shall be
Class A tension splices. The continuous reinforcement required in 1(a) and 1(b) shall be enclosed by the corners of U-stirrups having not less than 135-deg hooks around the continuous top bars, or by one-piece closed stirrups with not less than 135-deg hooks around one of the continuous top bars. Stirrups need not be extended through any joints.

In other than perimeter beams, when stirrups as defined in 2 are not provided, at least one-quarter of the positive moment reinforcement required at mid span, but not less than two bars, shall be continuous or shall be spliced over or near the support with a Class A tension splice and at non-continuous supports shall be terminated with a standard hook.

2.4 Analysis and Design

2.4.1 Design Methods

In design of structural concrete, members shall be proportioned for adequate strength in accordance with provisions of this specification, using load factors and strength reduction factors $\phi$.

Loading

Design provisions of this specification are based on the assumption that structures shall be designed to resist all applicable loads.

Service loads shall be in accordance with the general building code ASCE 7.

In design for wind and earthquake loads, integral structural parts shall be designed to resist the total lateral loads.

2.4.2 Method of Analysis

All members of frames or continuous construction shall be designed for the maximum effects of factored loads as determined by the theory of elastic analysis.

As an alternate to frame analysis, the following approximate moments and shears shall be permitted for design of continuous beams and one-way slabs (slabs reinforced to resist flexural stresses in only one direction), provided (a) through (e) are satisfied:

There are two or more spans;

Spans are approximately equal, with the larger of two adjacent spans not greater than the shorter by more than 20 percent;

Loads are uniformly distributed;

Unfactored live load, $L$, does not exceed three times unfactored dead load, $D$; and

Members are prismatic.

For calculating negative moments, $in$ is taken as the average of the adjacent clear span lengths.

Positive moment

End spans
Discontinuous end unrestrained: $W_u l_n^2 / 11$

Discontinuous end integral with support: $W_u l_n^2 / 14$

Interior spans: $W_u l_n^2 / 16$

Negative moments at exterior face of first interior support

Two spans: $W_u l_n^2 / 9$

More than two spans: $W_u l_n^2 / 10$

Negative moment at other faces of interior supports: $W_u l_n^2 / 11$

Negative moment at face of all supports for Slabs with spans not exceeding 10 ft; and beams where ratio of sum of column stiffness to beam stiffness exceeds 8

At each end of the span: $W_u l_n^2 / 12$

Negative moment at interior face of exterior support for members built integrally with supports

Where support is spandrel beam: $W_u l_n^2 / 24$

Where support is a column: $W_u l_n^2 / 16$

Shear in end members at face of first interior support

Interior support: $1.15 W_u l_n / 2$

Shear at face of all other supports: $W_u l_n / 2$
Fig. 4.1 Summary of ACI moment coefficients: (a) beams with than two spans; (b) beams with two spans only; (c) slabs with spans not exceeding 10 ft; (d) beams in which the sum of column stiffness exceeds 8 times the sum of the beam stiffness at each end of the span.

Modulus of Elasticity

Modulus of elasticity, $E_c$, for concrete shall be permitted to be taken as:

$$E_c = w_c^{1.5} \frac{33}{l_c} \quad \text{for } w_c \text{ between } 90 \text{ to } 155 \text{ lb/ft}^3$$

$$E_c = 57000 \sqrt{l_c} \quad \text{for normal weight concrete}$$

Modulus of elasticity, $E_p$, for nonprestressed reinforcement shall be permitted to be taken as 29,000,000 psi

Span Length
Span length of members not built integrally with supports shall be considered as the clear span plus the depth of the member, but need not exceed distance between centers of supports.

In analysis of frames or continuous construction for determination of moments, span length shall be taken as the distance center-to-center of supports.

**Columns**

Columns shall be designed to resist the axial forces from factored loads on all floors or roof and the maximum moment from factored loads on a single adjacent span of the floor or roof under consideration. Loading condition giving the maximum ratio of moment to axial load shall also be considered.

In frames or continuous construction, consideration shall be given to the effect of unbalanced floor or roof loads on both exterior and interior columns and of eccentric loading due to other causes.

In computing gravity load moments in columns, it shall be permitted to assume far ends of columns built integrally with the structure to be fixed.

Resistance to moments at any floor or roof level shall be provided by distributing the moment between columns immediately above and below the given floor in proportion to the relative column stiffness and conditions of restraint.

**Arrangement of Live Load**

It shall be permitted to assume that:

The live load is applied only to the floor or roof under consideration; and

The far ends of columns built integrally with the structure are considered to be fixed.

It shall be permitted to assume that the arrangement of live load is limited to combinations of:

Factored dead load on all spans with full factored live load on two adjacent spans; and

Factored dead load on all spans with full factored live load on alternate spans.

**T-Beam Construction**

In T-beam construction, the flange and web shall be built integrally or otherwise effectively bonded together.

Width of slab effective as a T-beam flange shall not exceed one-quarter of the span length of the beam, and the effective overhanging flange width on each side of the web shall not exceed:

Eight times the slab thickness; and

One-half the clear distance to the next web.

For beams with a slab on one side only, the effective overhanging flange width shall not exceed:
One-twelfth the span length of the beam;

Six times the slab thickness; and

One-half the clear distance to the next web.

2.5 **Strength and Serviceability Requirements**

2.5.1 **General**

Structures and structural members shall be designed to have design strengths at all sections at least equal to the required strengths calculated for the factored loads and forces in such combinations as are stipulated in this manual.

The basic requirement for strength design may be expressed as follows:

Design Strength ≥ Required Strength

\[ \phi (\text{Nominal Strength}) \geq U \]

**Table 5-1 Required Strength for Simplified Load Combinations**

<table>
<thead>
<tr>
<th>Condition</th>
<th>Factored load or load effect U</th>
</tr>
</thead>
<tbody>
<tr>
<td>Basic</td>
<td>( U = 1.2D + 1.6L )</td>
</tr>
<tr>
<td>Dead plus fluid, snow, rain temperature and wind</td>
<td>( U = 1.4(D + F) )</td>
</tr>
<tr>
<td></td>
<td>( U = 1.2(D + F + T) + 1.6(L + H) + 0.5(L_r \text{ or } S \text{ or } R) )</td>
</tr>
<tr>
<td></td>
<td>( U = 1.2D + 1.6(L_r \text{ or } S \text{ or } R) + (1.0L \text{ or } 0.8W) )</td>
</tr>
<tr>
<td></td>
<td>( U = 1.2D + 1.6W + 1.0L + 0.5(L_r \text{ or } S \text{ or } R) )</td>
</tr>
<tr>
<td></td>
<td>( U = 0.9D + 1.6W + 1.6H )</td>
</tr>
<tr>
<td>Earthquake</td>
<td>( U = 1.2D + 1.0E + 1.0L + 0.2S )</td>
</tr>
<tr>
<td></td>
<td>( U = 0.9D + 1.0E + 1.6H )</td>
</tr>
</tbody>
</table>
Where;

L = roof live load, L = live load, D = dead load, E = earthquake load, W = wind load,

F = fluids, H = pressure from soil, S = snow, R = rain, T = temperature.

**Design Strength**

Design strength provided by a member, its connections to other members, and its cross sections, in terms of flexure, axial load, shear, and torsion, shall be taken as the nominal strength calculated in accordance with requirements and assumptions of this manual, multiplied by the strength reduction factors $\phi$.

Strength reduction factor $\phi$ shall be as given:

- Tension-controlled sections ......................... 0.90
- Compression-controlled sections, as defined below:
  - (a) Members with spiral reinforcement ........... 0.70
  - (b) Other reinforced members ....................... 0.65
- Shear and Torsion ..................................... 0.75
- Bearing on Concrete ................................... 0.65

For sections in which the net tensile strain in the extreme tension steel at nominal strength, $\varepsilon_t$, is between the limits for compression-controlled and tension-controlled sections, $\phi$ shall be permitted to be linearly increased from that for compression-controlled sections to 0.90 as $\varepsilon_t$ increases from the compression-controlled strain limit to 0.005.
Fig. 5.1 Variation of $\phi$ with net tensile strain in extreme tension steel, $\varepsilon_t$, and $c/d_t$ for grade 60 reinforcement.

The values of $f_y$ used in design calculations shall not exceed 80,000 psi.

**Control of Deflections**

Reinforced concrete members subjected to flexure shall be designed to have adequate stiffness to limit deflections or any deformations that adversely affect strength or serviceability of a structure.

**One-Way Construction**

Minimum thickness stipulated in Table 4.2 shall apply for one-way construction not supporting or attached to partitions or other construction likely to be damaged by large deflections, unless computation of deflection indicates a lesser thickness can be used without adverse effects.

<table>
<thead>
<tr>
<th>Table 4.2 Minimum Thickness of Beams or One-way Slabs unless Deflections are Calculated</th>
</tr>
</thead>
<tbody>
<tr>
<td>Simply supported</td>
</tr>
<tr>
<td>Member</td>
</tr>
<tr>
<td>Members not supporting or attached to partitions or other construction likely to be damaged by large deflections.</td>
</tr>
<tr>
<td>Solid one way slabs</td>
</tr>
<tr>
<td>Beams or ribbed one way slabs</td>
</tr>
</tbody>
</table>
Note: Values given shall be used directly for members with normal weight concrete ($w_c = 145 \text{ lb/ft}^3$) and Grade 60 reinforcement. For other conditions, the values shall be modified as follows:

For structural lightweight concrete having unit weight, $w_c$, in the range 90-120 lb/ft$^3$, the values shall be multiplied by $(1.65 - 0.005 w_c)$ but not less than 1.09.

For $f_y$ other than 60,000 psi, the values shall be multiplied by $(0.4 + f_y/100,000)$.

**Two-way Construction**

Section 5.3.2 shall govern the minimum thickness of slabs or other two-way construction. The thickness of slabs without interior beams spanning between the supports on all sides and thickness of slabs with beams spanning between the supports on all side shall satisfy the requirements of the following paragraphs respectively.

For slabs without interior beams spanning between the supports and having a ratio of long to short span not greater than 2, the minimum thickness shall be in accordance with the provisions of Table 5.1 and shall not be less than the following values:

- Slabs without drop ...................................................................... 5 in;
- Slabs with drop panels.............................................................. 4 in.

<table>
<thead>
<tr>
<th>$f_y$, psi</th>
<th>Without drop panels</th>
<th>With drop panels</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Exterior panels</td>
<td>Interior panels</td>
</tr>
<tr>
<td>40,000</td>
<td>$\frac{l_n}{33}$</td>
<td>$\frac{l_n}{36}$</td>
</tr>
<tr>
<td>60,000</td>
<td>$\frac{l_n}{30}$</td>
<td>$\frac{l_n}{33}$</td>
</tr>
<tr>
<td>75,000</td>
<td>$\frac{l_n}{28}$</td>
<td>$\frac{l_n}{31}$</td>
</tr>
</tbody>
</table>

For two-way construction, $l_n$ is the length of clear span in the long direction, measured face-to-face of supports in slabs without beams and face-to-face of beams or other supports in other cases.
For \( f_y \) between the values given in the table, minimum thickness shall be determined by linear interpolation.

For slabs with beams spanning between the supports on all sides, the minimum thickness, \( h \), shall be as follows:

For \( \alpha_{fm} \) equal to or less than 0.2, the provisions of 5.3.3.2 shall apply;

For \( \alpha_{fm} \) greater than 0.2 but not greater than 2.0, \( h \) shall not be less than

\[
h = \frac{l_n (0.8 + \frac{f_y}{200,000})}{36 + 5\beta(\alpha_{fm} - 0.2)} \quad \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots 5.1
\]

And not less than 5 in.:

For \( \alpha_{fm} \) greater than 2.0, \( h \) shall not be less than

\[
h = \frac{l_n (0.8 + \frac{f_y}{200,000})}{36 + 9\beta} \quad \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots 5.2
\]

And not less than 3.5 in.:

At discontinuous edges, an edge beam shall be provided with a stiffness ratio \( \alpha_f \) not less than 0.80 or the minimum thickness required by Eq. (5.1) or (5.2) shall be increased by at least 10 percent in the panel with a discontinuous edge.

Where;

\( l_n \) = clear span in long direction, in.

\( \alpha_{fm} \) = average value of \( \alpha_f \) for all beams on edges of a panel.

\( \beta \) = ratio of clear span in long direction to clear span in short direction.

\[ \alpha_f = \frac{E_{cb}l_b}{E_{cs}l_s} \quad \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots 5.3 \]

In which \( E_{cb} \) and \( E_{cs} \) are the moduli of elasticity of the beam and slab concrete and \( l_b \) and \( l_s \) are the moments of inertia of the effective beam and slab.

**FLEXURE AND AXIAL LOADS**

Provisions of this Chapter shall apply for design of members subject to flexure or axial loads or to combined flexure and axial loads.
2.5.2 Design Assumptions

Strength design of members for flexure and axial loads shall be based on assumptions given through the following paragraphs and on satisfaction of applicable conditions of equilibrium and compatibility of strains.

Strain in reinforcement and concrete shall be assumed directly proportional to the distance from the neutral axis.

Maximum usable strain at extreme concrete compression fiber shall be assumed equal to 0.003.

Stress in reinforcement below \( f_y \) shall be taken as \( E_s \) times steel strain. For strains greater than that corresponding to \( f_y \), stress in reinforcement shall be considered independent of strain and equal to \( f_y \).

Tensile strength of concrete shall be neglected in axial and flexural calculations of reinforced concrete.

The relationship between concrete compressive stress distribution and concrete strain shall be assumed to be rectangular, trapezoidal, parabolic, or any other shape that results in prediction of strength in substantial agreement with results of comprehensive tests.

Requirements of the above paragraph are satisfied by an equivalent rectangular concrete stress distribution defined by the following:

Concrete stress of \( 0.85 f'_c \) shall be assumed uniformly distributed over an equivalent compression zone bounded by edges of the cross section and a straight line located parallel to the neutral axis at a distance \( \alpha = 0.85 c \) from the fiber of maximum compressive strain.

Distance from the fiber of maximum strain to the neutral axis, \( c \), shall be measured in a direction perpendicular to the neutral axis.

For \( f'_c \) between 2500 and 4000 psi, \( \beta_1 \) shall be taken as 0.85. For \( f'_c \) above 4000 psi, \( \beta_1 \) shall be reduced linearly at a rate of 0.05 for each 1000 psi of strength in excess of 4000 psi, but \( \beta_1 \) shall not be taken less than 0.65.

2.5.3 General Principles and Requirements

Design of cross sections subject to flexure or axial loads, or to combined flexure and axial loads, shall be based on stress and strain compatibility using assumptions in 102.6.1.

Balanced strain conditions exist at a cross section when tension reinforcement reaches the strain corresponding to \( f_y \), just as concrete in compression reaches its assumed ultimate strain of 0.003.

Sections are compression-controlled if the net tensile strain in the extreme tension steel, \( \varepsilon_t \), is equal to or less than the compression-controlled strain limit when the concrete in compression reaches its assumed strain limit of 0.003. The compression-controlled strain limit is the net tensile strain in the reinforcement at balanced strain conditions. For Grade 60 reinforcement, and for all prestressed reinforcement, it shall be permitted to set the compression-controlled strain limit equal to 0.002.
Sections are tension-controlled if the net tensile strain in the extreme tension steel, \( \varepsilon_t \), is equal to or greater than 0.005 when the concrete in compression reaches its assumed strain limit of 0.003. Sections with \( \varepsilon_t \) between the compression-controlled strain limit and 0.005 constitute a transition region between compression-controlled and tension-controlled sections.

Design axial strength \( \varphi P_n \) of compression members shall not be taken greater than \( \varphi P_n, \text{ max} \), computed by Eq. (6.1) or (6.2).

For non-prestressed members with spiral reinforcement conforming to 3.8.1:

\[
\varphi P_{n, \text{max}} = 0.85 \phi \left[ 0.85 f'_c \left( A_g - A_{st} \right) + f_y A_{st} \right] \quad \ldots \ldots \ldots (6.1)
\]

For non-prestressed members with tie reinforcement conforming to 3.8.2:

\[
\varphi P_{n, \text{max}} = 0.80 \phi \left[ 0.85 f'_c \left( A_g - A_{st} \right) + f_y A_{st} \right] \quad \ldots \ldots \ldots (6.2)
\]

Members subject to compressive axial load shall be designed for the maximum moment that can accompany the axial load. The factored axial force \( P_u \) at given eccentricity shall not exceed that given in 102.6.2. The maximum factored moment \( M_u \) shall be magnified for slenderness effects.

Minimum Reinforcement of Flexural Members

At every section of a flexural member where tensile reinforcement is required by analysis, \( A_s \) provided shall not be less than that given by

\[
A_{s, \text{min}} = \frac{3 \sqrt{f'_c}}{f_y} b_w d \geq \frac{200 b_w d}{f_y} \quad \ldots \ldots \ldots \ldots \ldots \ldots \ldots (6.3)
\]

For structural slabs and footings of uniform thickness, \( A_{s, \text{min}} \) in the direction of the span shall be the same as that required by 102.3.10 (1). Maximum spacing of this reinforcement shall not exceed three the thickness, nor 18 in.

Limits for Reinforcement of Compression Members

Area of longitudinal reinforcement, \( A_{st} \), for non-composite compression members shall be not less than 0.01\( A_g \) or more than 0.08\( A_g \).

Minimum number of longitudinal bars in compression members shall be 4 for bars within rectangular or circular ties, and 6 for bars enclosed by spirals.

Volumetric spiral reinforcement ratio, \( \rho_s \), shall be not less than the value given by

\[
\rho_s = 0.45 \left( \frac{A_g}{A_{ch}} - 1 \right) \frac{f'_c}{f_y} \quad \ldots \ldots \ldots \ldots \ldots \ldots \ldots (6.4)
\]

Where the value of \( f_y \) used in Eq. (6-4) shall not exceed 100,000 psi. For \( f_y \) greater than 60,000 psi, lap splices according to 102.3.8.1 (a-1) shall not be used.
Where $f_{yts}$ is the yield strength of spiral steel, $A_g$ and $A_c$ are, respectively, the gross and core concrete areas.

### 2.5.4 Shear

The design of beams for shear is based on the relation

$$V_u \leq \phi V_n \quad (7.1)$$

Where $V_n$ is the factored shear force at the section considered and $V_u$ is nominal shear strength computed by

$$V_n = V_c + V_s \quad (7.2)$$

For vertical stirrups:

$$V_u \leq \phi V_c + \frac{\phi A_v f_{yts} d}{s} \quad (7.3a)$$

For inclined bars:

$$V_u \leq \phi V_c + \frac{\phi A_v f_{yts} d (\sin \alpha + \cos \alpha)}{s} \quad (7.3b)$$

Where $V_c$ is nominal shear strength provided by concrete, and $V_s$ is nominal shear strength provided by shear reinforcement.

**Shear Strength provided by Concrete**

For members subject to shear and flexure only,

$$V_c = 2 \lambda \sqrt{f_c} b_w d \quad (7.4)$$

$V_c$ shall be permitted to be computed by the more detailed calculation. For members subject to shear and flexure only,

$$V_c = (1.9 \lambda \sqrt{f_c} + 2500 \rho_w \frac{V_u d}{M_u}) b_w d \quad (7.5)$$

But not greater than $3.5 \lambda \sqrt{f_c} b_w d$. When computing $V_c$ by Eq. (7.5), $V_u d / M_u$ shall not be taken greater than 1.0, where $M_u$ occurs simultaneously with $V_u$ at section considered.

Where $\rho_w = $ longitudinal reinforcement ratio $A_s / b_w d$ or $A_s / b_d \cdot \lambda = $ modification factor for normal weight concrete $\lambda=1.0$.

**Shear Strength Provided by the Shear Reinforcement**

Spacing limits for shear reinforcement
Spacing of shear reinforcement placed perpendicular to axis of member shall not exceed \( d/2 \) in, nor 24 in.

Inclined stirrups and bent longitudinal reinforcement shall be so spaced that every 45-degree line, extending toward the reaction from mid-depth of member \( d/2 \) longitudinal tension reinforcement, shall be crossed by at least one line of shear reinforcement.

Where \( V_s \) exceeds \( 4\sqrt{f'_c b_w d} \), maximum spacing given in 102.7.3.1 shall be reduced by one-half.

**Minimum shear reinforcement**

If \( V_u \), the shear force at factored loads, is no larger than \( \phi V_c \), calculated by Eq. (7.4) or alternatively by Eq. (7.5), then theoretically no web reinforcement is required. Even in such a case, however, ACI code requires provision of at least a minimum area of web reinforcement equal to

\[
A_{w,\text{min}} = 0.75 \sqrt{f'_c} \frac{b_w S}{f_y t} \geq 50 \frac{b_w S}{f_y t} \ldots \ldots \ldots \ldots (7.6)
\]

\( S \) = longitudinal spacing of web reinforcement, in.

\( f_y t \) = yield strength of web steel, psi

\( A_{w,\text{min}} \) = total cross-sectional area of web steel within distance \( s \), in²

This provision holds unless \( V_u \) is one-half or less of the design shear strength provided by the concrete \( \phi V_c \). Specific exceptions to this requirement for minimum web steel are made for slabs and footings; for concrete joist construction; for beams with total depth \( h \) not greater than 10 in.; and for beams integral with slabs with \( h \) not greater than 24 in. and not greater than the larger of 2.5 times the thickness of the flange and 0.5 times the thickness of the web. These members are excluded because of their capacity to redistribute internal forces before diagonal tension failure, as confirmed by both tests and successful design experience. In addition, beams constructed of steel fiber reinforced, normal weight concrete with \( f'_c \) not exceeding 6000 psi, total depth \( h \) not greater than 24 in., and \( V_u \) not greater than \( \phi 2\sqrt{f'_c} b_w d \) are not required to meet the requirements for minimum web reinforcement because beams meeting these requirements have been shown to have shear strength in excess of \( 3.5\sqrt{f'_c} b_w d \).

**Region in Which Web Reinforcement is required**

If the required shear strength \( V_u \) is greater than the design shear strength \( \phi V_c \) provided by the concrete in any portion of a beam, there is a theoretical requirement for web reinforcement. Elsewhere in the span, web steel at least equal to the amount given by Eq. (7.6) must be provided, unless the factored shear force is less than \( \frac{1}{2} \phi V_c \).

The portion of any span through which web reinforcement is theoretically necessary can be found from the shear diagram for the span, superimposing a plot of the shear strength of the concrete. Where the shear force \( V_u \) exceeds \( \phi V_c \), shear reinforcement must provide for the excess. The
additional length through which at least the minimum web steel is needed can be found by superimposing a plot of \( \phi V_c / 2 \).

### 2.5.5 Design of Web Reinforcement

The design of web reinforcement, under the provisions of the ACI Code, is based on the Eq. (7.3a) for vertical stirrups and Eq. (7.3b) for inclined stirrups or bent bars. In design, it is usually convenient to select a trial web-steel area \( A_v \) based on standard stirrup sizes [usually in the range of No. 3 to 5 for stirrups, and according to the longitudinal bar size for bent-up bars], for which the required spacing \( s \) can be found. Equating the design strength \( \phi V_u \) to the required strength \( V_u \) and transposing Eqs. (7.3a) and (7.3b) accordingly, one finds the required spacing of web reinforcement is, for vertical stirrups,

\[
s = \frac{\phi A_v f_{yt} d}{V_u - \phi V_c} \quad \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots (7.7a)
\]

And for bent bars

\[
s = \frac{\phi A_v f_{yt} d(\sin \alpha + \cos \alpha)}{V_u - \phi V_c} \quad \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots (7.7b)
\]

It is undesirable to space vertical stirrups closer than about 4 in.; the size of the stirrups should be chosen to avoid a closer spacing. When vertical stirrups are required over a comparatively short distance, it is good practice to space them uniformly over the entire distance, the spacing being calculated for the point of greatest shear (minimum spacing). If the web reinforcement is required over a long distance, and if the shear varies materially throughout this distance, it is more economical to compute the spacing required at several sections and to place the stirrups accordingly, in groups of varying spacing.

Where web reinforcement is needed, the code requires it to be spaced so that every 45° line, representing a potential diagonal crack and extending from the mid depth \( d/2 \) of the member to the longitudinal tension bars, is crossed by at least one line of web reinforcement; in addition, the code specifies a maximum spacing of 24 in. When \( V_s \) exceeds \( 4 \sqrt{f_c} b_w d \), these maximum spacings are halved.

For design purposes, Eq. (7.6) giving the minimum web-steel area \( A_v \) is more conveniently inverted to permit calculation of maximum spacing \( s \) for the selected \( A_v \).

Thus, for the usual case of vertical stirrups, with \( V_s \leq 4 \sqrt{f_c} b_w d \), the maximum spacing of stirrups is the smallest of

\[
s_{\text{max}} = \frac{A_v f_{yt}}{0.75 \sqrt{f_c} b_w} \quad \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots (7.8a)
\]

\[
s_{\text{max}} = \frac{d}{2} \quad \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots (7.8b)
\]

\[
s_{\text{max}} = 24 \text{ in} \quad \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots (7.8c)
\]
2.5.6 Development and Splices of Reinforcement

Development of Tension Reinforcement

Basic Equation for Development of Tension Bars

\[ l_d = \left( \frac{3}{40} \frac{f_y}{\lambda \sqrt{f_c}} \left( \frac{(c_b + K_{tr})}{d_b} \right)^2 \right) d_b \quad \ldots \quad (8.1) \]

In which the term \( \frac{(c_b + K_{tr})}{d_b} \) shall not be taken greater than 2.5. in Eq. (7.9), terms are defined and values established as follows.

\( \psi_r = \) Reinforcement location factor

Horizontal reinforcement so placed that more than 12 in. of fresh concrete is cast in the member below the development length or

- Splice: 1.3
- Other situation: 1.0

\( \psi_e = \) Coating factor

- Epoxy-coated bars or wires with cover less than 3d_b or clear spacing less than 6d_b: 1.5
- All other epoxy-coated bars or wires: 1.2
- Uncoated and zinc-coated (galvanized) reinforcement: 1.0

\( \psi_s = \) Reinforcement size factor

- No.6 (No.19) and smaller bars: 0.8
- No.7 (No.22) and larger bars: 1.0

\( \lambda = \) Lightweight aggregate concrete factor

- When lightweight aggregate concrete is used: 0.75
- When normal weight concrete is used: 1.0

\( c = \) Spacing or cover dimension, in.

\( K_{tr} = \) Transverse reinforcement index: \( 40 A_{tr} / sn \)

Where \( A_{tr} = \) total cross-sectional area of all transverse reinforcement that is within the spacing \( s \) and that crosses the potential plane of splitting through the reinforcement being developed, in²

\( S = \) maximum spacing of transverse reinforcement within \( l_d \) center to center, in

\( n = \) number of bars being developed along the plane of splitting
The value of $\sqrt{f_c'}$ is not to be taken greater than 100 psi.

**Simplified Equations for Development Length**

**Table 8.1 Simplified tension development length in bar diameters**

<table>
<thead>
<tr>
<th>No. 6 (No. 19) and Smaller Bars and Deformed Wires</th>
<th>No. 7 (No. 22) and Larger Bars</th>
</tr>
</thead>
<tbody>
<tr>
<td>Clear spacing of bars being developed or spliced $\geq d_b$, clear cover $\geq d_b$, and stirrups or ties thought $l_d$ not less than the code minimum</td>
<td>$l_d = \left(\frac{f_y \Psi_s \Psi_e}{25\lambda \sqrt{f_c'}}\right) d_b$</td>
</tr>
<tr>
<td>$l_d = \left(\frac{3f_y \Psi_s \Psi_e}{50\lambda \sqrt{f_c'}}\right) d_b$</td>
<td>$l_d = \left(\frac{3f_y \Psi_s \Psi_e}{40\lambda \sqrt{f_c'}}\right) d_b$</td>
</tr>
</tbody>
</table>

**Development of Bars in Compression**

The development length in compression is the greater of

$$l_{dc} = \frac{0.02f_y}{\lambda \sqrt{f_c'}} d_b \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots (8.2)$$

And

$$l_{dc} = 0.0003f_y d_b \ldots \ldots \ldots \ldots \ldots \ldots \ldots (8.3)$$

In no case is $l_{dc}$ to be less than 8 in., according to the ACI code.

**2.5.7 Bar Cutoff and Bend Points in Beams**

Practical Considerations and ACI Code Requirements:

ACI code requires that every bar be continued at least a distance equal to the effective depth of the beam or 12 bar diameters (whichever is larger) beyond the point at which it is theoretically no longer required to resist stress, except at supports of simple spans and at the free end of cantilevers.

In addition, it is necessary that the calculated stress in the steel at each section be developed by adequate embedded length or end anchorage, or a combination of the two. For the usual case, with no special end anchorage, this means that the full development length $l_d$ must be provided beyond critical sections at which peak stress exists in the bars. These critical sections are located at points of
maximum moment and at points where adjacent terminated reinforcement is no longer needed to resist bending.

Further reflecting the possible change in peak stress location, ACI Code requires that at least one-third of the positive-moment steel (one-fourth in continuous spans) be continued uninterrupted along the same face of the beam a distance at least 6 inch into the support. When a flexural member is a part of a primary lateral load resisting system, positive-moment reinforcement required to be extended into the support must be anchored to develop the yield strength of the bars at the face of support to account for the possibility of reversal of moment at the supports. According to ACI Code, at least one-third of the total reinforcement provided for negative moment at the support must be extended beyond the extreme position of the point of inflection a distance not less than one-sixteenth the clear span, or d, or 12d, whichever is greatest.

Requirements for bar cut off or bend point locations are summarized in Fig. 8.1. If negative bars L are to be cut off, they must extend a full development length d beyond the face of the support. In addition, they must extend a distance d or 12d beyond the theoretical point of cut off defined by the moment diagram. The remaining negative bars M (at least one-third of the total negative area) must extend at least ld beyond the theoretical point of cutoff of bars L and in addition must extend d or 12d, or l /16 (whichever is greatest) past the point of inflection of the negative-moment diagram.

If the positive bars N are to be cut off, they must project ld past the point of theoretical maximum moment, as well as d or 12d, beyond the cutoff point from the positive-moment diagram. The remaining positive bars O must extend ld past the theoretical point of cut off ob bars N and must extend at least 6 inches into the face of the support.

When bars are cut off in a tension zone, there is a tendency toward the formation of premature flexural and diagonal tension cracks in the vicinity of the cut end. This may result in a reduction of shear capacity and a loss in overall ductility of the beam.

ACI Code requires special precautions, specifying that no flexural bar shall be terminated in a tension zone unless one of the following conditions is satisfied.

The shear is not over two-thirds of the design strength $\phi V_n$.

Stirrups in excess of those normally required are provided over a distance along each terminated bar from the point of cut off equal to $3/4d$. These “blinder” stirrups shall provide an area $A_v \geq 60b_w s / f_{y_t}$. In addition, the stirrup spacing shall not exceed $d / \beta_b$, where $\beta_b$ is the ratio of the area of bars cut off to the total area of bars at the section.

The continuing bars, if No. 11 (No. 36) or smaller, provide twice the area required for flexural at that point, and the shear does not exceed three-quarters of the design strength $\phi V_n$.

As an alternative to cutting off the steel, tension bars may be anchored by bending them across the web and making them continuous with the reinforcement on the opposite face. Although this leads to some complication in detailing and placing the steel, thus adding to construction cost, some engineers prefer the arrangement because added insurance is provided against the spread of diagonal tension cracks. In some cases, particularly for relatively deep beams in which a large
percentage of the total bottom steel is to be bent, it may be impossible to locate the bend-up points for bottom bars far enough from the support for the same bars to meet the requirements for top steel. The theoretical points of bend should be checked carefully for both bottom and top steel.

Because the determination of cut off or bend points may be rather tedious, particularly for frames that have been analyzed by elastic methods rather than by moment coefficients, many designers specify that bars be cut off or bent at more or less arbitrarily defined points that experience has proved to be safe. For nearly equal spans, uniformly loaded, in which not more than about one-half the tensile steel is to be cut off or bent, the locations shown in Fig. 8.2 are satisfactory. Note, in Fig. 8.2, that the beam at the exterior support at the left is shown to be simply supported. If the beam is monolithic with exterior columns or with a concrete wall at that end, details for a typical interior span could be used for the end span as well.
requirements of the ACI Code.

Fig. 8.2. Cutoff or bend points for bars in approximately equal spans with uniformly distributed loads.

**Development of Standard Hooks in Tension**

Development length for deformed bars in tension terminating in a standard hook (see 3.1), $l_{dh}$, shall be determined from 1 and the applicable modification factors of 2, but $l_{dh}$ shall not be less than the larger of $8d_b$ and 6 in.

For deformed bars, $l_{dh}$ shall be:

$$l_{dh} = \frac{0.02\psi_e f_y}{\lambda \sqrt{f'_c} d_b} \ldots \ldots (8.4)$$

With $\psi_e$ taken as 1.2 for epoxy-coated reinforcement, and $\lambda$ taken as 0.75 for lightweight concrete. For other cases, $\psi_e$ and $\lambda$ shall be taken as 1.0.

Length $l_{dh}$ in 1 shall be permitted to be multiplied by the following applicable factors:

For No. 11 bar and smaller hooks with side cover (normal to plane of hook) not less than 2.5 inch., and for 90-degree hook with cover on bar extension beyond hook not less than 2 inch……………………………………………………0.7

For 90-degree hooks of No. 11 and smaller bars that are either enclosed within ties or stirrups perpendicular to the bar being developed, spaced not greater than $3d_b$ along $l_{dh}$; or enclosed within ties or stirrups parallel to the bar being developed, spaced not greater than $3d_b$ along the length of the tail extension of the hook plus bend…………………………………………………………….0.8
For 180-degree hooks of No. 11 and smaller bars that are enclosed within ties or stirrups perpendicular to the bar being developed, spaced not greater than $3d_b$ along $l_{dh}$

Where anchorage or development for $f_y$ is not specifically required, reinforcement in excess of that required by analysis

In above $d_b$ is the diameter of the hooked bar, and the first tie or stirrups shall enclose the bent portion of the hook, within $2d_b$ of the outside of the bend.

For bars being developed by a standard hook at discontinuous ends of members with both side cover and top (or bottom) cover over hook less than 2-1/2 in., the hooked bar shall be enclosed within ties or stirrups perpendicular to the bar being developed, spaced not greater than $3d_b$ along $l_{dh}$. The first tie or stirrup shall enclose the bent portion of the hook, within $2d_b$ of the outside of the bend, where $d_b$ is the diameter of the hooked bar. For this case, the factors of 2(b) and (c) shall not apply.

Hooks shall not be considered effective in developing bars in compression.

---

Fig. 8.3 Standard bar hooks: (a) main reinforcement, (b) Stirrups and ties
2.5.8 Splices of Reinforcement

In general, reinforcing bars are stocked by suppliers in lengths of 60 ft for bars from No. 5 to No. 18 (No. 16 to No. 57) and in 20 or 40 ft lengths for smaller sizes. For this reason, and because it is often more convenient to work with shorter bar lengths, it is frequently necessary to splice bars in the field. Splices in reinforcement at points of maximum stress should be avoided, and when splices are used, they should be staggered, although neither condition is practical, for example, in compression splices in columns.

Splices for No. 11 (No. 36) bars and smaller are usually made simply by lapping the bars a sufficient distance to transfer stress by bond from one bar to the other. The lapped bars are usually placed in contact and lightly wired so that they stay in position and the concrete is placed. Alternatively, splicing may be accomplished by welding or by sleeves or mechanical devices. ACI Code prohibits use of lapped splices for bars larger than No. 11 (No. 36), except that No. 14 and No. 18 (No. 43 and No. 57) bars may be lapped in compression with No. 11 (No. 36) and smaller bars per ACI Code. For bars that will carry only compression, it is possible to transfer load by end bearing of square cut ends, if the bars are accurately held in position by a sleeve or other device.
Splices of Deformed Bars and Deformed Wire in Tension

Minimum length of lap for tension lap splices shall be as required for Class A or B splice, but not less than 12 in., where:

Class A Splice: .......................................................... 1.0\(l_d\)

Class B Splice: .......................................................... 1.3\(l_d\)

Where \(l_d\) is calculated in accordance with 8.1 to develop \(f_{y}\), but without the 12 in. minimum of 8.1.2 and without the modification factor.

Lap splices of deformed bars in tension shall be Class B splices except that Class A splices are allowed when:

The area of reinforcement provided is at least twice that required by analysis over the entire length of the splice; and

One-half or less of the total reinforcement is spliced within the required lap length.

The effect of these requirements is to encourage designers to locate splices away from regions of maximum stress, to a location where the actual steel area is at least twice that required by analysis, and to stagger splices.

When bars of different size are lap spliced in tension, splice length shall be the larger of \(l_d\) of larger bar and tension lap splice length of smaller bar.

Splices of Deformed Bars in Compression

Compression lap splice length shall be:

\[
\text{for bars with } f_y \leq 60,000 \text{ psi } \quad 0.0005f_yd_b
\]
\[
\text{for bars with } f_y > 60,000 \text{ psi } \quad (0.0009f_y - 24)d_b
\]

But not less than 12 in. for \(f_{c'}\) less than 3000 psi, length of lap shall be increased by one-third.

When bars of different size are lap spliced in compression, splice length shall be the larger of \(l_{dc}\) of larger bar and compression lap splice length of smaller bar. Lap splices of No. 14 and No. 18 bars to No. 11 and smaller bars shall be permitted.

Splice Requirements for Columns

Lap splices, mechanical splices, butt welded splices, and end-bearing splices shall be used with the limitations of 1. A splice shall satisfy requirements for all load combinations for the column.

2.5.9 Lap Splices in Columns

Where the bar stress due to factored loads is compressive, lap splices shall conform to 8.4.2, and, where applicable, to 1(d) or 1(e).
Where the bar stress due to factored loads is tensile and does not exceed $0.5f_y$ in tension, lap splices shall be Class B tension lap splices if more than one-half of the bars are spliced at any section, or Class A tension lap splices if half or fewer of the bars are spliced at any section and alternate lap splices are staggered by $l_d$.

Where the bar stress due to factored loads is greater than $0.5f_y$ in tension, lap splices shall be Class B tension lap splices.

In tied reinforced compression members, where ties throughout the lap splice length have an effective area not less than $0.0015hs$ in both directions, lap splice length shall be permitted to be multiplied by 0.83, but lap length shall not be less than 12 in. Tie legs perpendicular to dimension $h$ shall be used in determining effective area.

In spirally reinforced compression members, lap splice length of bars within a spiral shall be permitted to be multiplied by 0.75, but lap length shall not be less than 12 in.

### 2.6 Footings

#### 2.6.1 Scope
Provisions of this Chapter shall apply for design of isolated footings and, where applicable, to combined footings and mats.

#### 2.6.2 Loads and Reactions
Footings shall be proportioned to resist the factored loads and induced reactions, in accordance with the appropriate design requirements of this manual and as provided in Chapter 9.

Base area of footing shall be determined from un-factored forces and moments transmitted by footing to soil.

#### 2.6.3 Moment in Footings
External moment on any section of a footing shall be determined by passing a vertical plane through the footing, and computing the moment of the forces acting over entire area of footing on one side of that vertical plane.

Maximum factored moment, $M_u$, for an isolated footing shall be computed as prescribed above at critical sections located as follows:

At face of column, pedestal, or wall, for footings supporting a concrete column, pedestal, or wall;

Halfway between middle and edge of wall, for footings supporting a masonry wall;

Halfway between face of column and edge of steel base plate, for footings supporting a column with steel base plate.
In one-way square or rectangular footings and two-way square footings, flexural reinforcement shall be distributed uniformly across the entire width of the footing. For two-way rectangular footings, the reinforcement must be distributed as shown in Table 9.1.

Table 9.1 Distribution of Flexural Reinforcement

<table>
<thead>
<tr>
<th>Footing Type</th>
<th>Square Footing</th>
<th>Rectangular Footing</th>
</tr>
</thead>
<tbody>
<tr>
<td>One-way</td>
<td><img src="a" alt="Diagram" /></td>
<td><img src="b" alt="Diagram" /></td>
</tr>
<tr>
<td>Two-way</td>
<td><img src="c" alt="Diagram" /></td>
<td><img src="d" alt="Diagram" /></td>
</tr>
</tbody>
</table>
Shear in Footings

Two different types of shear strength are distinguished in footings: Two-way, or Punching, shear and One-way, or Beam, shear.

A column supported by the slab shown in Fig. 9.2 tends to punch through that slab because of the shear stresses that act in the footing around the perimeter of the column. At the same time, the concentrated compressive stresses from the column spread out into the footing so that the concrete adjacent to the column is in vertical or slightly inclined compression, in addition to shear. As a consequence, if failure occurs, the fracture takes the form of the truncated pyramid shown in Fig. 9.2 with sides sloping outward at an angle approaching 45°. The average shear stress in the concrete that fails in this manner can be taken as that acting on vertical planes laid through the footing around the column on a perimeter a distance d/2 from the faces of the column (vertical section through abcd in Fig. 9.3). The concrete subject to this shear $\nu_{c1}$ is also in vertical compression from the stresses spreading out from the column, and in horizontal compression in both major directions because of the biaxial bending moments in the footing. This triaxiality of stress increases the shear strength of the concrete. Tests of footings and of flat slabs have shown, correspondingly, that for punching-type failures the shear stress computed on the critical perimeter area is larger than in one-way action.

ACI Code equations give the normal punching-shear strength on this perimeter:

$$V_c = 4\lambda \sqrt{f_c} b_d d$$

Except for columns of elongated cross section, for which
For cases in which the ratio of critical perimeter to slab depth $b_o/d$ is very large,

$$V_c = \left(2 + \frac{4}{\beta}\right) \lambda \sqrt{f_c^*} b_o d \quad \cdots \cdots \cdots \cdots \cdots \cdots (9.1b)$$

Where $b_o$ is the perimeter $abdd$ in Fig. 9.3; $\beta = \frac{a}{b}$ is the ratio of the long to short sides of the column cross section; and $\alpha_s$ is 40 for interior loading, 30 for edge loading, 20 for corner loading of a footing. The punching-shear strength of the footing is to be taken as the smallest of the values given by Eqs. (9.1a), (9.1b), and (9.1c); and the design strength is $\varnothing V_c$, as usual, where $\varnothing = 0.75$ for shear.

The application of Eqs. (9.1) to punching shear in footings under columns with other than a rectangular cross section is shown in Fig. 9.4. For such situations, ACI Code indicates that the perimeter $b_o$ must be of minimum length but need not approach closer than $d/2$ to the perimeter of the actual loaded area.

Shear failures can also occur, as in beams or one-way slabs, at a section a distance $d$ from the face of the column, such as section $ef$ of Fig. 9.3. Just as in beams and one-way slabs, the nominal shear strength is given by Eq. (7.5), that is,

$$V_c = \left(1.9 \lambda \sqrt{f_c^*} + 2500 \rho_w \frac{V_u d}{M_{uu}} \right) b d \leq 3.5 \lambda \sqrt{f_c^*} bd. \cdots \cdots \cdots \cdots (9.2a)$$

Where $b =$ width of footing at distance $d$ from face of column

$V_u =$ total factored shear force on that section

$q_u =$ times footing area outside that section (area $efgh$ in Fig. 9.3)

$M_{uu} =$ moment of $V_u$ about $ef$

In footing design, the simpler and somewhat more conservative Eq. (7.4) is generally used, i.e.,

$$V_c = 2 \lambda \sqrt{f_c^*} b_w d \quad \cdots \cdots \cdots \cdots (9.2b)$$

The required depth of footing $d$ is then calculated from the usual equation

$$V_u \leq \varnothing V_c \quad \cdots \cdots \cdots \cdots \cdots \cdots \cdots \cdots \cdots \cdots (9.3)$$

Applied separately in connection with Eqs. (9.1) and (9.2). For Eq. (9.1), $V_u = V_{u1}$ is the total upward pressure caused by $q_u$ on the area outside the perimeter $abcd$ in Fig. 9.3. For Eq. (9.2), $V_u = V_{u2}$ is the total upward pressure on the area $efgh$ outside the section $ef$ in Fig. 9.3. The required depth is then the larger of those calculated from either Eq. (9.1) or (9.2).
Transfer of Forces at Base of Column

When a column rests on a footing or pedestal, it transfers its load to only a part of the total area of the supporting member. The adjacent footing concrete provides lateral support to the directly loaded part of the concrete. This causes triaxial compressive stresses that increase the strength of the concrete that is loaded directly under the column. Based on test, ACI Code provides that when the supporting area is wider than the loaded area on all sides, the design bearing strength is

\[ \phi P_n = \phi (0.85 f'_c A_1) \frac{A_2}{A_1} \leq \phi (0.85 f'_c A_1) x 2 \ldots \ldots \ldots \ldots \ldots (9.4) \]

For bearing on concrete, \( \phi = 0.65 \), \( f'_c \) is the specified compressive strength of the footing concrete, which frequently is less than that of the column, and \( A_1 \) is the loaded area. \( A_2 \) is the area of the lower base of the larger frustum of a pyramid, cone, or tapered wedge contained wholly within the support and having for its upper base the loaded area and having side slopes of 1 vertical to 2 horizontal. The meaning of this definition of \( A_2 \) may clarified by Fig. 9.5. For the somewhat unusual case shown, where the top of the support is stepped, a step that is deeper or closer to the loaded area than that shown may result in reduction in the value of \( A_2 \). A footing for which the top surface is sloped away from the loaded area more steeply than 1 to 2 will result in a value of \( A_2 \) equal to \( A_1 \). In most usual cases, for which the top of the footing is flat and the sides are vertical, \( A_2 \) is simply the maximum area of the portion of the supporting surface that is geometrically similar to, and concentrate with, the loaded area.
All axial forces and bending moments that act at the bottom section of a column must be transferred to the footing at the bearing surface by compression in the concrete and by reinforcement. With respect to the reinforcement, this may be done either by extending the column bars into the footing or by providing dowels that are embedded in the footing and project above it. In the latter case, the column bars merely rest on the footing and in most cases are tied to dowels. This results in a simpler construction procedure then extending the column bars into the footing. To ensure the integrity of the junction between column and footing, ACI Code requires that the minimum area of reinforcement that crosses the bearing surface (dowels or column bars) be 0.005 times the gross area of the supported column. The length of dowels or bars of diameter $d_b$ must be sufficient on both sides of the bearing surface to provide the required development length for compression bars, that is, $l_{dc} = 0.02 f_c d_b / f_y$ and $\geq 0.0003 f_y d_b$. In addition, if dowels are used, the lapped length must be at least that required for a lap splices in compression.

Minimum Footing Depth

Depth of footing above bottom reinforcement shall not be less than 6 in. for footings on soil, nor less than 12 in. for footings on piles.

**Combined Footings and Mats**

Footings supporting more than one column, pedestal, or wall (combined footings or mats) shall be proportioned to resist the factored loads and induced reactions, in accordance with appropriate design requirements of the code.

The direct design method shall not be used for design of combined footings and mats.

Distribution of soil pressure under combined footings and mats shall be consistent with properties of the soil and the structure and with established principles of soil mechanics.
Minimum reinforcing steel in non-prestressed mat foundations shall meet the requirements of 102.3.9 in each principal direction. Maximum spacing shall not exceed 18 in.

3 Water and Waste Water Management

3.1 Introduction to Hydraulic design
Hydraulic design of structures includes water supply and waste water management.

Water Supply: The design should consider modeling existing locally available water resources, in case of local water supply is not available. The dwells or any other water supply method can be adopted where clean and drinkable water can be supplied; the water should be supply for use after three laboratory tests, Physical, chemical and biological.
Waste Water: The waste water should be diverted to locally existing sewer and fouls system by applying an adequate design system. If the existing system is not available then the sump should be design with considerations for the risk and safety of the local people.

Cost Considerations for Hydraulic Design

The following factors should be considered during design for economic development.

Cost: Size of water distribution network to satisfy the function requirements should be based on the least cost.

Type: Pump type (single or parallel) to determine the efficiency of pump and minimize the initial and life time cost.

Quality: Various commercially available sizes and qualities of material should be checked to determine the suitability; high cost is a factor quality cannot be compromised.

General Hydraulic Design Considerations

The following factors should be considered to develop adequate hydraulics design.

- Topographic features of terrain.
- Component parameters and fluid properties and required network size.
- Consumer water demand: residential, commercial and industrial.
- Design Period or Length of time, the life time and the period of requirement of purpose project should be determined.
- It should be considered, if the proposed water project is to improve the water quality, the water demand may increase from the existing water supply system.
- Local factors should be considered such as: climate, industrial activity, standard of living, water price and age of community and availability of private wells in the area.
- Analysis availability of water over long periods of time.
- Per capita water consumption peak flow factors, minimum and maximum pipe sizes, pipe material and reliability considerations.
- The design period should be based on useful cost of components and project cost.
- Future population growth or increase in water demand should be considered.
- Project costs should be considered within the donor’s budget.
- Cost of energy, the annual cost for maintenance and flow discharge of pump should be determined.
- A pipeline should be designed to a minimum of 60 years.
- A pump should be design to a minimum of 12 years.
Unit of Measurement:

MKS (meter-kilograms-Second) system is the internationally agreed version of metric system (SI) of units. All physical quantities can be described by a set of three primary units, Mass (kg), Length (m) and Time (s) designated by M, L and T respectively.

The unit of force is called newton (N) and 1 N is the force which accelerates a mass of 1 kg at a rate of 1 m/ s².

The unit of work is called the joule (J) and it is the energy needed to move a force of 1 N over a distance of 1 m, i.e 1 Nm.

Power is the energy or work done per unit time and its unit is the watt (W). 1 W = 1 Nm/s = 1 J/s

Table of Units:

<table>
<thead>
<tr>
<th>Unit</th>
<th>mass</th>
<th>Length</th>
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<th>Force</th>
<th>Acceleration</th>
<th>work</th>
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<td>m</td>
<td>s</td>
<td>N</td>
<td>a 1 m/ s²</td>
<td>Joule (J)</td>
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</tbody>
</table>

<table>
<thead>
<tr>
<th>Unit</th>
<th>mass Density</th>
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<th>Temperature</th>
<th>Pressure</th>
<th>Discharge</th>
<th>Velocity</th>
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</thead>
<tbody>
<tr>
<td>Measurement unit</td>
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<td>γ(N/m³)</td>
<td>°C</td>
<td>N/m²</td>
<td>Liter /second</td>
<td>m/s</td>
</tr>
</tbody>
</table>

Conversion of English units to Metric:

**Unit of Distance and Length**

<table>
<thead>
<tr>
<th>English Unit</th>
<th>multiplied by</th>
<th>= Metric Unit</th>
</tr>
</thead>
<tbody>
<tr>
<td>inches (in)</td>
<td>25.4</td>
<td>Millimeters (mm)</td>
</tr>
<tr>
<td>inches</td>
<td>2.54</td>
<td>centimeters(cm)</td>
</tr>
<tr>
<td>inches</td>
<td>0.254</td>
<td>meters (m)</td>
</tr>
<tr>
<td>feet (ft)</td>
<td>30.48</td>
<td>centimeters(cm)</td>
</tr>
<tr>
<td>yards (yd)</td>
<td>91.44</td>
<td>centimeters(cm)</td>
</tr>
<tr>
<td>yards</td>
<td>0.9144</td>
<td>meters (m)</td>
</tr>
<tr>
<td>miles (mi)</td>
<td>1.609</td>
<td>kilometers (Km)</td>
</tr>
</tbody>
</table>
Unit of Mass:

<table>
<thead>
<tr>
<th>English Unit</th>
<th>Multiplied by</th>
<th>= Metric Unit</th>
</tr>
</thead>
<tbody>
<tr>
<td>ounce (oz)</td>
<td>28</td>
<td>grams (g)</td>
</tr>
<tr>
<td>pound (lb)</td>
<td>0.4536</td>
<td>kilograms (kg)</td>
</tr>
<tr>
<td>tons (t)</td>
<td>0.9072</td>
<td>metric tons (mt)</td>
</tr>
</tbody>
</table>

3.2 Introduction to Water Supply

As a water supplier, engineers are responsible for making the water supply safe to drink for water consumers. In Afghanistan, the most common method of water supply in urban and rural areas are wells, springs, and fresh river water.

Wells and Spring Water Supply

Wells: Groundwater occurs in a variety of geological strata, including unconsolidated materials, porous rocks, fractured rocks, and limestone with solution channels. Any of these formations can be developed into an acceptable source, as long as:

- The water can be brought up to acceptable quality standards by treatment.
- The permeability of the formation permits adequate flows into the wells.
- The volume and recharge rate of the aquifer permits useful long-term withdrawal rates.

The following factors should be considered when designing the water supply system.

Required tests for wells and spring water

The Afghanistan Ministry of Public Health recommends mixing a small amount of Chlorine in drinking water.

The following test and procedure are required in most of international water treatment codes, even if drinking water looks, tastes, and smells fine, for the following reason test is required.

- Germs, chemicals and toxic waste in or on the ground can contaminate underground drinking water. Consumers might not see, smell, or taste them.
- Some local rocks contain harmful substances that can dissolve into purpose well water.

To provide safe drinking supply the following test should be carried out, taken from University of Rhode Island website.
### Well, Spring Testing Schedule

#### Every Year

<table>
<thead>
<tr>
<th>Test for:</th>
<th>Reason for testing</th>
</tr>
</thead>
<tbody>
<tr>
<td>Coliform bacteria (germs)</td>
<td>Key water quality test</td>
</tr>
<tr>
<td>Nitrate</td>
<td>Infant blood problems</td>
</tr>
<tr>
<td>Nitrite</td>
<td>Infant blood problems</td>
</tr>
<tr>
<td>Color</td>
<td>Key water quality test</td>
</tr>
<tr>
<td>Turbidity (cloudy water)</td>
<td>Key water quality test</td>
</tr>
<tr>
<td>Chloride</td>
<td>Salty tasting water</td>
</tr>
</tbody>
</table>

#### Every 3 - 5 Years Recommended test

<table>
<thead>
<tr>
<th>Test for:</th>
<th>Reason for testing</th>
</tr>
</thead>
<tbody>
<tr>
<td>Fluoride</td>
<td>Too much fluoride: Bone disease, stained teeth</td>
</tr>
<tr>
<td></td>
<td>Too little fluoride: tooth decay</td>
</tr>
<tr>
<td>Iron</td>
<td>Laundry or plumbing fixture stains</td>
</tr>
<tr>
<td>Lead</td>
<td>Physical and mental development delays, kidney problems, high blood pressure</td>
</tr>
<tr>
<td>Manganese</td>
<td>Laundry or plumbing fixture stains</td>
</tr>
<tr>
<td>Sulfate</td>
<td>Diarrhea</td>
</tr>
<tr>
<td>pH</td>
<td>Can ruin appliances and plumbing</td>
</tr>
</tbody>
</table>

### Tests for corrosiveness:

- **Alkalinity**
- **Total dissolved solids**
- **Hardness**
- **Specific Conductance**

Can ruin appliances and plumbing
Every 5 - 10 Years Recommended Test

<table>
<thead>
<tr>
<th>Test for:</th>
<th>Reason for testing</th>
</tr>
</thead>
<tbody>
<tr>
<td>Volatile Organic Compounds (VOCs)</td>
<td>Increased chance of cancer and other negative health effects</td>
</tr>
<tr>
<td>MTBE</td>
<td>Gasoline pollution</td>
</tr>
</tbody>
</table>

Some other additional tests may be planned, if changes are seen, for the following reasons:

- Test whenever you notice a change in your drinking water's taste, smell, or color.
- Test if your drinking water well has been flooded.

Some additional tests may consider depending on well type, plumbing type, and where the location is. See the Additional Tests requirements chart below taken from the University of Rhode Island website.

### Additional Tests

<table>
<thead>
<tr>
<th>Pollutant</th>
<th>Reason for testing</th>
</tr>
</thead>
<tbody>
<tr>
<td>Arsenic</td>
<td>If wells or spring is near agricultural land - especially fruit orchards or land that was a fruit orchard. Also, if it’s near industrial sites.</td>
</tr>
<tr>
<td>Beryllium</td>
<td>Occurs naturally in the bedrock areas.</td>
</tr>
<tr>
<td>Copper</td>
<td>If the plumbing system has copper pipes it is possible that the copper in the pipes can dissolve into the water.</td>
</tr>
<tr>
<td>Man-made Chemicals</td>
<td>In some areas of the state industrial solvents, manufacturing chemicals, ammunition wastes, and pesticides have been detected in groundwater.</td>
</tr>
<tr>
<td>Radon</td>
<td>There is no drinking water standard for radon. However, radon in the water can contribute to radon levels in indoor air, which is of greater concern.</td>
</tr>
</tbody>
</table>

**Places to test wells or spring water:**

The water sample should be tested in a certified laboratory, the following point should be considered.

The steps to testing your well or spring water:

- Find a certified lab close to the area where the well or spring is based.
- Consult the lab technician and tell them which tests are required.
The lab should supply bottles and directions to collect water samples. If you are not sure what to do, consult the lab for more help. Some labs will do the sampling, right at required area.

After the collection of water samples, they should be taken to the lab within 24 hours. The lab report result should be checked within 2 weeks with lab. The lab results should clearly point out anything over the safe limit.

Understanding The test report or result
Lab report results should point out anything that is over the safe limit. Some water quality problems are quick and easy to fix. Others take longer and cost more. The consultation should be made with lab technician or lab report analyzer to solve the problem.

Wells or Spring Water Treatments
After drinking water has been tested at a certified lab and lab report results have been received the designer should be ready to determine if there is a need for a water treatment system. Some water quality problems can be quick and easy to fix. Others take longer and cost more. Treatment systems are designed to fix specific problems and a responsible person or team needs to make sure to implement the right solution to solve the problem and the following factors should be considered for water treatmen:

Get at least 3 quotes from treatment professionals.

Determine whether the treatment product has been rated by the National Sanitation Foundation, International or other 3rd party organization.

Clearly understand what is included in the purchase price of the treatment system.

Who is responsible for maintenance?

How often does maintenance need to happen?

3.3 Construction of Wells
The following five types of wells can be constructed, in Afghanistan, but dug wells and bored wells are most common. The type of well should be determined by the design engineer based on advantages, available funds and resources.

Dug wells
Bored wells
Driven wells
Jetted wells
Drilled Well

Dug wells: The dug well has been the most common well in Afghanistan for many years; it is constructed by hand using pick and shovel to excavate below the groundwater table until incoming water exceeds the digger's bailing rate. It is more difficult to protect from contamination, although if finished properly
it may provide a satisfactory supply. Because of advantages offered by other types of wells, the following factor should be considered during design and construction.

The well should then be lined (cased) with stones, brick, tile, or other material to prevent collapse. It should be covered with a cap of wood, stone, or concrete. Being so shallow, dug wells have the highest risk of becoming contaminated. To minimize the likelihood of contamination, the dug well should have certain features. These features help to prevent contaminants from traveling along the outside of the casing or through the casing and into the well.

**Bored Wells:** In general, bored wells have the same characteristics as dug wells, but they may be extended deeper into the water-bearing formation and avoid water shortage in drought season. Bored wells are commonly constructed with earth augers turned either by hand or by power.

The following factor should be considered for construction of bored wells:

In suitable material, holes from (5cm to 75cm) in diameter can be bored to about 70m water table depending on water table without caving.

Bored wells should be cased with vitrified tile, concrete pipe, standard wrought iron, steel casing, or other suitable material capable of sustaining imposed loads.

Proper protection from surface drainage should be provided by sealing the casing with cement grout to the depth necessary to protect the well from contamination.

3.4 **Well Construction Features and Site Preparation**

The top of the well must be constructed so that no foreign matter or surface water can enter. The well should be covered by a concrete curb and cap that stands about a 30cm above the ground and check to make sure it is properly covered and sealed.

The well site should be properly drained and adequately protected against erosion, flooding, damage and contamination from animals.

Surface drainage should be diverted away from the well.

The pumping equipment for either power-driven or manual systems should be so constructed and installed as to prevent the entrance of contamination or objectionable material either into the well or into the water that is being pumped.

Design the pump head or enclosure so as to prevent pollution of the water by lubricants or other maintenance materials used during operation of the equipment. Pollution from hand contact, dust, rain, birds, flies, rodents, or animals, and similar sources should be prevented from reaching the water chamber of the pump or the source of supply.

Design the pump base or enclosure so as to facilitate the installation of a sanitary well seal within the well cover or casing.

The well should be cased with a watertight material (for example, tongue-and-groove precast concrete) and a cement grout or betonies clay sealant poured along the outside of the casing to the top of the
well.

The land surface around the well should be mounded so that surface water runs away from the well and is not allowed to pond around the outside of the wellhead.

Ideally, the pump for the well should be inside user home or in a separate pump house, rather than in a pit next to the well.

Another problem should be considered during the design relating to the shallowness of a dug well it may go dry during a drought when the ground water table drops.

3.5 River water

3.5.1 Introduction
Water is generally drawn from lakes, rivers and reservoirs through relatively simple submerged intakes or through structure that rise above the water, in Afghanistan river water is commonly used especially in rural area where the water table is in much depth or difficult to dig, well water quality and taste is poor and public water supply system is unavailable. Water that is flowing swiftly over rocks is normally much cleaner than water in stagnant pools.

3.5.2 River Intakes Locations
The following factors should be considered for location of intakes:

- Water collection should be made from to fast moving stream if possible and purify water with to suitable technique or adopt to technique from the water treatment section describe above.
- River intakes should be constructed upstream from points of sewage and industrial waste if possible.
- If possible the tower should be constructed for water storage, to provide a) stable discharge b) better water quality c) protected against floods, debris, and river traffic.
- If the discharge is from a natural reservoir or lake, shore water should be avoided and avoid pollution, and prevailing wind.

3.5.3 River intakes design
The following factors should be considered in river or lake intake design.

- In cold in freezing areas ice troubles can be reduced if velocity is kept between 8-10 cm/s.
- Low velocity will avoid transporting ice and minimize the entering of leaves and debris to a minimum and fish also can escape from the intake current.
- Ice troubles can be minimized if the intake ports are placed as deep as 8 meters below the water surface and bottom sediments are kept out of the intake.
- If constructing pipes for intakes from a river, they should be designed to carry water to shore with a self-cleansing velocity (1-1.2m/s) and should be laid in trenches that are dredged, backfilled and well protected.
- Pumps used for intakes should be analysis for suitability and serviceability of different season (drought),
times of flooding.

3.5.4 Water Transmission Line

Water can be transported from a given water resource to the main storage and distribution system by the following two methods.

1) Gravity Main
2) Supply Water Pumps

Water can be transported through pipelines by a gravity main (conduits supply) or a water pump (Electrical Transmission Lines) the pumping Gravity Main system differs in construction and function requirements.

Gravity Main

The following factors should be considered for the design of at the gravity main:

The pressure head $h_o$ (on account of water level in the collection tank) various from time to time, much reliance cannot be place on it, and for the design purpose pressure head $h_o$ should be neglected.

For design purpose ignore losses of energy at the entrance and exit points.

For the determination of head loss the following formula should be used.

$$h_L = Z_o - Z_L - H$$

to determine the pipe diameter the following formula should be used.

$$D = 0.66 \left[ e^{1.25 \frac{LQ^2}{g(Z_o-Z_L-H)}}^{4.75} + VQ^{9.4} \left( \frac{L}{g(Z_o-Z_L-H)} \right)^{5.2} \right]^{0.04}$$

To determine the Capitalized cost of the gravity main the following formula should be used.

$$F = 0.66^n L K m \left[ e^{1.25 \frac{LQ^2}{g(Z_o-Z_L+H)}}^{4.75} + VQ^{9.4} \left( \frac{L}{g(Z_o-Z_L+H)} \right)^{5.2} \right]^{0.04}$$

The definition for symbols used in above formula is as follow:

$h_L$ = Total head loss, $Z_o$ = Nodal elevation at input point, $Z_L$ = Nodal elevation at supply point, $H$= minimum Prescribed terminal head, $D$= pipe link diameter, Gravitational acceleration, $L$= Pipe Link Length, $Q$= Discharge of flow, $V$= velocity of flow, $F$= cost function, $K_m$ = Pipe cost coefficient.

The constraints for which the design must be checked are the minimum and maximum pressure head constraints, the pressure head $h_x$ at a distance $x$ from the source.

$$h_x = Z_o + h_o - Z_x - 1.07 \frac{XQ^2}{gD^3} \ln \left( \frac{e}{3.7D} \right) + 4.618 \left( \frac{V}{Q} \right)^{0.9}$$

Where $Z_x$= elevation of the pipeline at distance $x$. The minimum pressure head can be negative (i.e., the pressure can be allowed to fall below the atmospheric pressure). The minimum allowable pressure head should be -2.5m. This pressure head ensures that the dissolved air in water does not come out resulting in the stoppage of flow. In case the minimum pressure head constraint is violated, the alignment of the
gravity main should be changed to avoid high ridges, or the main should pass far below the ground level at the high ridges.

If the maximum pressure head constraint is violated, one should use pipes of higher strength or provide break pressure tanks at intermediate locations and design the connected gravity mains separately. A break pressure tank is a tank of small plan area (small footprint) provided at an intermediate location in gravity main. The surplus elevation head is nullified by providing a fall within the tank. Thus, a break pressure tank divides a gravity main into two parts to be designed separately.

The design must be checked for the maximum velocity constraint. If the maximum velocity constraint is violated marginally, the pipe diameter may be increased to satisfy the constraint. In case the constraint is violated seriously, break pressure tanks may be provided at the intermediate locations, and the connecting gravity mains should be designed separately.

3.5.5 Water Pump System

Introduction

Gravity systems tend to be seen as requiring little maintenance, certainly when compared with systems involving a significant amount of mechanical equipment or the need to maintain fixed pressures. And while neglect is undesirable, so is unnecessary maintenance, and so gravity systems prevail. This can be seen as the result of an implicit decision to accept high capital costs if they result in low operating costs. But in some cases gravity is not enough, usually when it is not cost effective to provide treatment facilities for each natural sub-catchment. In these circumstances, it is appropriate to pump, and the overall methods and technology used are considered. Some engineers have favored non-gravity systems for more general application.

General arrangement of a pumping system

The following points should be considered for pumping systems:

The pump should be only considered where gravity flow is not possible.

Pumped sections require comparatively high levels of maintenance. Engineers, therefore, prefer to keep the pumped lengths to a minimum to lift the flow as required and then the system can revert to gravity flow as soon as possible.

If the liquid being pumped contains solids (for sewer purpose) and therefore pumps must be designed with the risk of clogging in mind. The nature of the liquid also creates risks of septicity, corrosion of equipment and production of explosive gases.

Analysis the type of pumps to deliver flow at a fairly constant rate for example single, parallel double or combine.

When considering the pump the flow rate handled by the pumping system, it must generally exceed the rate of flow arriving from the gravity system; otherwise there would be a risk of flooding.

The pumping systems should work on a stop–start basis, with flow arriving at a reception storage (a ‘wet well’). When the pumps are operating, the wet well empties; and when the pumps are not operating the
wet well fills. The water level in the sump should trigger the stop and start of the pumps.

**Pump Types:** Three types of pumps are commonly used in individual water transmission link systems. They are the positive displacement, the centrifugal, and the jet. These pumps can be used in a water system utilizing either a ground or surface source. They are desirable in areas where electricity or other power (gasoline, diesel oil, or windmill) make use of a power-operated pump possible. When a power supply is not available, a hand pump or some other manual method of supplying water must be used. Special types of pumps with limited application for individual water-supply systems include air-lift pumps and hydraulic rams.

**Positive-Displacement Pumps.** The positive-displacement pump forces or displaces the water through a pumping mechanism. These pumps are of several types. One type of positive-displacement pump is the reciprocating pump.

**Centrifugal Pumps.** Centrifugal pumps are pumps containing a rotating impeller mounted on a shaft turned by the power source. The rotating impeller increases the velocity of the water and discharges it into a surrounding casing shaped to slow down the flow of water and convert the velocity to pressure. This decrease of the flow further increases the pressure. Each impeller and matching casing is called a stage. The number of stages necessary for a particular installation will be determined by the pressure needed for the operation of the water system, and the height the water must be raised from the surface of the water source. When the pressure is more than can be practicably or economically furnished by a single stage, additional stages are used. A pump with more than one stage is called a multistage pump. In a multistage pump, water passes through each stage in succession, with an increase in pressure at each stage. Multistage pumps commonly used in individual water systems are of the turbine and submersible types.

**Jet (Ejector) Pumps.** Jet pumps are actually combined centrifugal and ejector pumps. A portion of the discharged water from the centrifugal pump is diverted through a nozzle and venturitube. A pressure zone lower than that of the surrounding area exists in the venturi tube; therefore, water from the source (well, spring or river) flows into this area of reduced pressure. The velocity of the water from the nozzle pushes it through the pipe toward the surface where the centrifugal pump can lift it by suction. The centrifugal pump then forces it into the distribution system.

**Pump Selection.** The type of pump selected for a particular installation should be determined on the basis of the following fundamental considerations.

- **a) Yield of the well or water source**
- **b) Daily needs and instantaneous demand of the users**
- **c) The “usable water” in the pressure or storage tank**
- **d) Size and alignment of the well casing**
- **e) Total operating head pressure of the pump at normal delivery rates, including lift and all friction losses**
- **f) Difference in elevation between ground level and water level in the well during pumping**
- **g) Availability of power**
- **h) Ease of maintenance and availability of replacement parts**
- **i) First cost and economy of operation**
j) Reliability of pumping equipment

3.5.6 Individual and Small Systems
As a result of the natural purifying capacity and protection of the soil, individual and small water supplies generally first try to develop springs and wells. Where groundwater is highly mineralized or unavailable, rainwater is next best in general safety and quality.

**Pump Capacity.** Counting the number of fixtures in the home permits a ready determination of required pump capacity using the Figure below. For example, a home with kitchen sink, water closet, bathtub, wash basin, automatic clothes washer, laundry tub, and two outside hose bibs has a total of eight fixtures. Referring to the figure, it is seen that eight fixtures correspond to a recommended pump capacity between 9 and 11 gpm (0.03 and 0.04 m³/min). The lower value should be the minimum. The higher value might be preferred if additional fire protection is desired, or if garden irrigation is contemplated.

**Pipe Material Selection:**

**Cast Iron:** For reasons of economy and ease of construction, distribution lines for small water systems are ordinarily made up with standard threaded galvanized iron or steel pipe and fittings. Other types of pipes used are cast iron, asbestos-cement, concrete, plastic, and copper. Under certain conditions and in certain areas, it may be necessary to use protective coatings, galvanizing, or have the pipes dipped or wrapped. When corrosive water or soil is encountered, copper, brass, wrought iron, plastic, or cast iron pipe, although usually more expensive initially, will have a longer and more useful life. Cast iron is not usually available in sizes below 2 in (1 cm) in diameter; hence, its use is restricted to the larger
transmission lines.

**Plastic pipe:** Plastic pipe for cold-water piping is usually simple to install, has a low initial cost, and has good hydraulic properties. When used in a domestic water system, plastic pipe should be certified by an acceptable testing laboratory to determine the pipe properties. It should be protected against crushing and from attack by rodents.

Fittings are usually available in the same sizes and materials as piping, but valves are generally cast in bronze or other alloys. In certain soil, the use of dissimilar metals in fittings and pipe may create electrolytic corrosion problems. The use of nonconductive plastic inserts between pipe and fittings or the installation of sacrificial anodes is helpful in minimizing such corrosion.

Pipe placement: Pipes should be laid as straight as possible in trenches, with air-relief valves or hydrants located at the high points on the line. Failure to provide for the release of accumulated air in a pipeline on hilly ground may greatly reduce the capacity of the line. It is necessary that pipeline trenches be excavated deep enough to prevent freezing in the winter. Pipes placed in trenches at a depth of more than 3 ft (1 m) will help to keep the water in the pipeline cool during the summer months.

**Pipe Flow.** The pipeline selected should be adequate to deliver the required peak flow of water without excessive loss of head, i.e., without decreasing the discharge pressure below a desirable minimum. The normal operating water pressure for household or domestic use ranges from (1.41 to 4.22 kgf/cm2), or about (13.72 to 42.67 m) of head at the fixture. The capacity of a pipeline is determined by its size, length, and interior surface condition. Assuming that the length of the pipe is fixed and its interior condition established by the type of material, the usual problem in the design of a pipeline is that of determining the required diameter.

The correct pipe size can be selected with the Willingford chart, which gives size as a function of head loss $H$, length of pipeline $L$, and peak discharge $Q$, Velocity $V$.

Globe values which do produce large head losses should be avoided in main transmission lines for small water systems. Interior piping, fittings, and accessories should conform to the minimum requirements for plumbing of the International Plumbing Code or equivalent applicable plumbing code of the locality.

### 3.6 Storage of Water

**3.6.1 Introduction**

An adequate supply of cold water should be provided to every residential building the water storage should be installed within the building. The insulation should be design to prevent waste, undue consumption, misuse, contamination of general supply, be protected against corrosion and frost damage and be accessible for maintenance purposes.

The following criteria must be considered for planning and designing water supply storage of buildings.

For design purposes the average usage of water in the USA is 260 liter per Capita but in Afghanistan, urban and rural areas it is considered to be in the range of 120-170 liter per capita.
design engineer should consider the situation and apply engineering judgment use the values within
the range of 120-170 liter per capita.

To overcome friction within the conveying pipes water stored prior to distribution will be required to
be under pressure and this normally is achieved by storing the water at a level above the level of
outlets and by considering the head pressure for outlet flow.

The seasonal temperature should be considered in design, especially for supplying cold and hot
water because the water becomes less dense as its temperature is raised therefore the warm water
will always displace colder water.

The storage has to be planned and designed adequately to store water for enough time to avoid
water shortages and stoppages in building, i.e. a temporary loss of power or supply water is unable
to reach the main water supply system or well.

The designer should consider an economical design, but consider high quality materials for durable,
excellent life time functions for water supply, water storage and wastewater purposes, and the
design should comply with International Codes and Standards.

If the structure design includes a garden then the garden water requirement should be considered.

If the water supply is from main water supply system then a meter should be installed for the
monthly usage of water, If , however, the water supply to storage is by water pump then the
monthly cost of electricity or generator should be defined in the design stages of water supply.

If the water is supplied from a main storage to different groups then water meter should be installed
for every group.

<table>
<thead>
<tr>
<th>Organization for MRRD Project</th>
<th>Water usage in an hour in liter per capita</th>
</tr>
</thead>
<tbody>
<tr>
<td>Health care Center</td>
<td>15</td>
</tr>
<tr>
<td>Community Center</td>
<td>6-10</td>
</tr>
<tr>
<td>School</td>
<td>15-30</td>
</tr>
</tbody>
</table>

A typical design is shown in the following figure, adopted from R. Chudlel, Building Construction
Handbook page 761.
3.6.2 Water Tank Design

Introduction

Water storage tanks are a commonly used facility in virtually all water distribution systems. There are various methods for sizing and locating these facilities to provide equalization and emergency storage. Because water-quality problems are usually worse in tanks with little turnover, no longer is it safe to assume that a bigger tank is a better tank. There is increased emphasis on constructing the right-sized tank in the right location. Water-quality concerns will not radically change tank design, but they do call for a reassessment of some design practices.

Water Tanks Systems design Criteria.

Water tanks should be constructed a water distribution system is proposed or as an upgrade to an existing water distribution system.

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 sufficient quantity to meet all points of demand in the distribution system.

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.

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.

For concrete type water storage, it should be designed with impermeable concrete.

Design mix of fine and coarse aggregates in cement to be produce high strength concrete.

Efficient compaction, preferably using vibration for concrete structures.

Considering effect of water cement ratio for compaction for concrete structures

Joints should be water-tight to prevent leakage.

Design of concrete should prevent cracks and tensile stress of concrete should be within permissible limits.

**Types of Storage:**

The following three types of water storage is commonly constructed in Afghanistan. Examples of standard water tank construction at USACE-AED (US Army corps of Engineering Afghanistan) projects are shown in Figures below.

**a) Ground Storage.** Ground storage is usually located away 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. A Typical example of a ground level reinforced concrete tank at USACE-AED (US ARM Corps Engineering) projects in Afghanistan is shown. 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 below. 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. 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.
3.6.3 Determining Tank Capacity

The capacity of the storage tank is important to the efficient operation of a water supply system. It should be large enough to store sufficient water to meet both average and peak daily demands. When designing a storage tank, keep in mind that demand for water varies during the year. In the hotter months, people use more water than in cooler months and on certain religious or cultural occasions water use may increase. The first step in determining storage capacity is calculating the demand for water in the community. Follow the steps below in estimating demand.

Determine the population of the community. Use census data or initiate a survey to obtain population figures. Check past records to determine the rate of population growth over the years. If funds permit, the storage tank should be designed to last for twenty years. Therefore, use the estimated population for twenty years in the future to determine demand for water. Use the growth factors from population Growth factors table, when estimating future population. For example, the present population of a community is approximately 1300 and it has been growing at a rate of 3 percent per year. To determine the population in twenty years, multiply 1300 by the population growth factor 1.81 found in the row marked 20 and the 3 percent column in population Growth factors table.

Population = 1300 x 1.81 Population = 2350 or approximately 2400.

The reservoir should be designed for a Population of 2400 people.

Once the population is known, the demand for water can be calculated. Demand can be estimated by considering the type of distribution system used. Estimated Water Consumption Table, shows estimated water consumption rates for different types of distribution arrangements. Another important factor affecting demand is the use of water for purposes other than household drinking
and cleaning. If the community has hotels and restaurants or if animals will be watered from the public system, consumption figures would reflect these uses. Water use requirement Table, shows estimated water use for various institutions and for animals. Use these figures when designing the capacity of the reservoir. The total daily demand for water can be calculated using Worksheet A. The calculations are done for a population of 2400 people in a town that has a small hospital with twenty beds, one hotel for 75 people and two schools. A large chicken farm with 5000 chickens also uses water from the public system. It is estimated that 40 percent of the population will be served by multiple taps, 35 percent by single taps in the yard and 20 percent by standpipe. Five percent will have no service.

Once the total daily demand is determined, peak demand should be considered.

Peak demand is the highest rate of demand during the day. Usually peak demand occurs during the morning when people get up to begin the day and in the early evening after work is completed. Peak demand is estimated by adding 20-40 percent to average daily demand. Multiply average daily demand by 1.2 or 1.4. For example, Average day = 120000 liters/day Peak day = 1.2 x 120000 = 144000 liters/day. A general rule to follow is that the capacity of the storage tank should be 20-40 percent of the peak day water demand. With a peak daily demand of 144000 liters, the capacity of the tank should be at least 30m³ or 144000 liters x .2 = 30000 liters. At the 40 percent value, the tank would be 58m³ or 144000 liters x .4 = 58000 liters. In this case, a reservoir of between 40-50m³ would be needed to meet peak demand.

The above factors can be calculated from the following tables taken from USAID Technical Report RWS S.D.3

<table>
<thead>
<tr>
<th>Type of Water Supply</th>
<th>Average Water Consumption (liters/Capita/Day)</th>
<th>Range (liters/Capita/Day)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Community water point (i.e., well, spring, public standpipe)</td>
<td>7</td>
<td>5-10</td>
</tr>
<tr>
<td>At distance 1000m</td>
<td>10</td>
<td>8-12</td>
</tr>
<tr>
<td>At distance 500-1000m</td>
<td>12</td>
<td>10-15</td>
</tr>
<tr>
<td>Village well (250m)</td>
<td>15</td>
<td>10-20</td>
</tr>
<tr>
<td>Standpipe (250m)</td>
<td>20</td>
<td>20-60</td>
</tr>
<tr>
<td>Yard connection or single tap</td>
<td>40</td>
<td></td>
</tr>
<tr>
<td>House connection (multiple taps)</td>
<td>70</td>
<td>50-120 (or more)</td>
</tr>
</tbody>
</table>
### 3.6.4 Site Conditions

The choice of site for a reservoir or tank is usually dictated by requirements outside the structural designer’s responsibility but if there is possibility the following factor should be considered.

Soil conditions may radically affect the design. A well-drained site with underlying soils having a uniform safe bearing pressure at foundation level is ideal. These conditions may be achieved for a service reservoir near to the top of a hill, because many sites where sewage tanks are being constructed, the soil has a poor bearing capacity and the ground water table is near to the surface.

Poor bearing capacity will lead to settlement and result in structure failing.

A soil survey is very important unless and accurate record of the soil is available. Boreholes of at least 150mm diameter should be drilled to a depth of 10m and the soil sample taken and tested to determine the sequence of strata and the allowable bearing pressure at various depths. The information from boreholes should be supplement by digging trial pits with a small excavator to a depth of 3-4 meters.
The soil investigation must also include chemical tests on the soils and ground water to detect the presence of sulphate or other chemicals in the ground which could attack the concrete and eventually cause corrosion of the reinforcement.

Careful analysis of the subsoil is particularly important when the site has previously been used for industrial purpose, or where ground water from an adjacent tip may flow through the site.

Foundation design for Storage Tank:

It is desirable that liquid-retaining structure is funded on good uniform soil, so that different settlement can be avoided. However, this desirable situation is not always obtainable. The following factors should be considered to design adequate foundation for storage tank.

Variations in soil conditions must be considered, and the degree of differential settlement estimated.

Joints may be used to allow a limited degree of articulation but on sites with particularly non-uniform soil, it may be necessary to consider dividing the structure into completely separate sections. Alternatively, cut-and-fill techniques may be used to provide a uniform platform of material on which to fund the structure.

Soils which contain bands of peat or other very soft strata may not allow normal support with very large settlements, and then pile foundation should be designed.

The design of structures in areas of mining activity requires the provision of extra joints, or the division of the whole structure into smaller units. Pre-stressed tendon may be added to a normal reinforced concrete design to provide increased resistance to cracking when movement takes place.

Loading Analysis:

Liquid-retaining structures are subject to loading by pressure from the retained liquid. Typical values of weights are listed in table below.

<table>
<thead>
<tr>
<th>Density of retained liquids</th>
<th>Weight (kN/m³)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Water</td>
<td>10.0</td>
</tr>
<tr>
<td>Raw sewage</td>
<td>11.0</td>
</tr>
<tr>
<td>Digested sludge aerobic</td>
<td>10.4</td>
</tr>
<tr>
<td>Digested sludge anaerobic</td>
<td>11.3</td>
</tr>
<tr>
<td>Sludge from vacuum filters</td>
<td>12.0</td>
</tr>
</tbody>
</table>

The designer must consider the following factors for the loading analysis.

A reservoir section may be empty when other sections are full and design each structural element for the maximum bending moments and forces that can occur.

Several loading cases may need to be considered.
Structural design of Reinforcement concrete water storage:

Design method is described to ensure compliance with the basic requirements of strength and serviceability of reinforced concrete water storage.

In contrast with normal structural design, where strength is the basic consideration, for liquid-retaining structures it is found that serviceability consideration control the design.

The design procedure should include the following important factors.

Estimate concrete member sizes.

Calculate the reinforcement to limit the design crack widths to the required value.

Check strength

Check other limit states

Repeat as necessary

**Wall Thickness:**

All liquid-retaining structures include wall elements to contain the liquid, and it is necessary to commence the design by estimating the overall wall thickness in relation to the height.

The overall thickness of wall should be no greater than necessary, as extra thickness will cause higher thermal stresses when the concrete is hardening.

The principal factors which govern the wall thickness are:

Ease construction

Structural arrangement

Avoidance of excessive deflections

Adequate strength

Avoidance of excessive crack widths.

**Protection against corrosion.** Protection measure shall be provided to the underside of the roof to prevent it from corrosion due to condensation.

**Minimum Reinforcement:**

The minimum reinforcement in walls, floors and roofs in each of two directions at right angles shall have an area of 0.3 per cent of the concrete section in that direction for sections up to 100mm, thickness. For sections of thickness greater than 100mm, and less than 450mm the minimum reinforcement in each of the two directions shall be linearly reduced from 0.3 percent for 100mm thick section to 0.2 percent for 450mm, thick sections. For sections of thickness greater than 450mm, minimum reinforcement in each of the two directions shall be kept at 0.2 per cent. In
concrete sections of thickness 225mm or greater, two layers of Reinforcement steel shall be placed one near each face of the section to make up the minimum reinforcement.

In special circumstances floor slabs may be constructed with percentage of reinforcement less than specified above. In no case the percentage of reinforcement in any member be less than 0.15% of gross sectional area of the member.

Minimum Cover to Reinforcement:

For liquid faces of parts of members either in contact with the liquid (such as inner faces or roof slab) the minimum cover to all reinforcement should be 25mm or the diameter of the main bar whichever is greater. In the presence of the sea water and soils and water of corrosive characters the cover should be increased by 12mm but this additional cover shall not be taken into account for design calculations.

For faces away from liquid and for parts of the structure neither in contact with the liquid on any face, nor enclosing the space above the liquid, the cover shall be as for ordinary concrete member.

3.6.5 Design of steel Weld storage tank:

General Design Consideration:

it is a weld steel water tank

The entire bottom of the tank sets directly on a reinforced concrete foundation slab with a thickened edge or ring wall.

The structure steel analysis and design should comply with specifications and requirements contained in the AISC “Manual of Steel Construction 9th Edition” for allowable stress design.

Bolted connection shall be a friction-type connection.

The required safety factor should be 1.5.

Parameters:

The following parameters should be determined during the design stage.

a) Water tank Capacity (gal), liter or m³
b) Nominal diameter of tank, (D)
c) Nominal Height of water Tank, Ht (m) or cm
d) Water storage height in tank, H (m) or cm
e) Bottom plate diameter of water tank, Db (m) or cm
f) Yield strength of structural steel, plate and pipe, Fy (KN/m³)
g) Portland cement for concrete construction
h) Specified compressive strength of concrete Fc (KN/m³)
i) Specified yield strength of reinforcement, Fy (KN/m³)
j) Specified yield Strength of welded wire fabric, fy (KN/m³)
k) Density of reinforced concrete, Dc (Kg)
l) Density of steel plate, Dst (kg)
m) Density of water, Dw (kg)

n) Specify gravity of water, G

o) Snow loads or deck live loads on the roof, Wsn (KN/m²)

p) Seismic zone factor,

q) Wind Load

**TANK REQUIREMENTS**

**WATER TANK**

**TANK SIZE:**
- Square feet: 1,000, 5,000, 3,001-4,500, over 4,500
- Gallons: 5,000, 7,500, over 10,000

*Recommended minimum size of 5,000 Gallons.*

**Notes:**
- Water is used for domestic use, add 1,500 Gal. to the above figures, see diagram for plumbing requirement.

**典型水箱设计**

3.7 Water Distributions and Pipe Networking of Buildings

3.7.1 Introduction

Water distribution networking analysis can be used for the design of new systems or improving and extension of existing system. The design of the pipe network in a distribution system is an iterative process based on the desired pressure in the system under different demand conditions. Trial pipe diameters are selected for the network of pipes, and a hydraulic analysis is performed for the range of conditions. Of the numerous issues that must be addressed in the network design, the following will be presented in this section.

a) Pipe material selection.
b) Pipe diameter and spacing.
c) Design equations.
d) Simple network evaluation.
e) Valve selection and spacing.
f) Hydrant spacing.
g) Minor loss calculation
h) Pipe Material Selection

Standards and specifications for pipes can be used from the American National Standards Institute (ANSI) and the American Water Works Association. These should be obtained for actual design specifications.

Common pipe materials for water distribution systems are ductile iron pipe (DIP), polyvinyl chloride (PVC) pipe, high density polyethylene (HDPE) pipe, reinforced concrete pressure pipe (RCPP), steel pipe, and asbestos-cement pipe (ACP). ACP has not recommended because of health concerns related to asbestos even though research has shown no association between water delivered by ACP and disease (Ysusi, 2000). Steel pipe is rarely used for pipelines smaller than 400 mm. It can be used for transmission pipelines in sizes larger than 600 mm. RCPP is not commonly used for water distribution and serves as an alternative material for transmission lines. It has the disadvantage that it is attacked by soft water, acids, sulfides, sulfates, and chlorides. It may be cracked by water hammer. The more common pipe materials are DIP, PVC, and HDPE.

**Ductile Iron Pipe (DIP).** This is the most common water distribution pipe used for water mains 400 mm in diameter or smaller. The standard length is 5.5 m. Sizes range from 100 to 1,350 mm. Current practice is to use cement mortar lining and an asphaltic outer coating. DIP manufacturers recommend that the pipe be encased in a loose-fitting flexible polyethylene tube (0.2 mm thick) in corrosive soils. These are commonly known as “baggies. Rubber push-on and mechanical joints are used to connect the pipes. These joints allow for about 2 to 5 degrees of deflection. Flanged joints are used for fitting and valve connections in locations where the pipe is not buried. Service connections, known as corporation stops, may be installed either before or after pipe installation. DIP is favored because service connections can be made while the pipe is in service without shutting off the water supply to other customers. AWWA Manual M41 (AWWA, 2003) provides detailed information on design criteria for earth loads, truck loads, railroad crossings, fittings, thrust restraint, and corrosion protection, as well as procedures for installation.

**Polyvinyl Chloride (PVC) Pipe.** This is the most common plastic pipe used. Although it is manufactured in sizes up to 900 mm, the commonly used sizes for water distribution systems are 300 mm and smaller. It is rated for pressure capacity at 23º C. As the operating temperature rises above 23ºC, the pressure rating decreases. There are two AWWA specifications for PVC pipes depending on the size. For the 100 to 300 mm sizes, pressure ratings are in three classes. These ratings include an allowance for hydraulic transients (pressure surges or waves). The larger sizes are not rated in the same fashion, and they do not provide an allowance for pressure surges. Rubber gasket bell and spigot type joints are used to connect the pipes. Ductile iron fittings are used. PVC is corrosion resistant, and no coating or lining is provided.

**High-Density Polyethylene (HDPE).** Although it is manufactured in sizes from 100 to 1,200 mm, this pipe has primarily served as a transmission line. Like PVC, it is rated for pressure capacity at 23º C. It is rated for pressure transients not exceeding two times the nominal pressure class. Thermal butt-fusion is the most widely used method for joining HDPE pipe. This procedure uses portable equipment to hold the pipe or fittings in close alignment, while opposing ends are faced, cleaned, heated, melted, fused, and cooled. The pipe is normally joined above ground and then placed in the trench. This method of joining requires a much higher skill level than push joints. This joint does not
allow for deflection. HDPE is not to be joined by solvent cements, adhesives, or threaded connections. Ductile iron fittings are used. HDPE is corrosion resistant, and no coating or lining is provided.

**Pipe Diameter and Spacing:**

Based on GLUMRB (2003) specifies that the minimum size of water main that provides fire protection and serves fire hydrants shall be 150 mm diameter. Larger size mains will be required to allow withdrawal of the fire flow while maintaining minimum pressures. The pipe diameter where fire protection is not to be provided should be a minimum of 75 mm diameter.

From the table below. Within these guidelines and commensurate with the demand estimates, a trial set of pipe diameters must be selected. Pipes are normally placed in the right-of-way (ROW) alongside a public road, so the road net sets the spacing. The pipes should not be placed under the pavement, except in crossings and unusual circumstances, because this makes repairs more difficult and expensive. House connections on the side of the street opposite the pipeline are made by boring under the roadway. The depth of the pipe in the ground is a function of the climate (it should be placed below the frost line), soil load, and wheel loading from vehicles.

### Typical distribution piping criteria

<table>
<thead>
<tr>
<th>Appurtenance</th>
<th>Typical minimum values</th>
</tr>
</thead>
<tbody>
<tr>
<td>Smallest pipes in network</td>
<td>150 mm</td>
</tr>
<tr>
<td>Smallest branching pipes (dead ends)</td>
<td>200 mm</td>
</tr>
<tr>
<td>Largest spacing of 150 mm grid</td>
<td>180 m</td>
</tr>
<tr>
<td>Smallest pipes in high-value district</td>
<td>200 mm</td>
</tr>
<tr>
<td>Smallest pipes on principal streets in central district</td>
<td>300 mm</td>
</tr>
<tr>
<td>Largest spacing of supply mains or feeders</td>
<td>900 m</td>
</tr>
</tbody>
</table>

\(^d\) 200 mm pipe is used for larger spacing.
Adapted from AWWA, 1998.

**Design Equations**

The objective of the hydraulic analysis of the trial set of pipe diameters is to ensure that the desired pressure and flow rate is achieved at specific locations in the system. The hydraulic analysis is based on an extended version of the Bernoulli equation. It may be expressed as:

\[
\frac{p_1}{\gamma} + \frac{(v_1)^2}{2g} + z_1 = \frac{p_2}{\gamma} + \frac{(v_2)^2}{2g} + z_2 + h_f
\]

Where

- \(p_1, p_2\) = pressure at points 1 and 2, m of water
- \(v_1, v_2\) = velocities at points 1 and 2, m³/s
- \(z_1, z_2\) = elevations of points 1 and 2, m
- \(h_f\) = headloss due to friction, m
- \(\gamma\) = specific weight of fluid, kN/m³
The headloss can be calculated using the Hazen-Williams equation (Equation 3-5) reproduced here for convenience.

\[ h_f = 10.7 \left( \frac{Q}{C} \right)^{1.85} \left( \frac{L}{D^{4.87}} \right) \]

Where:

- \( h_f \) = headloss, m
- \( Q \) = flow rate, m³/s
- \( L \) = equivalent length of pipe, m
- \( C \) = Hazen-Williams coefficient of roughness
- \( D \) = diameter of pipe, m

Velocities are calculated based on the continuity equation:

\[ Q = vA \]

Where

- \( Q \) = flow rate, m³/s
- \( v \) = velocity of flow, m/s
- \( A \) = cross-sectional area of flow, m²

Example Simple Network Evaluation:

At this point in the discussion, a simplified network evaluation is useful in illustrating the steps of the pipe network design process. A small distribution system at a camp has been selected for this illustration (Figure below).
Fire protection is not included. Water is supplied to the system by a well pumping to an elevated storage tank. Water from the storage tank supplies the pressure to the distribution system. The peak hour demand at various points in the distribution system and the corresponding elevations are given in Table below.

### Peak hour demand and elevations for Example 17-4

<table>
<thead>
<tr>
<th>Point</th>
<th>Peak hour demand, m³/h</th>
<th>Elevation of pipe, m</th>
</tr>
</thead>
<tbody>
<tr>
<td>Bottom of storage tank (A)</td>
<td>50.0</td>
<td>200.0</td>
</tr>
<tr>
<td>B</td>
<td>50.0</td>
<td>177.14</td>
</tr>
<tr>
<td>C</td>
<td>10.0</td>
<td>176.53</td>
</tr>
<tr>
<td>D</td>
<td>6.00</td>
<td>176.23</td>
</tr>
<tr>
<td>E</td>
<td>3.00</td>
<td>175.62</td>
</tr>
<tr>
<td>F</td>
<td>40.0</td>
<td>174.74</td>
</tr>
<tr>
<td>H</td>
<td>25.0</td>
<td>175.04</td>
</tr>
<tr>
<td>I</td>
<td>15.0</td>
<td>175.04</td>
</tr>
</tbody>
</table>

**Solution:**

The hydraulic analysis was performed in a tabular fashion. The resulting table is shown below. The calculations are explained below the table.

<table>
<thead>
<tr>
<th>Example</th>
<th>hydraulic analysis of camp</th>
</tr>
</thead>
<tbody>
<tr>
<td>Head available, m</td>
<td>Head remaining</td>
</tr>
<tr>
<td>Distance From To Meters</td>
<td>Flow, m³/h</td>
</tr>
<tr>
<td>A C</td>
<td>51.8</td>
</tr>
<tr>
<td>C D</td>
<td>12.2</td>
</tr>
<tr>
<td>C G</td>
<td>106.7</td>
</tr>
<tr>
<td>G H</td>
<td>30.5</td>
</tr>
</tbody>
</table>

*Note: (m of water)²/(9.8067) = kPa.*

The starting point for the run of pipe from A to C is at the bottom of the water tower. The head available is that just before the water tower becomes empty. This is the lowest head that will be available. At this starting point (A) the head available is the “Initial” head of 0.00 m shown in column 4 in the row labeled “A to C.”

The “Fall” is the elevation difference between A and C. That is 200.0 m - 177.14 m = 22.86 m, as shown in column 5 in the row labeled “A to C.”

There is no “Rise” in elevation between A and C.

The “Total” head available is then 0.00 m + fall-rise = 0.00 + 22.86 m - 0.00 = 22.86 m.
The “Headloss” is the friction loss per 100 m of pipe. Using the table below.

<table>
<thead>
<tr>
<th>Capacity m³/h</th>
<th>200</th>
<th>250</th>
<th>300</th>
<th>350</th>
<th>375</th>
<th>400</th>
</tr>
</thead>
<tbody>
<tr>
<td>m³/s</td>
<td>v²/2g</td>
<td>hꜜ</td>
<td>v²/2g</td>
<td>hꜜ</td>
<td>v²/2g</td>
<td>hꜜ</td>
</tr>
<tr>
<td>10</td>
<td>0.004</td>
<td>0.08</td>
<td>0.004</td>
<td>0.07</td>
<td>0.004</td>
<td>0.06</td>
</tr>
<tr>
<td>20</td>
<td>0.002</td>
<td>0.04</td>
<td>0.002</td>
<td>0.03</td>
<td>0.002</td>
<td>0.02</td>
</tr>
<tr>
<td>30</td>
<td>0.001</td>
<td>0.02</td>
<td>0.001</td>
<td>0.01</td>
<td>0.001</td>
<td>0.01</td>
</tr>
<tr>
<td>40</td>
<td>0.000</td>
<td>0.01</td>
<td>0.000</td>
<td>0.01</td>
<td>0.000</td>
<td>0.01</td>
</tr>
<tr>
<td>50</td>
<td>0.000</td>
<td>0.00</td>
<td>0.000</td>
<td>0.00</td>
<td>0.000</td>
<td>0.00</td>
</tr>
</tbody>
</table>

Headloss (hꜜ) is in m/100 m. 
Velocity headloss (v²/2g) in m.
The loss is estimated as 5.8 m/100 m of pipe. For the 51.8 m run of pipe, the Total headloss is:

\[
\left(\frac{5.8\text{ m}}{100\text{ m}}\right)(51.8\text{ m}) = 3.00\text{ m}.
\]

The “Head remaining” is the difference between the Total head available and the Total headloss:

\[
22.86\text{ m} - 3.00\text{ m} = 19.86\text{ m}
\]

Converting the Head remaining in m to kPa:

\[
(19.86\text{ m})(9.8067\text{ kPa/m of water}) = 194.8\text{ kPa}
\]

This is below the design goal of 207 kPa.

The head remaining (19.86 m) at point C becomes the “Initial” head for the next reach of pipe (C to D and C to G).

The water flow around the junction labeled “C” must balance. The flow into the junction is 50 m³/h. The flow to D is 10 m³/h, so the flow to G must be 50 m³/h - 10 m³/h = 40 m³/h.

The head remaining at H is shown in the last column in the last row. It is 170.76 kPa.

Comment:
While it might seem appropriate to stop the calculation at the end of the first row and adjust the design selections, the computations are continued to point H. This is done to illustrate that further losses in the system up to point H must also be considered. So rather than stop and readjust at each point, the adjustment is made for the system as a whole.

One alternative adjustment would be to raise the bottom tank elevation \( [(207 \text{ kPa} - 170.76 \text{ kPa})/9.8067=3.70 \text{ m}] \). Another would be to select larger diameter pipes. These alternatives would have to be evaluated based on cost.

The Bernoulli equation is related to the tabular calculation as follows

\[
\frac{P_2}{\gamma} = \frac{P_1}{\gamma} + \frac{(v_1)^2}{2g} + z_1 + \frac{(v_2)^2}{2g} - z_2 - h_f
\]

The pressure at the bottom of the tank is zero so \( p_1 = 0 \). As noted in the assumptions, the velocity head is ignored so \( v_1 = 0 \) and \( v_2 = 0 \). There is no rise, so \( Z_2 = 0 \). \( Z_1 = 22.86 \text{ m} \).

\[h_f = 3.00 \text{ m}.\] The Bernoulli equation is then

\[
\frac{P_2}{\gamma} = 0 + 0 + 22.86 - 0 - 0 - 3.00 = 19.86 \text{ m}
\]

The calculations were performed on a spreadsheet. This allows for adjustment of one value, such as the height of the bottom of the storage tank, to see its implications on other parts of the system. The headloss was obtained from the given table, but it can easily be calculated in the spreadsheet.

This system has no loops. This is not common design practice. However, the inclusion of loops requires flow balancing. This process is can be easily carried out using a computer program.

### 3.7.2 Valve Selection

Valves are a significant component of any water distribution system. They are commonly used for isolating a section of pipeline for maintenance or repairs, controlling the flow rate, releasing air, and preventing backflow. Two elements are considered in selecting valves for a distribution system design. The first is headloss as the water passes through the valve. The second is the method of controlling the flow. The headloss characteristics are intrinsically related to the method of control. Four basic closure methods are used for flow control (AWWA, 2006):

A disc or plug moves against or into an opening. Examples are globe and piston valves.

A flat, cylindrical, or spherical surface slides across an opening. An example is a gate valve.

A disc or ellipse rotates across the diameter of the pipe. Examples are plug, ball, butterfly, and cone valves.

A flexible material moves into a flow passage. Examples are diaphragm and pinch valves.

Gate valves are the most commonly used type of valve for isolating portions of a distribution system for pipes in the range 150 to 400 mm. Resilient seat gate valves are favored because the resilient material (e.g., vulcanized rubber) seats against the prismatic body of the valve. There is no pocket at the bottom of the valve to collect grit. They are not suited for precise flow control because flow...
reduction is not proportional to travel of the closure disc. The characteristics and applications of this and other types of valves are given in Manual M44 (AWWA, 2006).

**Special Purpose Valves:** For specific system applications, special valves are available to make operation simpler, more efficient, or automatic. These include:

**Check valves.** This is a single direction valve that allows flow in one direction and stops reverse flow. They cause significant pressure loss and are recommended for use only where reverse flow operation would be catastrophic. An example is the discharge from a pump where flow reversal might damage the pump.

**Air release valves.** An air release valve is a self-actuated valve that automatically vents small pockets of air that accumulate at the high point in a water line.

**Altitude valves.** These are frequently globe-type or piston valves that are installed in storage tank inlet-outlet lines. They remain open as the tank is filled. They close during normal flow conditions. They open again when the pressure in the distribution system becomes less than the static head of the height of water in the tank.

**Pressure relief valves.** These are used to protect against excessive pressure in the water line.

**Pressure reducing valves.** These are used to provide water to a pressure district or zone of lower elevation from a district of higher elevation. They are often globe valves similar to those of altitude valves. By design, they have a very high pressure drop.

**Reduced pressure zone backflow prevention valve.** This valve is used to prevent the reversal of flow that might cause contaminated water to flow into the water line. An example application would be a multistory hospital or laboratory connection to a water main. The water main could potentially have a lower pressure than the building because of fire demand, and water from instrument washing sinks could flow into the water main.

### 3.7.3 Valve Placement
GLUMRB (2003) recommends that distribution system valves be located at not more than 150 m intervals in commercial districts and at not more than one block or 250 m in other districts. It is common to place valves so that sections may be isolated for repairs while continuing to provide service to other segments of the system.

### 3.7.4 Hydrant Spacing
GLUMRB (2003) recommends that hydrants be placed at each street intersection and at intermediate points. This placement ranges from 70 to 300 m. Fire departments normally require a maximum lineal distance between hydrants of 90 m in congested areas and 180 m in light residential districts AWWA (1998). The actual distance between hydrants is dependent on the amount of hose the local fire department normally carries.

**Minor Losses**

The head losses that occur due to bends, elbows, joints, valves, and so on are often referred to as *minor losses*. In some instances this is a misnomer because they may be greater than the losses due to pipe friction. The general equation for estimating these losses is repeated here for convenience:
Where $K = \text{energy loss coefficient.}$

**General Design factors of Pipe networking**

Specifying the minimum flow rates and pressure heads must be attained at the outflow points of the network.

The flow and pressure distributions across networking are affected by the arrangement and sizes of the pipes and the distribution of the outflows.

Change in the size of diameter in one pipe length will affect the flow and the pressure distribution everywhere in network, so the redesign of pipe network will require meeting the specifications.

Design of building or area pipe networking is a complex process; the designer should continuously till meeting the design specifications.

### 3.8 General Wastewater Collection and Treatment:

#### 3.8.1 Introduction

Like with water supply design, a fundamental prerequisite to begin the design of wastewater facilities is a determination of the design capacity. This, in turn, is a function of the wastewater flow rates. The determination of wastewater flow rates consists of five parts: (1) selection of a design period, (2) estimation of the population and commercial and industrial growth, (3) estimation of wastewater flows, (4) estimation of infiltration and inflow, and (5) estimation of the variability of the wastewater flow rates.

#### 3.8.2 Design of foul Sewer:

Base on design requirement the sewer can be design as foul sewer, storm sewer, flooding sewer and combine sewer.

**Design stage:** the sewer should be design for the minimum of 25-50 years life time, it may use longer than that period. The following criteria should be considered during the design stage.

- Useful life of structure, mechanical and electrical components according to international code standard.
- Feasibility study for planning of future extensions of the system.
- Considering future plan in area, residential, commercial or industrial development.
- Financial considerations.

The designer should consider the condition of building and area for suitability conditions.

#### 3.8.3 General Design Procedure

The following procedure should be follow for the foul sewer:

Assume the pipe roughness ($K_s$)
Preparing a preliminary layout of sewers, including tentative inflow locations.

Mark each pipe as number in drawing in standard networking method.

Define contributing area DWF (Dry weather flow) in each pipe.

Find cumulative contributing area DWF

The flow rate must be measure for main production period

Estimate peak flow ($Q_p$).

Try to setting garden and diameters of each pipe.

Check $d/D < 0.75$ and $V_{max} > V>V_{min}$

The pipe diameter and gradient can be change if necessary at the design stage to meet the design criteria.

The pipe gradient is selected to ensure a minimum self-cleansing velocity is achieved.

The self-cleansing velocity is that which avoids long-term deposition of solids, and should be reached at least once per day.

---

**Foul Sewer Design**

**Design Periods**

Select suitable design periods:
- Population growth rate
- Water consumption growth rate.

**Contributing area**

Quantify:
- Domestic population
- Unit water consumption
- Infiltration.

**Hydraulic design**

Establish hydraulic constraints:
- Pipe roughness
- Velocities
- Depths:

**Dry Weather Flows**

Select design method:

Calculate:
- Dry weather flows
- Peak flow-rates:
3.8.4 Design Period

The constraints in selecting a design period (design life). Design periods that are commonly employed in practice and commonly experienced life expectancies are shown in Table below.

<table>
<thead>
<tr>
<th>Type of facility</th>
<th>Characteristics</th>
<th>Design period, y</th>
<th>Life expectancy, y</th>
</tr>
</thead>
<tbody>
<tr>
<td>Treatment plants</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Fixed facilities</td>
<td>Difficult and expensive to</td>
<td>20–25</td>
<td>50+</td>
</tr>
<tr>
<td></td>
<td>enlarge/replace</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Equipment</td>
<td>Easy to refurbish/replace</td>
<td>10–15</td>
<td>10–20</td>
</tr>
<tr>
<td>Collection systems</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Trunk lines and intercepts &gt; 60 cm</td>
<td>Replacement is expensive and difficult</td>
<td>20–25</td>
<td>60+</td>
</tr>
<tr>
<td>Lateral and mains ≤ 30 cm</td>
<td>Easy to refurbish/replace</td>
<td>To full development&lt;sup&gt;a&lt;/sup&gt;</td>
<td>40–50</td>
</tr>
</tbody>
</table>

Components of Wastewater

Wastewater may be classified into the following components:

- **Domestic or sanitary wastewater.** Wastewater discharged from residences, commercial (e.g., banks, restaurants, retail stores), and institutional facilities (e.g., schools and hospitals).

- **Industrial wastewater.** Wastewater discharged from industries (e.g., manufacturing and Chemical processes).

- **Infiltration and inflow.** Water that enters the sewer system from groundwater infiltration and storm water that enters from roof drains, foundation drains, and submerged manholes.

- **Storm water.** Runoff from rainfall and snow melt.

**Preliminary Investigations:**

The preliminary investigations include gathering of data such as demographics, wastewater production estimates and maps. It also includes an underground survey to locate obstacles such as existing sewers, water mains, gas lines, electrical and telephone lines, and similar features. An environmental review will be conducted to identify potential soil contamination from abandoned waste disposal sites and service stations. Geologic and hydrologic investigations may also be appropriate.

**Surveying and Mapping**

In order to prepare construction drawings, the following survey work must be conducted: location of streets, right-of-ways (ROW), basements and their elevations (usually estimated for residences), location of natural features such as streams and ditches, and construction of elevation profiles. In addition, benchmarks must be established for use during construction.
For sewer system layout, the map scale used is on the order of 1:1,000 to 1:3,000. For construction drawings, the map scale is on the order of 1:480 to 1:600. When there is significant relief, contours are shown at intervals ranging from 250 mm to 3 m. Elevations of street intersections, abrupt changes in grade, building foundations, and existing structures (sewers, lift stations, etc.) that new construction must connect with are included on the map. For projects encompassing more than one or two streets, aerial photogrammetry is often used.

3.9 Gravity Sewer Collection System Design

The design of the sewer network in a collection system is an iterative process based on the required capacity of the system for the anticipated flow rates. Trial pipe diameters are selected for the network of pipes, and a hydraulic analysis is performed for the anticipated range of conditions. Of the numerous issues that must be addressed in the network design, the following will be presented in this section:

a) Estimation of wastewater flow rates.
b) Pipe material selection.
c) Design criteria.
d) Design equations.
e) Collection system layout.
f) Design of a lateral or branch.
g) Estimation of Wastewater Flow Rates

The required wastewater flow rates at the beginning of the service life and at the design life are the average daily flow rate, peak hour flow rate, and the peak infiltration allowance.

3.9.1 Pipe Material Selection

The principal sewer material for pipes with small or medium diameters is polyvinyl chloride (PVC). For larger pipe diameters, ductile iron pipe (DIP), high density polyethylene (HDPE) pipe, or reinforced concrete pipe (RCP) may be specified. Truss pipes are becoming more common for larger pipe diameters.

Vitrified Clay Pipe (VCP). This classic pipe material has demonstrated its durability in use in the United States for over a century. It has a high resistance to corrosion and abrasion. Its major disadvantage is its high mass per unit length that makes it more difficult to handle and increases installation costs. It is rarely installed today. This pipe is made of clay or shale that has been ground, wet, molded, dried, and fired in a kiln. Near the end of the burning process, sodium chloride is added to the kiln. It vaporizes to form a hard waterproof glaze by reacting with the pipe surface. The firing of the clay produces a verification of the clay that makes it very hard and dense (Steel and McGhee, 1979). The pipe is manufactured with integral bell and spigot ends fitted with polymeric rings. It is available in diameters from 75 mm through 1,050 mm and lengths up to 3 m (ASCE, 1982). Pipes are typically joined with push-on gasket joints.

Polyvinyl Chloride Pipe (PVC). This pipe is made by extrusion of polyvinyl chloride. It is available in diameters from 10 mm through 1.2 m and lengths up to 6 m (ASCE, 1982). Rubber gasket bell and spigot type joints are used to connect the pipes. This pipe has been in use for over half a century. It is almost exclusively the material of choice for pressure and vacuum sewers. Its advantages are
corrosion resistance and low mass per unit length. It is subject to attack by certain organic chemicals and excessive deflection if improperly bedded. The low mass per unit length gives it some cost advantage in installation.

**Ductile Iron Pipe (DIP).** Its primary application for sewers is for force mains. Because wastewater is often corrosive, current practice is to use a cement mortar lining and an asphaltic outer coating. Epoxy coating may be used in trunk sewers. DIP manufacturers recommend that the pipe be encased in a loose-fitting flexible polyethylene tube (0.2 mm thick) when the pipe is to be placed in corrosive soils.

**High-Density Polyethylene (HDPE).** Its primary use is as an alternative pressure pipe for force mains.

**Reinforced Concrete Pipe (RCP).** Precast RCP is manufactured by a variety of techniques including centrifugation, vibration, packing, and tamping for consolidating the concrete in forms. Adjustment of the wall thickness, concrete strength, and reinforcing allow for a wide variety of strengths. The pipe is manufactured with integral bell and spigot ends. It is available in diameters from 300 mm through 5.0 m, and lengths up to 7.5 m (ASCE, 1982). These pipes are typically joined with push-on gasket joints. The normal service for RCP is for trunk lines and interceptor sewers. Its major limitations are its high mass per unit length and its susceptibility to crown corrosion.

### 3.9.2 Location

In the construction for new residential areas, the sewer is should be placed if possible on one side of the roadway in the right-of-way (ROW). Connections to the sewer from buildings on the opposite side of the street may be made by boring under the street. In established communities (or where local codes require), it may be found in alley ways behind the residence or in the Street. Sewers should be at such a depth that they can receive the contributed flow by gravity. Where houses have basements, the invert of the sewer is placed a minimum of 3.0 to 3.5 m below grade. Where there is no basement, it is placed to provide sufficient cover to protect the pipe from live load and dead load damage. Moser (2001) provides guidance on design to prevent live and dead load damage. Where building codes are in place, they should be consulted for the appropriate depth. In the absence of other guidance, a rule of thumb is to use a sewer invert depth of 1.8 to 2.4 m below grade when basements are not present. Building codes may prohibit gravity service from the basement. If sewage is to be removed from the basement level, grinder pumps are installed.

When sewers cannot be placed at a depth sufficient to prevent freezing, for example, when bedrock is near the surface, they must be insulated (GLUMRB, 2004). Countermash (1998) discusses alternative designs for these conditions.

Maximum sewer depth is approximately 8 to 9 m. When the depth exceeds 8 to 9 m, a lift station is provided. In exceptional circumstances, the sewer may reach a practical construction limit of 10 to 12 m depth before a lift station is constructed. GLUMRB (2004) specifies that gravity sewers shall be laid at least 3 m horizontally (edge to edge) from any existing or proposed water mains. Sewers crossing water mains shall be laid to provide a minimum vertical distance of 0.45 m between the outside of the water main and the outside of the sewer. It is preferable that the water main be located above the sewer. At crossings, one full length of water pipe shall be located so both joints...
will be as far from the sewer as possible. The sewer shall be designed and constructed equal to water pipe, and shall be pressure tested at 1,035 kPa to assure water tightness.

**Pipe Size.** No public gravity sewer conveying raw wastewater shall be less than 200 mm in diameter (GLUMRB, 2004). This size has been selected to minimize clogging when extraneous material enters the sewer. Some engineers design sewer pipes to flow half full at the design capacity to provide a factor of safety. This practice is favored when designing laterals or branches that have the potential to be extended to accommodate growth. It is not justified for mains, trunk lines, or interceptors (Steel and McGhee, 1979).

### 3.9.3 Slope

All sewers shall be designed and constructed to give mean velocities, when flowing full, of not less than 0.6 m/s based on Manning’s formula using an “n” value of 0.013 (GLUMRB, 2004). Slopes are commonly calculated using the depth of the invert of the pipe. Minimum slopes to achieve 0.6 m/s are shown in Table below. Slopes greater than these may be desirable to maintain self-cleansing velocities at all rates of flow, for construction, or to control sewer gases. A mean velocity of 0.3 m/s is usually sufficient to prevent the deposition of the organic solids in wastewater.

To prevent deposition of mineral matter, a mean velocity of 0.75 m/s is required. Slopes that result in mean velocities of 0.5 m/s have been used, but these require frequent cleaning (Metcalf & Eddy, 1981). Sewers 1.2 m and larger should be designed and constructed to give mean velocities, when flowing full, of not less than 0.9 m/s based on Manning’s formula and an “n” value of 0.13. Oversizing sewers to justify flatter slopes is prohibited. The use of larger pipes at flatter slopes will reduce the velocity well below the self-cleaning velocity.

The erosive action of the material suspended in the wastewater depends on the nature of the material and the velocity at which it is carried along. The erosive action determines the maximum safe velocity of the wastewater. In general, maximum mean velocities of 2.5 to 3.0 m/s at the design depth of flow will not damage the sewer (Metcalf & Eddy, 1981). Where velocities greater than 4.6 m/s are anticipated, special provision must be made to protect against displacement by erosion and impact. Sewers on slopes greater than 20 percent must be securely anchored. The slope between manholes must be uniform.

### 3.9.4 Alignment

In general, sewers less than or equal to 600 mm in diameter must be laid with straight alignment between manholes. Curvilinear alignment of sewers larger than 600 mm, may be permitted if
compression joints are specified. Slopes must be increased with curvilinear alignment to maintain a minimum velocity above 0.6 m/s. The recommended practice is to use extra manholes and straight alignment between manholes.

### 3.9.5 Changes in Pipe Size

When a smaller pipe joins a larger one, the invert of the larger sewer should be lowered sufficiently to maintain the same energy gradient. An approximate method for securing this result is to place the 0.8 depth point of both sewers at the same location.

In no instance should a larger pipe discharge into a smaller pipe. Even though a smaller pipe at a steeper slope may be able to carry the larger flow, there is the potential for objects that will travel freely in the larger pipe to obstruct the smaller pipe.

### 3.9.6 Manhole

Manholes are placed at the junction of two or more sewers, at changes in vertical or horizontal alignment, at changes in sewer size, and at the end of each line. The spacing for straight runs is shown in Table below. Drop manholes are used when the inflow and outflow sewers differ in elevation by more than 0.6 m. They may also be used to reduce the slope when the velocities exceed erosive velocities (2.5–3.0 m/s). The manholes in small sewers are typically about 1.2 m in diameter. A minimum access diameter of 0.6 m is provided. Although the same size manhole barrel is used for both small and large manholes, the base will be larger for sewers larger than 600 mm. Although the American Society of Civil Engineer’s manual on sewer design (ASCE, 1982) suggests that it is unnecessary, current practice is to provide an arbitrary minimum drop of 30 mm across the standard manhole. Otherwise, the grade through the manhole should match the energy grade line for larger diameters or with size change.

#### Typical manhole spacing for straight runs

<table>
<thead>
<tr>
<th>Pipe diameter</th>
<th>Spacing</th>
</tr>
</thead>
<tbody>
<tr>
<td>375 mm or less</td>
<td>120 m or less</td>
</tr>
<tr>
<td>450 to 750 mm</td>
<td>150 m</td>
</tr>
<tr>
<td>or</td>
<td></td>
</tr>
<tr>
<td>450 to 750 mm</td>
<td>180 m with adequate cleaning equipment</td>
</tr>
<tr>
<td>825 to 1,200 mm</td>
<td>180 m</td>
</tr>
<tr>
<td>1,200 mm or greater</td>
<td>460 m</td>
</tr>
</tbody>
</table>

*The actual spacing is highly dependent on local conditions and client preference.*

### 3.9.7 Hints from the Field

Experience has yielded the following useful rules of thumb. These are not design criteria but rather practical considerations in applying the design criteria.

In normal practice, the ground slope is used as a first trial for selecting the slope. However, there are a number of exceptions. For example:
a) If the ground is flat, select the minimum slope to achieve a velocity of 0.6 m/s with the sewer flowing full.
b) If there is a slight upgrade for a short distance, select the minimum slope to achieve a velocity of 0.6 m/s with the sewer flowing full.
c) If the ground slope yields a velocity greater than 2.5 m/s, then select a lower slope.

A drop manhole may be used to minimize excessive velocity in a steep sewer. Alternatively, consider designing the sewer as a gravity force main or pressure pipe with energy dissipation at the downstream end of the pipe (Orsatti, 1996).

Because lift stations are expensive to build, operate, and maintain, avoid them to the maximum extent possible by considering alternative routing.

Because drop manholes are expensive to build and often become plugged or fail structurally, avoid them to the maximum extent possible by considering the alternative of extra excavation.

In the upper reaches of the collection system (e.g., in residential subdivisions), there will seldom be enough flow at average or, perhaps, even at maximum discharge rates to achieve a minimum velocity of 0.6 m/s in a 200 mm diameter sewer at minimum slope. This means that a regular maintenance program that includes cleaning the sewer will be required.

4 General Guidance to Electrical Design

4.1 Introduction

All electrical system should be design adequately and meet the Standard specification. Any specification specified by the designer should follow by the contractors; the contractor has responsibility to employ electrical Engineer to achieved design standard. The following point should be follow by the contractor.

All electrical components should be purchased according to the design specifications.

If any electrical component is unavailable in the market then the contractor should consult with the designer to find alternative.

The following international codes can be used for the design and installation of building electrical system.


Russian Electrical system guidance book
General Information about Site Investigation

All information should be collected during the site surveying stage. The information should be collected according to the following table; the following table is example of data collections.

<table>
<thead>
<tr>
<th>Date:</th>
<th>Purpose of Survey:</th>
<th>Surveying Team:</th>
</tr>
</thead>
<tbody>
<tr>
<td>No</td>
<td>Description</td>
<td>Surveyor Comments:</td>
</tr>
<tr>
<td>1</td>
<td>Cable higher than 15KV should be installed over electrical pole.</td>
<td></td>
</tr>
<tr>
<td>2</td>
<td>Investigate the availability of Transformer or generator in the field, in case of availability the user should be define and consider for the initial design plan.</td>
<td></td>
</tr>
<tr>
<td>3</td>
<td>Initial sketch of the building area, including transformer and electrical generator area should be produced.</td>
<td></td>
</tr>
<tr>
<td>4</td>
<td>If any industry is located in the site, full description and location of machineries and equipment’s should be provided.</td>
<td></td>
</tr>
<tr>
<td>5</td>
<td>If any fuse box is located in the area, its location should be defined in the survey report.</td>
<td></td>
</tr>
<tr>
<td>6</td>
<td>For considering electricity supply method from local transformer or electrical generator, the permission should be approved by the local electricity supplier.</td>
<td></td>
</tr>
<tr>
<td>7</td>
<td>If for building electricity supply a new transformer or generator required, then its location should be define and permission should be approved from authority.</td>
<td></td>
</tr>
<tr>
<td>8</td>
<td>If any information available related to the communication system or cable of communication crossing along the building area should be included.</td>
<td></td>
</tr>
</tbody>
</table>

General Information before Designing Electrical System

After site survey, the architect should be consulted, if the building requires electricity room or IT system room? If it’s required then the location should be plan in the center of building. Also consultation should be obtained for connecting external cable to the building and discussion of any other important issues. Architect should consider previous conditions of the building area and then
start design of building, after 35% design is completed, the electrical engineering design team should be consulted regarding board panel, lighting, cable, duct and etc.

After 65% Architecture design is completed, the design will be presented by architect to electrical design team, the design should include, plan view, Elevation, Furniture plans, if the design is for health care center, laboratory or workshop then the location of machinery for installation should be selected.

Study and Investigation of Architecture plan

After receiving architecture plan, It should be properly study to understand the concept of propose system.

The fuse box selection should be checked for the installation and maintenance accessibility.

For lighting purpose, the following types of light should be selected.

For a large hall brighter light
For office neon light
For wet place or bathroom water proof
For open space or pedestrian deportation light
For some building emergency Alarm light (fire, Earthquake etc.)

The walls and type of rooms also should be considered for suitability purpose.

If the building ceiling has suspended ceiling then the suspended design light should be used.

For installation of sockets and switches the closest distance should be consider to the computer disk or other equipment that requires electricity for convince.

Each socket has design to the specific purpose of use; the type of socket should be installed according to purpose of use.

For ICT (Information Communication Technology) cables and switches should be design in secure place in the building.

For the design of pipe and cable networking the external and internal cable should be specified.

In the design of emergency alarm, the fire sound and light flashing should be design for main alarm and sub alarm system.

Design procedure of electrical system

Step 1: Selecting types of light, Installation procedure, Calculation of voltage for brightness.

Step 2: Selecting location of socket and analysis of requires power for the usage purpose.

Step3: Design of circuit and cable connection for all users in the building.
Step 4: Selecting location of fuse boxes, joint boxes and connection of external cables from main electricity source.

Step 5: Analysis of main cables and sub cables cross sections and preparing table for analysis of main cables.

Step 6: Analysis of system protection such as electric Grounding wire (Ground wire) Thunder, Emergency alarm.

Step 7: Analysis and draw or sketch of ICT connection.

Step 8: Draw or sketch of all lights, sockets, switches, diagrams, protection system and connection system of ICT.

Step 9: In the final stage when specification is completed then determine the volumes of work.

Step 10: Allocate the jobs to relevant and expert person or department.

Standard Codes related to the design of electricity system

Pipe covers for cables and wires below the ground level.

All wires should have plastic pipe cover.

Wiring should be according to the specifications, a selected plastic pipe cover should have 40% empty spaces for convince covering of wires and cables.

Wiring should be place inside concrete, the selected wire and cable should be checked and approved by responsible engineer and should be purchased according to the specification table.

The minimum cross section size of plastic pipe should be 20mm.

All pipes covering system should not be less than 40 schedules.

Depth of cable below the ground level should not be less than 60cm.

A minimum horizontal distance between two cables should be 10cm.

A minimum vertical distance between two cables should be 10cm.

Wiring and Cabling:

A cable and wire should be selected according to use of electricity power in the building and should be design to electricity standard codes.

A minimum cross section of wire for light should be $2.5mm^2$.

A minimum cross section of wire for normal socket should be $4mm^2$.

In case of wire has plastic cover than fixwell double wire should be used.

American Wire gauge
Identification and specification of cables are displayed on the cover of cable, for example the display of PVC (polymerizing vinyl chloride); PE (polyethylene) high voltage of 1000v can be seen.

- N-copper cable according to standard
- NA- Aluminum cable according to standard

<table>
<thead>
<tr>
<th>COMPUTED CONVERSION, mm</th>
<th>AWG, KCMIL</th>
<th>ADVISED CROSS SECTION mm</th>
</tr>
</thead>
<tbody>
<tr>
<td>3.31</td>
<td>12</td>
<td>4</td>
</tr>
<tr>
<td>5.27</td>
<td>10</td>
<td>6</td>
</tr>
<tr>
<td>8.4</td>
<td>8</td>
<td>10</td>
</tr>
<tr>
<td>13.3</td>
<td>6</td>
<td>16</td>
</tr>
<tr>
<td>21.2</td>
<td>4</td>
<td>25</td>
</tr>
<tr>
<td>26.7</td>
<td>3</td>
<td>25</td>
</tr>
<tr>
<td>33.6</td>
<td>2</td>
<td>35</td>
</tr>
<tr>
<td>42.4</td>
<td>1</td>
<td>50</td>
</tr>
<tr>
<td>53.4</td>
<td>1/0</td>
<td>50</td>
</tr>
<tr>
<td>67.5</td>
<td>2/0</td>
<td>70</td>
</tr>
<tr>
<td>85.0</td>
<td>3/0</td>
<td>95</td>
</tr>
<tr>
<td>107.2</td>
<td>4/0</td>
<td>120</td>
</tr>
<tr>
<td>126.7</td>
<td>250</td>
<td>120</td>
</tr>
<tr>
<td>152.0</td>
<td>300</td>
<td>150</td>
</tr>
<tr>
<td>177.4</td>
<td>350</td>
<td>185</td>
</tr>
<tr>
<td>202.7</td>
<td>400</td>
<td>240</td>
</tr>
</tbody>
</table>
Y- Insulator PVC (if the first letter is Y)

2Y- Insulator (if the first two letters is 2Y)

H- Conducted sheet to limit electrical space area.

C- Protective copper cover of voltage conductivity.

CW- A thin copper bounded sheet cable 1KV up to 0/6

S- A thin wider copper sheet bounded over wire, it is use as conductive sheet to limit electricity space area.

SE- A copper protective in fixwell in bounded each wire for space limiting of electric region space and use for replacement of H.

T- A copper protective wire above the ground form as twist.

F- Protective from open or uncovered copper wire.

Y- An insulator external cables cover PVC (if the second letter is Y)

2Y- insulator external cable covers PE (if the second and third letter is 2Y)

Once the cable is identified and specify, the other abbreviation can be check to define the number and cross section of cables. The following are the definition of abbreviation.

re- A whole single circular

rm- A circular fixwell

se- A single triangle type

sm- A triangle type fixwell

From above specification numbers and cross section can be determine, the below is example.

Example: NYY1X50re 0.6/1KW

Some single copper cable with circular cross section has 50mm² areas, and external insulator voltage fuse is 16KV and voltage of wire is 1KV

Switches, Sockets and Joint boxes

For selection of color of sockets and switches architect should be consulted.

All electric system should have a grounding system.

Boiler socket should be installed close to the boiler location.

The socket of boiler should be 4mm wire direct from join box, and should have own direct automatic fuse.
In the area of joints, in the section of socket and switch connections the maximum wires limit should be 6 and the way should be joints that can be place back in plastic pipe cover and does not restrict installation of socket.

If switch has two poles then the number of wires should be 4.

For installation of joints box extreme caution should be taken to place below the plaster.

Joints wire should be properly connected.

The normal socket usage should be 1000 watt in 36m², for some rooms the usage of electricity should be consider.

The normal socket should be installed 45cm above the floor level.

The switches should be installed 120cm above the floor level.

Switches should have 30cm horizontal distance from the door.

Joints box should not be installed below 30cm from ceiling.

The vertical distance for installation of water proof socket form floor level should be 120cm.

The vertical distance for installation of waterproof socket for boiler from floor level should be 180cm.

The minimum distance for switches and sockets from basin should be 60cm.

Distance between two switches should be greater than 1cm.

Bulbs (Lights or Lamps)

For installation of bulbs, the extreme effort should be made to distribute brightness of light equally in the room, and installed the bulbs at the center of rooms.

For joints of wires, the positive wire should have on/off switch button, and also a grounding system should be consider in bulbs, switch bulbs should be installed above the mirror and its switch should be installed next to it, the minimum bulbs wire cross section should be 2.5mm².

For areas with humid and wet weather, the waterproof bulbs should be installed, the reason for installing waterproof bulbs that wet weather doesn’t damage the bulbs, and it is usually installed in the bathroom and carwash.

Brightness of rooms according to the Standard:

A bulb installation for study room should be 500 lux.

A bulb installation for administrative office should be 300Lux.

A bulb installation for setting room should be 300 Lux.

A bulb installation for bathroom should be 100 Lux.
A bulb installation for corridors should be 150 Lux.

A bulb installation for meeting room should be 700 Lux.

Some unspecified rooms bulbs installation should be similar and according to the project standard.

Colors of Cables according to Voltage of 1000v

In the cables up to 1000v voltage internal wires are specify with types of colors, but in cables higher voltage of 1000v should be careful all wires colors are same.

The colors of wires according to standard code of VDE are specified as follow.

Cable of two wires: light gray and black.

Cable of three wires: light gray, black, and red.

Cable of four wires: light gray, black, red and blue.

Cable of five wires: light gray, black, red, blue and green.

In the VED standard, for specification of cables up to 1000v voltage, color of light grey is a 0 voltage, red color wire is for protection and if the cable has complete different color then is use for a 0 voltage wire.

Note: in some other code of standard blue color is use as 0 voltage and yellow color with off green together are used for protection or a grounding system.

Protective system, Equipment of electricity and fuse boxes:

Fuse should be selected carefully and adequately, otherwise could lead to short-circuit.

Fuse boxes should be place somewhere that can be easily access and visible, so the responsible unit can control the short circuit.

The minimum size of fuse for light of switch should be 16 ampere.

A vertical distance from concrete floor to the top of fuse box should be 180cm.

Fuses:

The sample and common method of protection for circle short and increasing power in fuses are to allow extra fuses, short term increasing power in fuses called extra loading, commonly does not have effect on component and electrical circulation system, generally is not control or stop by fuses but in the situation of short fuses immediately action should be taken to stop circulation.

A melted or slack fused are works that circulation of electricity with a single wire which place within fire proof circle and circulate in the normal situation and stand allowable circulation, in the situation the allowable circulation is increase then the wire fuse produce warmth(heat) for some period and stop circulation.
Fuses are used for protection of cables, electronic machines, transformers, a system for measurement and protection for users and used against extra circulation and short circle.

Fuses are based on voltages and structure is divided as follow.

Warmth fuse (melted)

Warmth fuse (none metal)

Magnetism fuse

High voltage fuse

The following table is for specifying melt fuses based on, color and allowable circulation.

<table>
<thead>
<tr>
<th>Allowable Circulation in fuses</th>
<th>Colours (fireproof cotton) of Button</th>
<th>Allowable Circulation in fuses</th>
<th>Colours (fireproof cotton) of Button</th>
</tr>
</thead>
<tbody>
<tr>
<td>2</td>
<td>Pink</td>
<td>50</td>
<td>White</td>
</tr>
<tr>
<td>4</td>
<td>Light Brown</td>
<td>63</td>
<td>Black</td>
</tr>
<tr>
<td>6</td>
<td>Green</td>
<td>80</td>
<td>Silver</td>
</tr>
<tr>
<td>10</td>
<td>Light Red</td>
<td>100</td>
<td>Dark Red</td>
</tr>
<tr>
<td>16</td>
<td>Grey</td>
<td>125</td>
<td>Yellow</td>
</tr>
<tr>
<td>20</td>
<td>Blue</td>
<td>160</td>
<td>Copper</td>
</tr>
<tr>
<td>25</td>
<td>Yellow</td>
<td>200</td>
<td>Blue</td>
</tr>
</tbody>
</table>

Electrical Supply System and Resistance of Grounding:

The system of electricity supply for single circulation fuse is 220 voltages and for three circulations fuse is 380 voltages.
The frequency system of electricity supply is 50 hertz.

In the area of humid and wet weather waterproof component should be used.

In the government building or offices neon fluorescent bulbs should be used.

Resistance of a grounding system should not be more than 20 Ohms.

A minimum cross section of a grounding rod should be 20mm.

**Ventilation System**

Ventilation system should be considered for bathroom, Toilet, Kitchen, wet places, and stinks area.

Switch should be next to ventilation or should be control form specify switch, wire cross section should be 2.5 or 4mm, bathrooms ventilation dimension should be 20X20 cm and plastic material, for other places should be design by mechanical system designer.

**Loss of Voltages:**

The maximum loss of voltage from supply resource to fuse box is 2%.

The maximum loss of voltage from main fuse box to users is 3%.

The maximum loss of voltage from supply resource to users is 5%.
5 Electric symbols

- Single fuse socket
- Single fuse socket with earth
- Plug
- Plug with earth
- Resistance to undo activity
- Active resistance
- Condenser
- Normal bulbs
- Fluorescent Bulbs
- Join kilts
- Temperature Rail
- Dis-connective switch
- Connective switch
- Magnetism fuse
- Micro Switch
- Main switch

- Transformer
- Transformer for measurement of voltage
- Transformer for measurement of circulation
- Motor
- Amber meter
- Voltage Meter
- Watt meter
- Single switch Pole
- Double Switch Pole
- Vexiel switch
- Cross switch
- Kilo watt-Hour
- Conductor
- Lockable Switch
- Electrical Heat or
- Mechanical joints
6 References

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Goulburn Valley Water Manuel, Design Guideline water Storage Tank (Ground)


### 7 Appendix A

**SI Conversion Factors**

#### Inch-Pound Units to SI Units

<table>
<thead>
<tr>
<th>Overall Geometry</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Spans</td>
<td>1 ft = 0.3048 m</td>
</tr>
<tr>
<td>Displacements</td>
<td>1 in = 25.4 mm</td>
</tr>
<tr>
<td>Surface Area</td>
<td>1 ft² = 0.0929 m²</td>
</tr>
<tr>
<td>Volume</td>
<td>1 ft³ = 0.0283 m³, 1 yd³ = 0.765 m³</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Structural Properties</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Cross-Sectional Dimensions</td>
<td>1 in = 25.4 mm</td>
</tr>
<tr>
<td>Area</td>
<td>1 in² = 645.2 mm²</td>
</tr>
<tr>
<td>Section Modulus</td>
<td>1 in³ = 16.39 x 10³ mm³</td>
</tr>
<tr>
<td>Moment of Inertia</td>
<td>1 in⁴ = 0.4162 x 10⁶ mm⁴</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Material Properties</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Density</td>
<td>1 lb/ft³ = 16.03 kg/m³</td>
</tr>
<tr>
<td>Modulus and Stress</td>
<td>1 lb/in² = 0.006895 MPa, 1 kip/in² = 6.895 MPa</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Loadings</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Concentrated Loads</td>
<td>1 lb = 4.448 N, 1 kip = 4.448 KN</td>
</tr>
<tr>
<td>Density</td>
<td>1 lb/ft³ = 0.1571 KN/m³</td>
</tr>
</tbody>
</table>
### Linear Loads

|          | 1 kip/ft = 14.59 KN/m |

### Surface Loads

|          | 1 lb/ft² = 0.0479 KN/m²  
|          | 1 kip/ft² = 47.9 KN/m² |

### Stress and Moments

#### Stress

|          | 1 lb/in² = 0.006895 MPa  
|          | 1 kip/in² = 6.895 MPa |

#### Moment or Torque

|          | 1 ft – lb = 1.356 N – m  
|          | 1 ft – kip = 1.356 KN – m |