FACULTAD DE INGENIERIA U.N.A.M.
DIVISION DE EDUCACION CONTINUA

A LOS ASISTENTES A LOS CURSOS

Las autoridades de la Facultad de Ingeniería, por conducto del jefe de la División de Educación Continua, otorgan una constancia de asistencia a quienes cumplan con los requisitos establecidos para cada curso.

El control de asistencia se llevará a cabo a través de la persona que le entregó las notas. Las inasistencias serán computadas por las autoridades de la División, con el fin de entregarle constancia solamente a los alumnos que tengan un mínimo de 80% de asistencias.

Pedimos a los asistentes recoger su constancia el día de la clausura. Estas se retendrán por el periodo de un año, pasado este tiempo la DECFI no se hará responsable de este documento.

Se recomienda a los asistentes participar activamente con sus ideas y experiencias, pues los cursos que ofrece la División están planeados para que los profesores expongan una tesis, pero sobre todo, para que coordinen las opiniones de todos los interesados, constituyendo verdaderos seminarios.

Es muy importante que todos los asistentes llenen y entreguen su hoja de inscripción al inicio del curso, información que servirá para integrar un directorio de asistentes, que se entregará oportunamente.

Con el objeto de mejorar los servicios que la División de Educación Continua ofrece, al final del curso deberán entregar la evaluación a través de un cuestionario diseñado para emitir juicios anónimos.

Se recomienda llenar dicha evaluación conforme los profesores impartan sus clases, a efecto de no llenar en la última sesión las evaluaciones y con esto sean más fehacientes sus apreciaciones.

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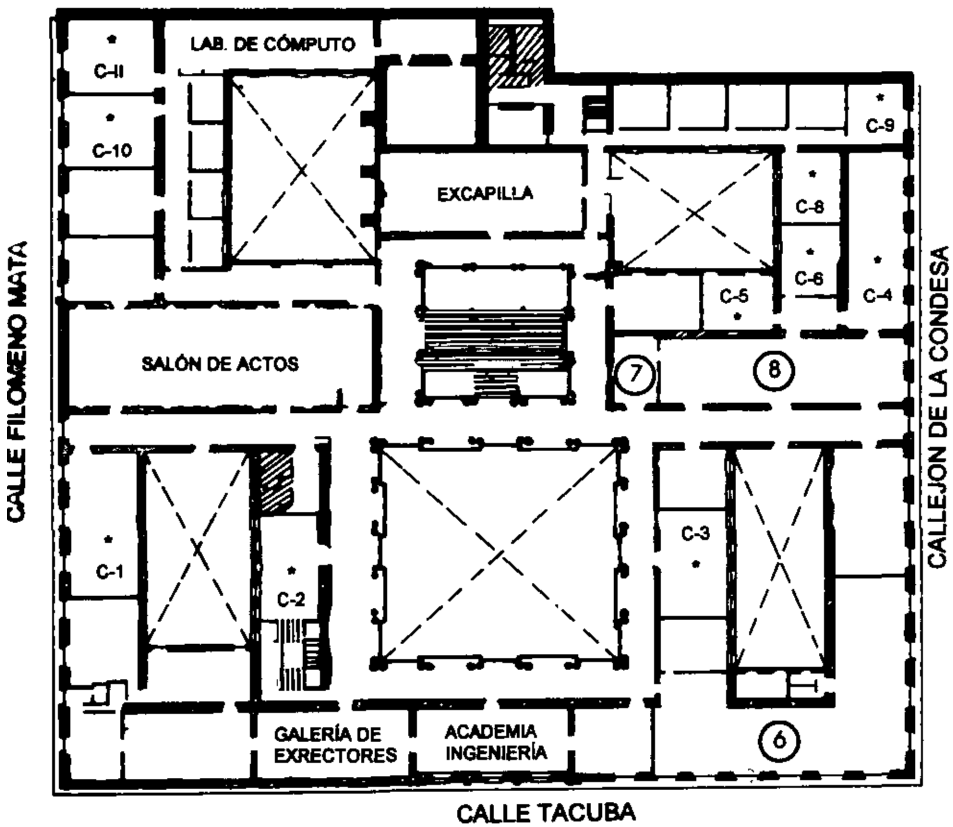

1er. PISO

GUÍA DE LOCALIZACIÓN

1. ACCESO
2. BIBLIOTECA HISTÓRICA
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4. CENTRO DE INFORMACIÓN Y DOCUMENTACIÓN "ING. BRUNO MASCANZONI"
5. PROGRAMA DE APOYO A LA TITULACIÓN
6. OFICINAS GENERALES
7. ENTREGA DE MATERIAL Y CONTROL DE ASISTENCIA
8. SALA DE DESCANSO

SANITARIOS

* AULAS

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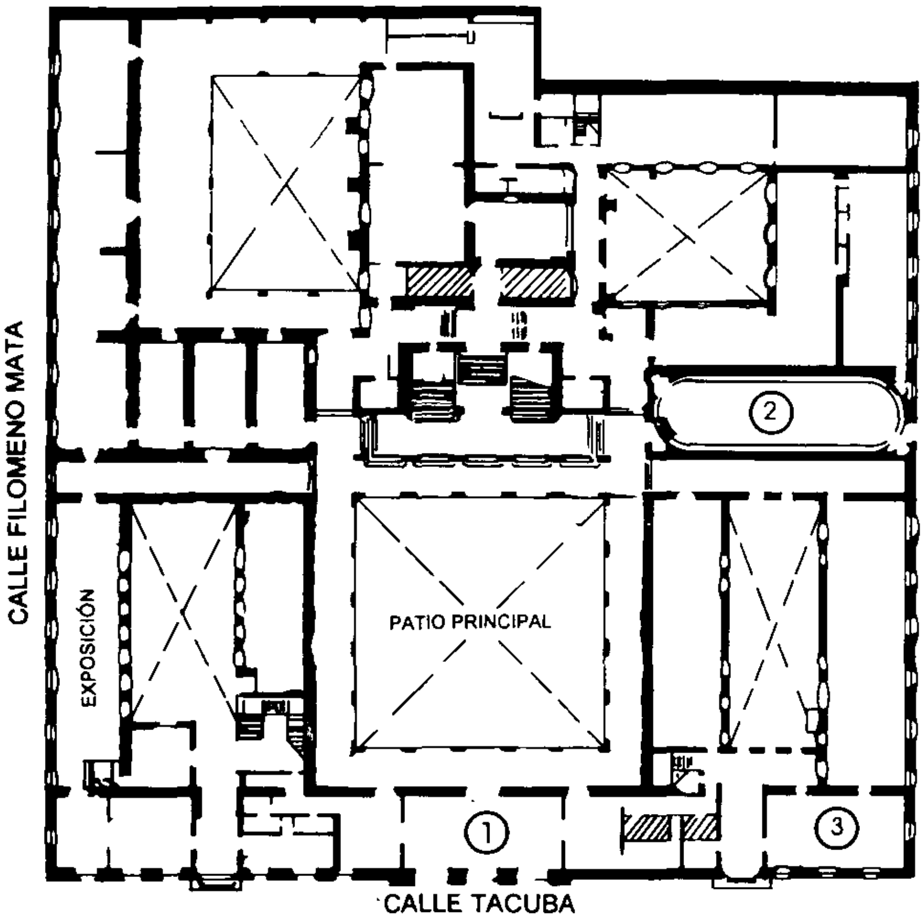

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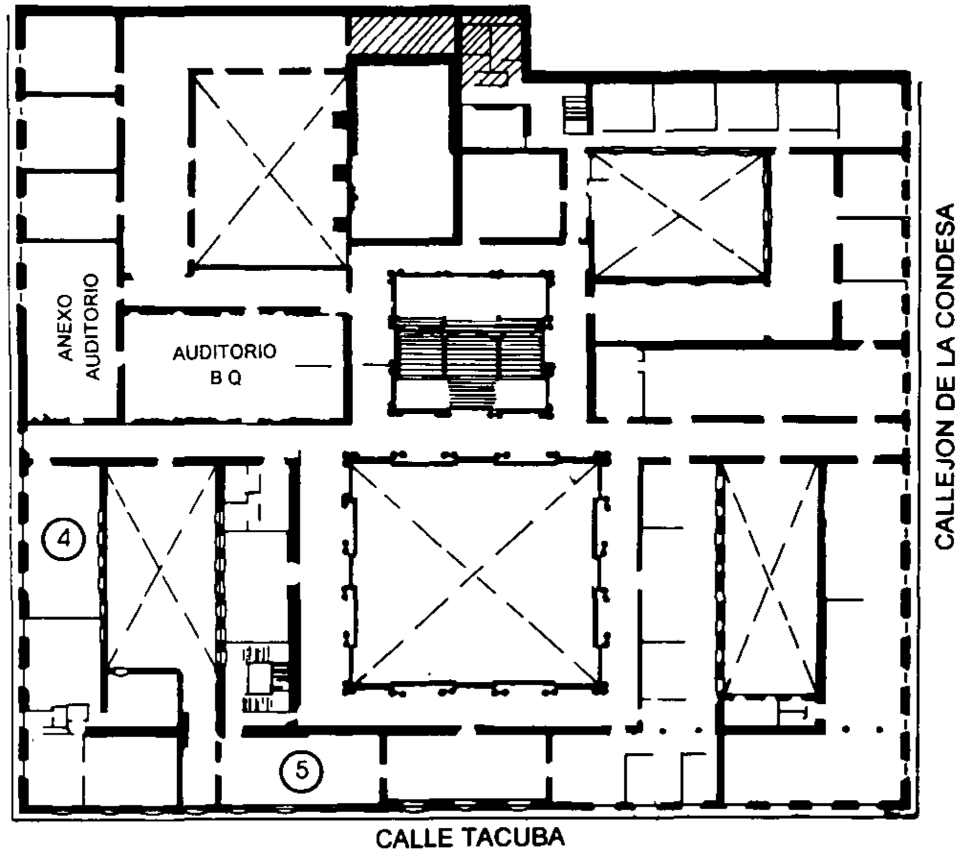

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FACULTAD DE INGENIERIA U.N.A.M.
DIVISION DE EDUCACION CONTINUA

CURSOS INSTITUCIONALES
COMISIÓN DEL AGUA DEL ESTADO DE MÉXICO

EQUIPAMIENTO DE LINEAS DE CONDUCCIÓN
Del 18 al 22 de octubre de 1999.

Apuntes Generales.

Ing. Alfonso Rodríguez Navidad
Palacio de Minería
1999.

Palacio de Minería Calle de Tacuba 5 Primer piso Deleg. Cuauhtémoc 06000 México, D.F. APDO. Postal M-2285
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CURSO DE EQUIPAMIENTO EN SISTEMAS DE BOMBEO

TEMARIO.

I. AREAS QUE SE INVOLUCRAN EN EL EQUIPAMIENTO DE UN SISTEMA, BASE PROGRAMA INTEGRAL.

1.- INGENIERIA :	PROCESO EQUIPO CONSTRUCCION
2.- ABASTECIMIENTOS :	LICITACIONES COSTOS Y PRECIOS UNITARIOS INSPECCION Y EXPEDITACION
3.- FINANZAS :	PROGRAMA DE INVERSIONES RECURSOS FINANCIEROS

4.- DEPARTAMENTO JURIDICO O LEGAL : CONDICIONES LEGALES Y COMERCIALES.

II.- SECUENCIA DEL EQUIPAMIENTO.

1.- INGENIERIA : GENERA LA DE DOCUMENTACION DE :

1.0.- CRITERIOS DE DISEÑO DE LOS EQUPOS Y SISTEMAS, CRITERIOS DE SELECCIÓN, TIPOS DE BOMBAS Y ARREGLOS.

1.1.- DIAGRAMAS DE TUBERIA E INSTRUMENTACION.

1.2.- PARAMETROS DE DISEÑO DE LOS EQUIPOS.

1.3.- DESCRIPCION DE LOS SISTEMAS.

1.4.- LISTAS DE EQUIPO Y MATERIALES : MECANICOS, ELECTRICOS, CONTROL E INSTRUMENTACION, CIVILES.

1.5.- PLANOS DE ARREGLO ; DE PLANTA Y DE EQUIPOS.

1.6.- ESPECIFICACIONES DE ADQUISICION.

1.7.- REQUISICIONES.

1.8.- SOLICITUDES DE COMPRA

1.9.- RECIBE INFORMACION FINAL DE EQUIPOS Y MATERIALES PARA APLICAR PARTICULARIDADES EN PLANOS APROBADOS PARA CONSTRUCCION.

2.- ABASTECIMIENTOS TRAMITA :

2.1.- CONVOCATORIA PARA LICITACION.

2.2.- LICITACION O CONCURSO.

2.3 - RECIBE OFERTAS DE CONCURSANTES.

2.4.- EVALUA OFERTAS COMERCIALES Y FINANCIERAS CON APOYO DE FINANZAS.

2.5.- ENVIA OFERTAS A INGENIERIA PARA EVALUACION TECNICA.

2.6.- ELABORA DICTAMEN UNA VEZ QUE LE RETORNAN RECOMENDACIÓN DE EVALUACION TECNICA Y FINANCIERA.

2.7.- EMITE FALLO DE CONCURSO Y FINCA PEDIDO DE FIRME.

2.8.- ESTABLECE PROCEDIMIENTO DE COMUNICACIÓN ENTRE PROVEEDOR Y AREAS DE INGENIERIA Y ABASTECIMIENTOS.

2.9.- PROMUEVE DE COMUN ACUERDO CON INGENIERIA LAS REUNIONES TECNICAS PARA RECEPCION DE INFORMACION TECNICA Y DEMAS.

2.10.- ESTABLECE COMPROMISOS DE ENTREGA DE INFORMACION TECNICA.

2.11.- DEFINE TIEMPOS DE ENTREGA CONTRACTUALES TANTO DE INFORMACION TECNICA Y DEMAS COMO TIEMPOS DE ENTREGA DE LOS EQUIPOS.

III.- ENLACE CON ACTIVIDADES DE CONSTRUCCION.

1.- UNA VEZ QUE SE HA LOGRADO LA ETAPA DE EQUIPAMIENTO DE UN SISTEMA, Y DE ACUERDO CON EL PROGRAMA INTEGRAL DEL PROYECTO SE ESTABLECE EL FLUJO DE INFORMACION DE INGENIERIA Y ABASTECIMIENTOS CON EL AREA DE CONSTRUCCION.

2.- INICIO DE ACTIVIDADES DE CONSTRUCCION DE EQUIPOS PRINCIPALES DEL SISTEMA.

ESTE TEMA NO FORMA PARTE DEL CURSO ; SOLO SE ENUNCIARA Y SE COMENTARA.

CURSO DE EQUIPAMIENTO EN SISTEMAS DE BOMBEO

INTRODUCCION

EL OBJETIVO DE ESTE CURSO ES ESTABLECER UN METODO PARA LLEVAR A CABO LA DOTACION DE EQUIPO REQUERIDOS EN UN SISTEMA DE BOMBEO, DE ACUERDO CON LOS RESULTADO DE UN DISEÑO, ADQUISICION DEL EQUIPO ASI COMO LA INSTALACION DE LOS EQUIPOS DE FORMA QUE ESTOS FUNCIONEN EN SERVICIO Y CUMPLAN CON PROPORCIONAR EL SERVICIO PARA EL CUAL FUERON CONSIDERADOS EN EL PROYECTO DEL SISTEMA.

EN ESTE CURSO SE HARA UNA DESCRIPCION GENERAL DEL PROYECTO, ASI COMO LOS FINES PARA LOS CUALES SE EJECUTO Y CUMPLAN CON LAS NECESIDADES DE UN SERVICIO ESPECIFICO DEMANDADO.

GENERALIDADES

EL EQUIPAMIENTO DE UN SISTEMA DE BOMBEO TIENE COMO FINALIDAD: LA ADQUISICION DE LOS EQUIPOS QUE LO CONFORMAN PARA LOGRAR QUE EL SISTEMA CUMPLA ADECUADAMENTE SUS FUNCIONES ESPECIFICAS UNA VEZ QUE HA SIDO INSTALADO, PROBADO Y PUESTO EN OPERACIÓN.

POR LO TANTO, ES NECESARIO ESTABLECER :

I. AREAS QUE INTERVIENEN EN EL EQUIPAMIENTO DE UN SISTEMA, EN BASE A UN PROGRAMA INTEGRAL DE UN PROYECTO.

1.- INGENIERIA :
- a- PROCESO
- b.- EQUIPO
- c.- CONSTRUCCION

2.- ABASTECIMIENTO.-
- a.- LICITACIONES
- b.- COSTOS Y PRECIOS UNITARIOS
- c.- INSPECCION Y EXPEDITACION

3 - FINANZAS.-
- a.- PROGRAMA DE INVERSIONES
- b.- RECURSOS FINANCIEROS

4.- DEPTO. JURIDICO O LEGAL
- a.- ESTABLECIMIENTO DE CONDICIONES CONTRACTUALES.
- b.- ESTABLECIMIENTO DE CONDICIONES COMERCIALES.

II. ESTABLECIMIENTO DE ACTIVIDADES SECUENCIALES PARA EL EQUIPAMIENTO.

PARA LO ANTERIOR ES NECESARIO ESTABLECER Y PROGRAMAR QUE AREAS DE UN PROYECTO EN PARTICULAR INTERVIENEN Y QUE TIPO DE ACTIVIDADES CORRESPONDE A CADA UNA PARA QUE EN FORMA COORDINADA SE CUBRAN LAS ETAPAS DE DISEÑO APROBADO DE LOS EQUIPOS, TRAMITES DE ADQUISICION, COMPRA DE EQUIPO Y FINALMENTE INSTALACION DEL MISMO (AUNQUE EN ESTE CURSO SOLO SE ENUNCIA ESTA ACTIVIDAD).

GENERALMENTE LA MAYORIA DE EMPRESAS DE INGENIERIA SE ORGANIZA DE FORMA TAL QUE DENTRO DE LA ORGANIZACIÓN DE LAS MISMAS, SE ESTABLECEN AREAS DE TRABAJO QUE LLEVAN A CABO LAS FUNCIONES DE DISEÑO, ABASTECIMIENTO O PROCURACION, CONSTRUCCION, PRUEBAS Y PUESTA EN MARCHA DE LOS EQUIPOS PARA PROYECTOS ESPECIFICOS.

CADA EMPRESA PARTICULAR O DEL SECTOR PRIVADO, ASI COMO LAS ENTIDADES DEL SECTOR PUBLICO QUE LLEVAN A CABO PROYECTO DE INGENIERIA Y CONSTRUCCION SE HAN ORGANIZADO QUE PARA CADA UNA DE LAS AREAS QUE LA FORMAN LLEVE A CABO LAS FUNCIONES Y RESPONSABILIDADES QUE LES CORRESPONDA Y DE ESTA FORMA SE CUMPLAN CABALMENTE CON LA REALIZACION DE UN PROYECTO O DE UNA OBRA PARA LOS FINES QUE ESTA FUE PREVIAMENTE PROYECTADA.

DE ESTA FORMA PARTICULARIZANDO EN LO REFERENTE A SISTEMAS DE BOMBEO, SE PUEDE PARTIR DE LOS SIGUIENTE :

DESARROLLO :

LAS AREAS QUE INTERVIENEN EN EL EQUIPAMIENTO DE SISTEMAS DE BOMBEO SON :

1. INGENIERIA.

a.- PROCESO : ESTA AREA EN SU ACTIVIDAD DE <u>PROCESO</u> ES RESPONSABLE DE LLEVAR A CABO LOS ESTUDIOS PRELIMINARES Y DEFINITIVOS QUE LLEVAN FINALMENTE A LA REALIZACION DE UN PROYECTO.

ESTABLECE Y DEFINE EL SISTEMA A TRAVES DE DIAGRAMAS DE FLUJO DEL <u>PROCESO</u>, <u>DIAGRAMAS DE TUBERIA E INSTRUMENTACION.</u>

DEFINE LOS CRITERIOS DE DISEÑO.

CALCULO DE PARAMETROS DE DISEÑO DE EQUIPO Y ACCESORIOS.

CRITERIOS DE SELECCIÓN : BOMBAS, RECIPIENTES, DISPOSITIVOS, ETC.

MODOS DE OPERACIÓN.

ARREGLO DE LOS EQUIPOS : ARREGLO DE PLANTA Y ARREGLO DE EQUIPO.

TIPOS DE BOMBAS APROPIADOS PARA EL SERVICIO ESPECIFICO.

DESCRIPCION DEL SISTEMA Y CONTROL.

EMITE LISTA DE EQUIPO.

SELECCIONA MATERIALES ADECUADOS AL SERVICIO.

UNA VEZ QUE LA DOCUMENTACION RELACIONADA CON LAS ACTIVIDADES ANTERIORMENTE DESCRITAS HA SIDO AVALADA Y APROBADA DE ACUERDO A PROCEDIMIENTOS DE CALCULO, SE ENVIA AL AREA DE EQUIPO.

b.- EQUIPO : EL AREA DE EQUIPO, EN BASE A LA INFORMACION DE PROCESO, SE ABOCA A PREPARAR LAS ESPECIFICACIONES DE ADQUISICION DE LOS EQUIPOS DEL SISTEMA.

LAS ESPECIFICACIONES DE ADQUISICION DEBEN SER LO SUFICIENTEMENTE CLARAS Y TAN AMPLIAS COMO PARA CUMPLIR CON LOS CRITERIOS DE DISEÑO. DEBERAN CUBRIR LOS ASPECTOS FUNCIONALES, EFICIENCIAS ADECUADAS, FACILIDADES DE OPERACIÓN, MANTENIMIENTO, SEGURIDAD DEL EQUIPO Y DEL PERSONAL DE OPERACIÓN, MATERIALES DE ADQUISICION NORMAL EN EL MERCADO O DE IMPORTACION SI ES PRECISO.

ES IMPORTANTE QUE EN ESTA ACTIVIDAD SE HAGA ACOPIO DE INFORMACION TAL COMO COGIDOS DE APOYO, NORMATIVA NACIONAL E INTERNACIONAL, RECOMENDACIONES DE EXPERTOS Y DE FABRICANTES, EXPERIENCIA EN ACTIVIDADES DE INGENIERIA.

LAS ESPECIFICACIONES DE ADQUISICIONES DEBEN DE CONTENER LO SIGUIENTE, DE PREFERENCIA :

OBJETIVO Y CAMPO DE APLICACIÓN
NORMAS
DATOS DE PARAMETROS DE DISEÑO BASICOS
ALCANCE DEL SUMINISTRO
CONDICIONES DE OPERACIÓN
CARACTERISTICAS TECNICAS
CARACTERISTICAS PARTICULARES
TIEMPOS DE ENTREGA
INFORMACION CON LA OFERTA, INFORMACION DE FABRICANTE
INFORMACION DESPUES DE COLOCADA LA ORDEN DE COMPRA

UNA VEZ QUE LA DOCUMENTACION SE HA REUNIDO Y ORDENADO EN EL DOCUMENTO DE EMISION, REVISION Y APROBACION. SE PROCEDE A ELABORAR UNA REQUISICION, DONDE SE SOLICITA SE LLEVEN A CABO LOS TRAMITES PERTINENTES PARA LA ADQUISICION DE LOS BIENES MATERIALES Y EQUIPOS, INDICANDO LAS FECHAS CLAVE DE LOS EVENTOS PRINCIPALES CON BASE AL PROGRAMA INTEGRAL DEL PROYECTO, ENVIANDO ESTA REQUISICION CON TODA LA DOCUMENTACION QUE COMPRENDE, AL AREA DE ABASTECIMIENTO.

ESTA REQUISICION ES ACOMPAÑADA DE UNA SOLICITUD DE COMPRA DE LOS BIENES QUE SE INDICAN CLARA Y AMPLIAMENTE EN LA ESPECIFICACION

c.- CONSTRUCCION : EN ESTE CURSO NO SE DETALLAN ESTAS ACTIVIDADES, SOLO SE ENUNCIARAN.

2. ABASTECIMIENTOS

a.- LICITACIONES: CON TODA LA DOCUMENTACION DE INGENIERIA, EL AREA DE LICITACIONES DEBE DE PREPARAR Y TRAMITAR LO QUE LE CORRESPONDE DENTRO DE LA ORGANIZACIÓN PARA LLEVAR A CABO LA LICITACION O CONCURSO PARA LA ADQUISICION DE LOS EQUIPOS CORRESPONDIENTES.

PARA TAL OBJETO PREPARARA UNA CONVOCATORIA SOLICITANDO A DIVERSOS FABRICANTES O PROVEEDORES QUE SE TIENE EN CATALOGO DE PROVEEDORES Y QUE ESTAN FAMILIARIZADOS CON LOS EQUIPOS MOTIVO DEL CONCURSO.

CONFORME SE VAN CUMPLIENDO DETERMINADAS FECHAS DEL PROGRAMA SE RECIBIRAN OFERTAS DE EQUIPOS QUE EN FECHA DETERMINADA SERAN ANALIZADAS POR ESTA AREA PARA FIJAR O DEFINIR UN CONCURSANTE GANADOR AL QUE SE LE ADJUDICARA EL CONTRATO, UNA VEZ QUE SE DICTE EL FALLO DEL CONCURSO.

b.- COSTOS Y PRECIOS: UNITARIOS: ESTA AREA SE ABOCARA A DETERMINAR LOS COSTOS Y PRECIOS UNITARIOS PARA QUE SE INTEGREN AL ANALISIS DE LAS OFERTAS Y QUE SERAN PARTE DE LA DECISION PARA ELEGIR LA MEJOR OFERTA.

c.- INSPECCION Y EXPEDITACION: POR LO QUE RESPECTA A ESTA AREA ES AUXILIAR AL DEPARTAMENTO DE ABASTECIMIENTOS PARA QUE UNA VEZ QUE SE TENGA CONCURSANTE GANADOR DEL CONTRATO SE INSPECCIONEN LOS EQUIPOS TANTO EN SU ALCANCE DE FABRICACION COMO EN SU ENTREGA; TODO ESTO SEGÚN SE ESTABLEZCA EN EL CONTRATO DE SUMINISTRO.

A SU VEZ VIGILARA LA EXPEDITACION O AVANCE DE FABRICACION PARA ASEGURAR EL CUMPLIMIENTO DE FECHA PACTADOS DE ENTREGA DE LOS EQUIPOS.

CABE MENCIONAR QUE DURANTE LA ETAPA DE FABRICACION EL PROVEEDOR SE COMPROMETE AL ENVIO DE INFORMACION DE INGENIERIA PARA QUE LAS PARTICULARIDADES DE SU EQUIPO SE INCLUYAN EN LA INFORMACION DE INGENIERIA DEL USUARIO COMO PARTE DE LA INFORMACION QUE DEBE SER APROBADA PARA CONSTRUCCION.

3. FINANZAS

a.- PROGRAMA DE INVERSIONES: ESTA AREA TIENE COMO FINALIDAD LA EMISION DEL PROGRAMA DE INVERSIONES A CORTO, MEDIANO O LARGO PLAZO CADA EMPRESA O ENTIDAD FEDERAL DEBE DE DESARROLLAR EN FUNCION DE UN PROGRAMA DE OBRAS E INVERSIONES POR PERIODOS DE TIEMPO QUE SE ANALIZAN EN FUNCION DE SERVICIOS O INSTALACIONES

QUE SE REQUIERAN CUBRIR SEGÚN LOS PERIODOS O PREDICCIONES YA SEA PARA SATISFACER NECESIDADES DE CONSUMO O AMPLIAR LAS REDES DE SERVICIOS POR CUBRIR, TANTO PUBLICOS COMO PRIVADOS.

b.- RECURSOS FINANCIEROS: ESTA AREA ES RESPONSABLE DE ANALIZAR Y OBTENER LOS RECURSOS FINANCIEROS PARA SATISFACER O CUBRIR A SU VEZ LOS PROGRAMAS DE OBRAS E INVERSIONES ; DICHOS RECURSOS PUEDEN SER PROPIOS DE LAS EMPRESAS O ENTIDADES O MEDIANTE PRESTAMO DEL EXTERIOR.

4. **DEPARTAMENTO LEGAL O JURIDICO**

a,b : ESTA AREA SERA LA RESPONSABLE DE APOYAR A INGENIERIA Y ABASTECIMIENTO PARA ANALIZAR Y DEFINIR LOS ASPECTOS LEGALES Y COMERCIALES QUE SE INCLUYEN EN LOS DOCUMENTOS DE ESPECIFICACIONES PARA LA ADQUISICION DE LOS BIENES POR ADQUIRIRSE COMO SON: CUMPLIMIENTO DE COMPROMISOS CONTRAIDOS, TIEMPOS DE ENTREGA DE INFORMACION Y DE LOS EQUIPOS, PENALIZACIONES POR INCUMPLIMIENTO, VARIACIONES DE PRECIOS DEBIDO A INFLACIONES, ESCALACIONES, CAMBIOS O SUBESTITUCIONES CONVENIDAS, ETC.

HASTA AQUÍ SE CONSIDERA LA DESCRIPCION DETALLADA DE LAS ETAPAS PRIMORDIALES DEL EQUIPAMIENTO.

POR LO QUE RESPECTA A **II SECUENCIA DEL EQUIPAMIENTO** ; ESTA SERA CUBIERTA MEDIANTE EJEMPLOS QUE MUESTRAN EN DETALLE CADA UNO DE LOS EVENTOS ENUNCIADOS QUE SE PROGRAMARAN Y SE DISCUTIRAN EN CADA UNA DE LAS 5 SESIONES DEL CURSO.

PROGRAMA DE EJECUCION DEL CURSO	
LUNES	INGENIERIA ABASTECIMIENTOS
MARTES	INGENIERIA ABASTECIMIENTOS
MIERCOLES	SISTEMAS BASICOS DISCUSION
JUEVES	EJEMPLOS FINANZAS
VIERNES	DEPARTAMENTO JURIDICO O LEGAL. EVALUACION, RESUMEN, OBSERVACIONES Y COMENTARIOS :

FACULTAD DE INGENIERIA U.N.A.M.
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CURSOS INSTITUCIONALES
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Del 18 al 22 de octubre de 1999.

Anexos.

Ing. Alfonso Rodríguez Navidad
Palacio de Minería
1999.

The Pressure Piping Code also calls attention to the following weight effects which should be taken into account in the design of piping:

1. Live loads such as the weight of the fluid transported and snow and ice loads if the latter will be encountered. If low-temperature piping is not insulated, there can be a build-up of ice on the pipe even in high ambient temperatures
2. Dead loads, consisting of the weight of the piping components and insulation and other superimposed loads.
3. Test loads which consist of the weight of the test fluid in the pipe.

REFERENCES

1. American Standard Safety Code for Mechanical Refrigeration, ASA B9.1-1964, American Society of Heating, Refrigerating and Air Conditioning Engineers, Inc., United Engineering Center, 345 E. 47th St., New York, N.Y. 10017, American Standards Association, Inc., 10 E. 40th St., New York, N.Y. 10016.
2. Refrigeration Piping, ASA B31.5, American Society of Mechanical Engineers, United Engineering Center, 345 E. 47th St., New York, N.Y. 10017, American Standards Association, Inc., 10 E. 40th St., New York, N.Y. 10016.
3. ASME Boiler and Pressure Vessel Code, Section VIII, Unfired Pressure Vessels, American Society of Mechanical Engineers, United Engineering Center, 345 E. 47th St., New York, N.Y. 10017.
4. "Heating, Ventilating and Air Conditioning Guide," American Society of Heating, Refrigerating and Air Conditioning Engineers, United Engineering Center, 345 E. 47th St., New York, N.Y. 10017.

"Refrigerant Piping Data," Air Conditioning and Refrigeration Institute, 1815 North Fort Myer Drive, Arlington, Va. 22209.

"Soldered and Brazed Joints in Copper Tube," Publication No. 25, Copper Development Association, Inc., 405 Lexington Ave., New York, N Y. 10017

18

SEWERAGE-SYSTEMS PIPING

William E. Dobbins*

This chapter deals with the design of sewerage systems which perform the functions of collecting water-borne wastes of domestic, commercial, and industrial origin and of storm-water runoff and conveying them to points of disposal.

The design of sewerage systems which carry domestic or industrial wastes must comply with the minimum standards of the city, county, and state regulatory agencies. Plans must ordinarily be approved, and permits must be obtained for the disposal of domestic and industrial wastes into natural water courses. When industrial wastes are to be disposed of by conveyance to a public sewerage system, the quantity and quality of the waste must ordinarily be in compliance with the local sewer ordinance.

DEFINITIONS

Terms commonly used in relation to sewerage systems have been defined as follows:[1,†]

Sewer. A pipe or conduit, generally closed but normally not flowing full, for carrying sewage and other waste liquids.

Sewage. Largely the water supply of a community after it has been fouled by various uses. From the standpoint of source, it may be a combination of the liquid or water-carried wastes from residences, business buildings, and institutions,

* Professor of Sanitary Engineering, New York University, New York, N.Y.; Partner, Tector-Dobbins, Consulting Engineers, West Islip, N.Y.

† Superscript numbers refer to corresponding entries in a list of references included at the end of this chapter.

together with those from industrial establishments, and with such ground water, surface water, and storm water as may be present.

Sanitary Sewer. A sewer which carries sewage and to which storm, surface, and ground waters are not intentionally admitted; also referred to as "separate sanitary sewer" or "separate sewer."

Storm Sewer. A sewer which carries storm water, surface water, street wash, and other wash waters or drainage but excludes sewage and industrial wastes; also called "storm drain."

Combined Sewer. A sewer receiving both surface runoff and sewage.

Building Sewer. The extension from a building drain to the public sewer or other place of disposal; also called the "house sewer" or "house connection."

Lateral Sewer. A sewer which discharges into a branch or other sewer and has only building sewers tributary to it.

Branch Sewer. A sewer which receives sewage from a relatively small area and discharges into a main sewer.

Main Sewer. A sewer to which one or more branch sewers are tributary and which serves a large territory; also called "trunk sewer."

Intercepting Sewer. A sewer which receives dry-weather flow from a number of transverse sewers or outlets and frequently additional predetermined quantities of storm water (if from a combined system) and conducts such waters to a point for treatment or disposal.

Outfall Sewer. A sewer which receives sewage from a collecting system and carries it to a point of final discharge.

Separate System. A sewer system comprised exclusively of sanitary sewers which carry only sewage and to which storm water, surface water, and ground water are not intentionally admitted; also referred to as "sanitary system" or "separate sanitary system."

Storm-sewer System. A system composed only of sewers carrying storm water, surface water, street wash, and other wash waters or drainage and from which sewage and industrial wastes are excluded.

Combined-sewer System. A system of sewers receiving both surface runoff and sewage.

QUANTITY OF SANITARY SEWAGE

General Considerations. Sanitary sewers must be designed to provide capacity for the present and estimated future quantities of domestic sewage, commercial and industrial wastes, and ground-water infiltration. Lateral and branch sewers should be designed for the ultimate population density to be expected in the area served. Larger sewers are commonly designed to handle the flows to be expected from 25 to 50 years in the future. The estimation of future flows should be arrived at only after a detailed study of the land usage, population growth trends, water-consumption rates, commercial and industrial growth, etc.

The prediction of future populations is made with much more certainty for large areas and dense populations than for small areas and low population densities. Figure 1 shows capacity factors given by S. A. Greeley and W. A. Stanley[2] for use in making allowances for the uncertainties of population distributions within a sewer district. The possible future population density for a portion of a large district may be estimated by multiplying the estimated future average density for the entire district by the appropriate factor.

The most important single index to the flow of sanitary sewage is the rate of consumption of water. The total consumption of water within a town or sewer district as a whole is generally considerably higher than the purely domestic

consumption, the difference being attributable to the use in public institutions, office buildings, commercial and industrial establishments, etc., and to the leakage from the water-distribution system. When estimates of sanitary sewage flow from small areas are being prepared, the most accurate procedure is to make separate estimates of the various classifications of flow which make up the total. The classifications which are commonly used are domestic, commercial, and industrial sewage flows and ground-water infiltration.

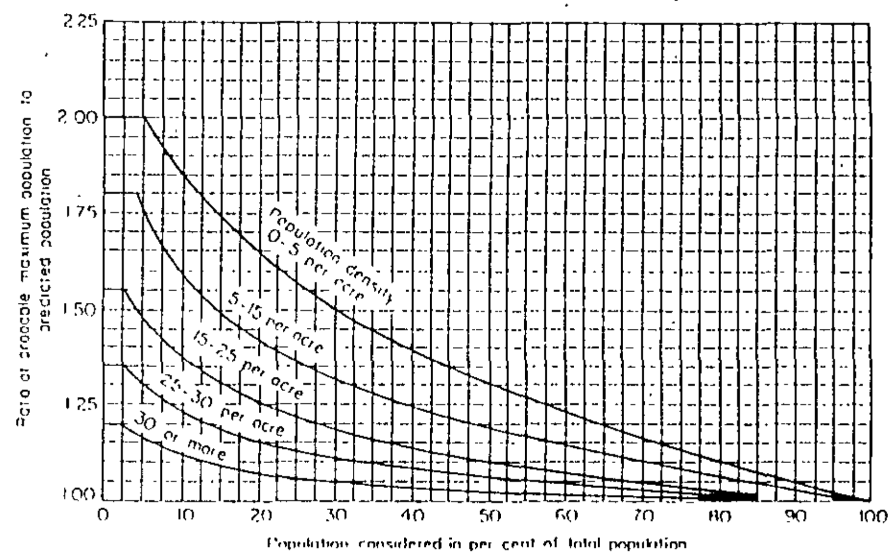


FIG. 1. Capacity factors for unequal population development.

Quantity of Domestic Sewage. The purely domestic sewage flow will generally be about 80 to 90 per cent of the domestic water consumption. The average per capita domestic water consumption varies from about 40 to 120 gpd, depending upon the character of the area and the economic status of the population. If water is supplied through meters, accurate estimates of average per capita consumption can be made. If water is supplied unmetered, estimates have to be based on the consumption rates which are known to prevail in other areas of similar character.

Quantity of Commercial Sewage. The quantity of sewage flow from commercial areas varies widely depending upon the nature of the commercial activity. Allowances made for the quantity of sewage from commercial areas in large sewer districts are commonly in terms of gallons per day per acre or gallons per capita per day. Table 1 gives the allowances which were made for flow from commercial areas in some American cities.[3] Allowances varied from 2,000 to 60,500 gpd per acre and from 15 to 500 gpd per capita. It is evident that, for any area in which commercial activity is an important factor, the estimate of sewage flow should be based on a special study of the area.

Industrial Sewage Flow. The flow of sewage from industrial establishments may be purely sanitary sewage, or it may also include water-borne industrial wastes. Estimates of the sanitary sewage are made by the procedures already considered. The quantity of industrial wastes can be determined only by special

Table I. Sewer Capacity Allowances for Commercial and Industrial Areas*

City	Year data	Commercial	Industrial
Baltimore, Md.†	1949	135 gpcd‡ (range 6,750 to 13,500 gpd per acre), resident population	7,500 gpd per acre minimum
Berkeley, Calif.	....	.....	50,000 gpd per acre
Boston, Mass.†	1949	No standard—each area specially studied	
Cincinnati, Ohio†	1949	Commercial areas not served by sanitary sewers	
Columbus, Ohio†	1946	40,000 gpd per acre; excess added to residential amount	
Cranston, R.I.†	1943	25,000 gpd per acre	
Dallas, Tex.†	1949	30,000 gpd per acre; downtown area rate added to domestic; outlying area rate added to domestic	
Grand Rapids, Mich.	..	40–50 gpcd;‡ office buildings 400–500 gpd per room, hotels 200 gpd per bed; hospitals 200–300 gpd per room; schools	250,000 gpd
Hagerstown, Md.	..	180–250 gpd per room; hotels 150 gpd per bed, hospitals 120–150 gpd per room, schools	
Las Vegas, Nev.	....	310–525 gpd per room; resort hotels 15 gpcd;‡ schools	
Los Angeles, Calif.	1948†	80–100 gpcd;‡ office buildings 450–500 gpcd;‡ hotels 800–1,000 gpd per bed; hospitals 35 gpcd;‡ schools 0.015 cfs per acre; light business district (= 9,700 gpd per acre; all commercial areas)†	0.021 cfs per acre; light industrial—individual studies for major industrial districts
Memphis, Tenn.	....	2,000 gpd per acre	2,000 gpd per acre
Milwaukee, Wis.†	1945	0.0936 cfs per acre = 60,500 gpd per acre	
New York, N.Y.†	1949	Allowances determined by special gagings	
Santa Monica, Calif.	....	0.015 cfs per acre; commercial 0.012 cfs per acre; hotels	0.021 cfs per acre
Shreveport, La.	....	3,000 gpd per acre; commercial	
Toledo, Ohio†	1946	15,000 to 30,000 gpd per acre, average to maximum allowances	
Washington, D.C., suburban district†	1949	None except in special cases	

* ASCE (WPCF) Manual, "Design and Construction of Sanitary and Storm Sewers."
† "Sewer Capacity Design Practice," by William F. Stanley and Warren J. Kaufman, *J. Boston Soc. Civil Engrs.*, October, 1953, p. 320, Table 3.
‡ Gallons per capita per day.

studies of the individual industrial activities. When large industrial waste flows are involved, the problem of collection and disposal of these wastes usually requires special engineering studies.

Quantity of Infiltration. The rate of infiltration of ground water into sewers is influenced by the size, age, and condition of the sewers; the position of the sewers with respect to the ground-water table, the character of the soil; and the amount of precipitation. The infiltration rate for any one system will vary from season to season. It is common practice to allow for infiltration of about 30,000 gpd per mile

of sewer and house connection.[3] Specifications for new work commonly allow infiltration rates of 3,500 to 5,000 gpd per mile for 8-in. pipe, 4,500 to 6,000 for 12-in. pipe, and 10,000 to 12,000 for 24-in. pipe.[4]

Flow Variations. Sewers must be designed to handle the peak flow rates to be expected at the end of the design period. It is also desirable to design them so as to minimize the problem of solids deposition during the early years of use when the flows may be much lower than the future flows. The flows vary from day to day and from hour to hour within each day. The ratio of the absolute maximum future flow rate to the initial minimum rate may vary from about 3 to 1 for large sewers serving highly developed areas to more than 20 to 1 for small sewers serving areas still under development. Figure 2, which was used for the design of sewerage for

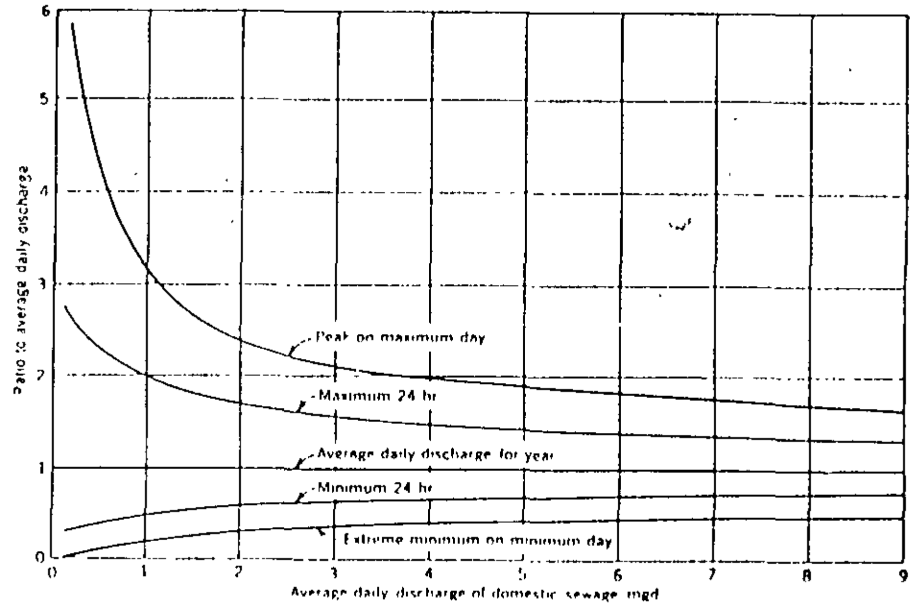


FIG. 2 Variations in flow of domestic sewage.

the Merrimack River Valley Sewerage District in Massachusetts (Massachusetts Senate Document 550 of 1947), is a good example of the magnitudes of fluctuations in flow which are commonly allowed for in design.

Examples of Sewage Flows Used for Design. Table 2 gives data on the sewage flows which have been used for the design of sanitary sewers in a number of United States cities.[3] The values reported range from 92 to 600 gpd per capita and this wide range in flow rates emphasizes the desirability of making an individual study to determine the proper design flow rates for any particular area.

Requirements of Regulatory Agencies. Some state regulatory agencies have established definite per capita flow rates to be used when detailed studies and estimates of expected flows have not been made. The recommended standards of the Great Lakes Upper Mississippi River Board of State Sanitary Engineers in regard to the design of sanitary sewers are as follows:[5]

Design Period

In general, sewer systems should be designed for the estimated ultimate tributary population, except in considering parts of the systems that can be readily increased in capacity. Similarly, consideration should be given to the maximum anticipated capacity of institutions

Table 2. Sewage Flows Used for Design *

City	Year of data	Average rate of water consumption, gpcd†	Population served, thousands	Per capita sewage flow average,‡ gpcd†	Sewer design basis,† gpcd†	Remarks
Baltimore, Md	..	160	1,300	100	135 × factor	Factor 4 to 2
Berkeley, Calif	...	76	113	60	92	
Boston, Mass	....	145	801	140	150	Flowing half full
Cleveland, Ohio§	1946	...	.....	100	....	
Cranston, R.I.§	1943	...	...	119	167	
Dallas, Tex.§	1949	...	..	150	575	Including storm water and infiltration
Des Moines, Iowa§	1949	..	.....	100	200	
Grand Rapids, Mich.	....	178	200	189.5	200	
Hagerstown, Md	...	100	38	100	250	
Jefferson County, Ala	...	102	500	100	300	
Las Vegas, Nev.	..	410	45	209	250	
Lincoln, Neb.	.	160	125	125	200	
Little Rock, Ark.	....	50	100	50	100	
Los Angeles, Calif	....	165	2,680	95	....	Single dwelling 0.004 cfs per acre; multiple dwelling 0.008 to 0.020 cfs per acre
Madison, Wis.§	1937	..	...	..	300	Maximum hourly rate
Milwaukee, Wis.§	1945	...	.	125	....	All in 12 hr—250-gpcd† rate
Memphis, Tenn	..	125	450	100	100	
Orlando, Fla	..	150	75	70	190	
Painesville, Ohio§	1947	..	..	125	600	Includes infiltration and roof water
Rapid City, S. Dak		122	40	121	125	
Rochester, N.Y.§	1946	.	..	....	250	New York State Board of Health standard
Santa Monica, Calif	...	137	75	92	92	
Shreveport, La	..	135	160	120	150	
Springfield, Mass §	1949	..	...	...	200	150 gpcd† was used on a special project
Toledo, Ohio§	1946	.	..	.....	160	
Washington, D.C., suburban districts§	1946	...	.	100	2 to 3.3 × average	

* ASCE (WPCF) Manual, "Design and Construction of Sanitary and Storm Sewers."
† Gallons per capita per day
‡ Measured or estimated domestic sewage.
§ "Sewer Capacity Design Practice," by William E. Stanley and Warren J. Kaufman, *J Boston Soc. Civil Engrs.*, October, 1953, p. 317, Table 2.

Design Factors

In determining the required capacities of sanitary sewers the following factors should be considered:

a. Maximum hourly quantity of sewage.
b. Additional maximum sewage or waste from industrial plants.
c. Ground water infiltration.

Design Basis

Per Capita Flow. New sewer systems should be designed on the basis of an average daily per capita flow of sewage of not less than 100 gallons per day. This figure is assumed to cover normal infiltration, but an additional allowance should be made where conditions are unfavorable. Generally the sewers should be designed to carry, when running full, not less than the following daily per capita contributions of sewage, exclusive of sewage or other waste from industrial plants.

Laterals and sub-main sewers-- 400 gallons.
Main, trunk and outfall sewers --250 gallons.
Interceptors: Intercepting sewers, in the case of combined sewer systems, should fulfill the above requirements for trunk sewers and have sufficient additional capacity to care for the necessary increment of storm water Normally no interceptor should be designed for less than 350% of the gauged or estimated dry weather flow.

Alternate Method. When deviations from the foregoing per capita rates are demonstrated, a brief description of the procedure used for sewer design must be included.

Table 3. Discharge Weights of Plumbing Fixtures *

Fixture or group	Weight per fixture or group in fixture units
Bathroom group:	
Flush valve water closet	8
Tank water closet	6
Bathtub (hot and cold)	3
Drinking fountain	½
Dishwasher domestic	2
Kitchen sink:	
Domestic	2
Domestic with food waste grinder	3
Lavatory (hot and cold)	2
Laundry tray (one or two compartments)	2
Shower stall, domestic	2
Showers (group), per head	3
Sink,	
Service (trap standard)	3
Service (P-trap)	2
Urinal	
Pedestal, siphon jet, blowout (flush valve)	8
Stall or wall type	4
Water closet:	
Tank operated	4
Flush valve operated	8

* From American Standard Plumbing Code, ASA A40.8-1955

† Consisting of water closet, lavatory, and bathtub or shower stall.

‡ A shower head over a bathtub does not increase the fixture value

For a continuous or semicontinuous flow into a drainage system, such as from a pump, pump ejector, air-conditioning equipment, or similar device, two fixture units shall be allowed for each gallon per minute of flow.

Fixture-unit Basis of Design. For small tributary populations and for institutions such as schools, hospitals, hotels, factories, etc., the required capacities of sanitary sewers may be estimated from the "fixture-unit flow rates" defined by the American Standard National Plumbing Code, ASA A40.8-1955. Table 3 gives the relative discharge weights in fixture units for various types of fixtures. The relationship between the probable peak discharge rate and the total number of fixture units, based on the probability studies of R. B. Hunter, is given by Fig. 3[6].

Summary. The following summary of the considerations necessary for the determination of sanitary sewer capacity is given in the ASCE Manual of Engineering Practice, No. 37 (WPCF Manual of Practice No. 9), "Design and Construction of Sanitary and Storm Sewers":

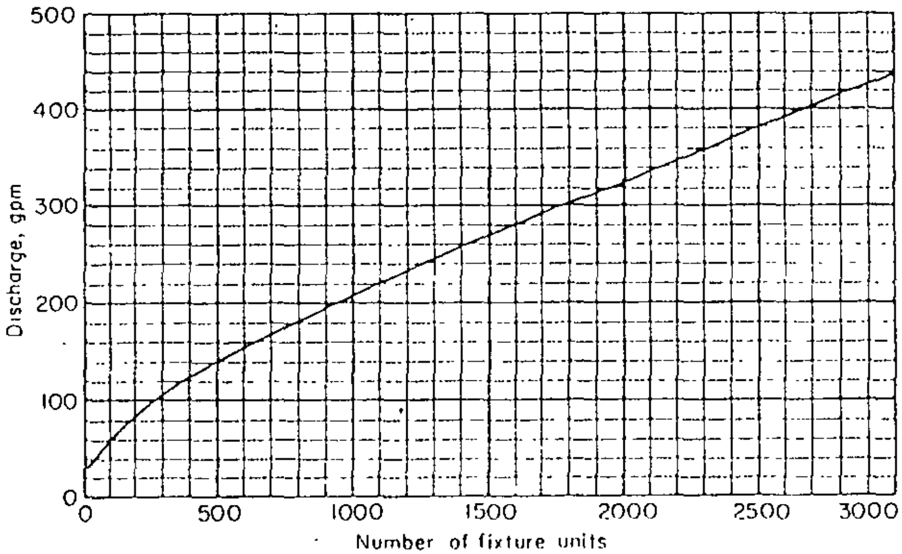


FIG. 3. Discharge from plumbing fixtures

The determination of the flow quantities for which to design a separate sanitary sewer requires consideration and determination of the following:

1. The design period during which the peak or maximum design flow is not expected to be exceeded—usually 25 to 40 or 50 yr in the future.
2. Domestic sewage contributions based upon probable future population and probable future per capita water consumption. Careful consideration is given to distribution of population and relationship of peak and minimum flow rates to average per capita sewage flows. The fixture-unit method of developing peak rates should be employed for small populations.
3. Commercial area contributions from stores, hotels, offices, and other businesses which are sometimes assumed to be amply cared for in the peak allowance for per capita sewage flows in small communities. Per-acre allowances based upon actual records or successful design quantities of record for comparable commercial areas are the most reasonable approach for larger communities.
4. Industrial wastes which include estimated domestic sewage from known populations per shift, estimated or gaged allowances per acre for industry as a whole, and actual or estimated flow rates from plants with large process wastes which can be permitted in sanitary sewers.

5. Institutional wastes, usually almost entirely domestic sewage. Peak and minimum design flow rates from persons in the institution are multiplied by the same basic per capita rates as are used by S. A. Greeley and W. A. Stanley.[2]
6. Air-conditioning and industrial cooling waters if permitted to be discharged to the sanitary sewers—assume 1½ to 2 gal per ton of non-water-conserving cooling units. There should be an enforced ordinance prohibiting the discharge of spent cooling waters into separate sanitary sewers.
7. Storm-water contributions to sanitary sewers. These should be prohibited, but the designer must recognize that some such storm and surface water does get into separate sanitary sewers and a judgment allowance therefore must be made.
8. Infiltration through defective joints. This needs as careful an evaluation as can be made from thought given to the several factors involved. Design allowances should be larger (under some circumstances very much larger) than those stipulated in construction specifications where acceptance tests are made very soon after construction. Underevaluation of infiltration is often the principal reason why some sewers have become overloaded.

The relative emphasis given to each of the foregoing factors varies among designers. Some have set up single values of peak design flow rates for the various classifications of tributary area, thereby integrating all these items. It is recommended, however, that actual maximum and minimum peak design flows be developed step by step, giving as thorough as possible consideration to each of the component items which influence design values.

QUANTITY OF STORM WATER

The Rational Method. The rational method is the most commonly used procedure for the computation of the rates of storm-water runoff for storm-sewer design. The rate of runoff Q is given by the equation

$$Q = CiA$$

in which A is the size of the drainage area, i is the average intensity of rainfall for a duration equal to the time of concentration of the area, and C is a runoff coefficient whose value depends principally upon the character of the area. For a steady rainfall rate of 1.0 in./hr the total precipitation deposited over an area of 1 acre would be at the rate of 1.008 cfs or, for practical purposes, 1.0 cfs. Therefore, if A is expressed in acres, i in inches per hour, and Q in cubic feet per second, C may be interpreted as a dimensionless coefficient which expresses the ratio of the runoff rate to the rainfall rate.

The assumption behind the rational method is that the runoff rate for a given rainfall intensity will increase and reach its maximum when the duration of the rainfall reaches the time of concentration of the area (the time required for the runoff to flow from the remotest point of the area to the point where Q is being measured). By this assumption, the maximum runoff rate Q which can be expected to occur with any given frequency will be produced by a storm having the maximum average rainfall intensity corresponding to the given frequency and duration. The application of the method requires knowledge of the rainfall intensity-duration-frequency characteristics for the locality. Although the assumptions are not strictly in accord with the mechanics of the runoff process, the rational method has proved to be a practical procedure for storm-drain design because the accumulated experience has resulted in practicable values for the runoff coefficient C.

Time of Concentration. The time of concentration is the time required for the runoff to flow from the remotest point of the drainage area to the point under design. This is the minimum time necessary to permit the entire area to contribute to the flow at the point. The time of concentration consists of the inlet time, or time required for the runoff at the upper end of the area to reach the nearest inlet, plus the time of flow in the sewer from this inlet to the point being considered.

Inlet time will vary with the nature of the surface, the slopes, the nature of the established drainage channels such as street gutters, and the antecedent conditions. Because the inlet time is small, it is commonly chosen on the basis of experience.

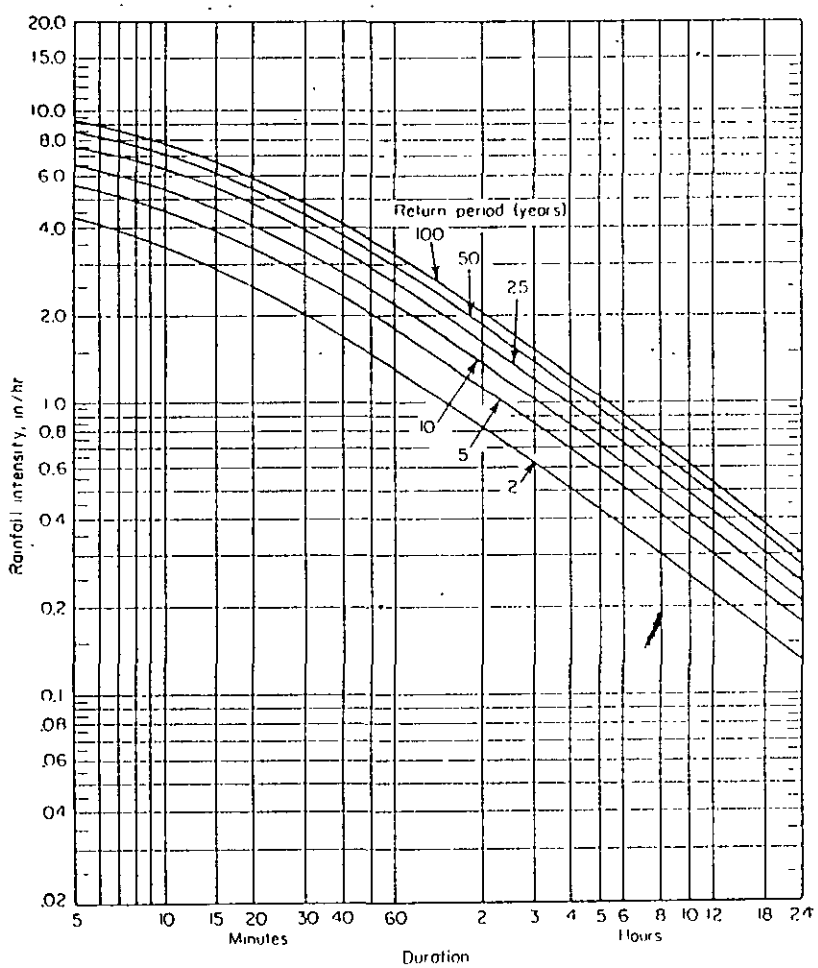


FIG. 4. Rainfall intensity-duration-frequency curves, New York, N.Y., 1903–1951. NOTE: Frequency analysis by method of extreme values, after Gumbel.

In densely developed areas with a high percentage of paved surfaces and closely spaced inlets, an inlet time as low as 5 min may be assumed. In moderately developed urban areas with flat slopes the inlet time may be from 10 to 15 min. In flat residential areas having a relatively low percentage of paved surface, the inlet time may be as high as 30 min. It is possible to make estimates of the inlet time by calculating the time of flow over the various types of surfaces, but such estimates can rarely be made with a high degree of accuracy.

The time of flow in the sewer is computed from the hydraulic properties of the sewer, the common practice being to use the average flowing-full velocity computed for the prevailing slope.

Rainfall Frequency. It is usually prohibitive, on the basis of cost, to construct storm sewers capable of handling the largest conceivable storms. Current practice is to use storm rainfalls having an average expected frequency of once every 3 to 10 years for the design of storm sewers in residential areas and storms of 10 to 30 years for commercial and high-value districts.

Rainfall Intensity-Duration-Frequency Relationships. The rainfall characteristics which must be known for storm-sewer design are presented in a very concise manner by the intensity-duration-frequency curves, which can be prepared from a long record of precipitation at a given station. Figure 4 shows such a set of curves prepared by the U.S. Weather Bureau from the precipitation recorded at New York City from 1903 to 1951.[7] This figure shows, for example, that an average intensity of 2.2 in./hr for a duration of 1 hr will be equaled or exceeded, on the average, once every 10 years. Similar data for many other localities have been compiled and published by the U.S. Weather Bureau.[7, 8] Figures 5–8 show the geographical distribution of certain rainfall characteristics for the area of the United States east of the 105th meridian.[7] For the region west of the 105th meridian, the local variations in rainfall characteristics make it impracticable to present them in the form of Figs. 5 to 8. For data covering the western portion of the United States, the reader is referred to *Weather Bureau Technical Paper* 28.[9]

Runoff Coefficient. The runoff coefficient C in the rational method is the variable which is least susceptible to precise determination. Whereas the use of the runoff coefficient implies that there is a constant ratio of runoff to rainfall, the actual ratio for a given area will depend upon the condition of the area at the time of occurrence of the storm and will increase with the duration of the storm. A more logical procedure than the rational method for storm-drain design would be to subtract the rainfall losses due to infiltration and retention in surface depressions and to distribute the remainder as an actual hydrograph of runoff.[10] However, because of the great variability in the time distribution of the rainfall itself as well as the difficulty in estimating the quantities of infiltration and surface depression storage, most engineers still prefer to estimate a value of the runoff coefficient C. A common practice is the use of average coefficients for various types of districts, the coefficients being assumed to be constant throughout the storm duration. The range of values reported to be in common use is as follows:[3]

Type of area	Runoff coefficient
Business:	
Downtown areas	0.70–0.95
Neighborhood areas	0.50–0.70
Residential:	
Single-family areas	0.30–0.50
Multiunits, detached	0.40–0.60
Multiunits, attached	0.60–0.75
Residential (suburban)	0.25–0.40
Apartment dwelling areas	0.50–0.70
Industrial:	
Light areas	0.50–0.80
Heavy areas	0.60–0.90
Parks, cemeteries	0.10–0.25
Playgrounds	0.20–0.35
Railroad yard areas	0.20–0.40
Unimproved areas	0.10–0.30

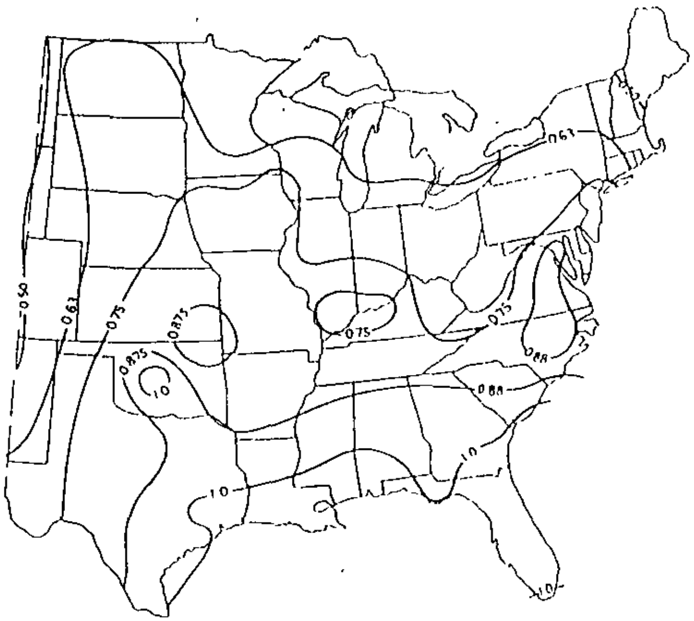


FIG. 5. Rainfall intensities in inches for 2-year frequency and 15-min duration.

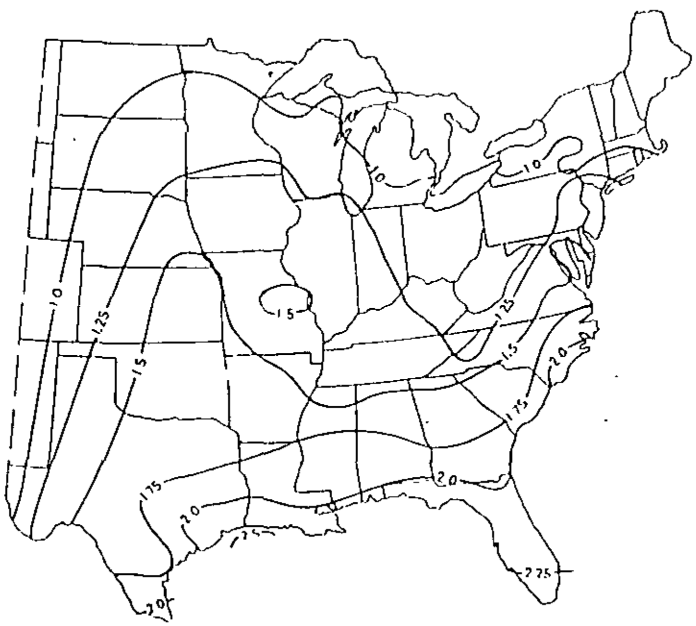


FIG. 6. Rainfall intensities in inches for 2-year frequency and 60-min duration.

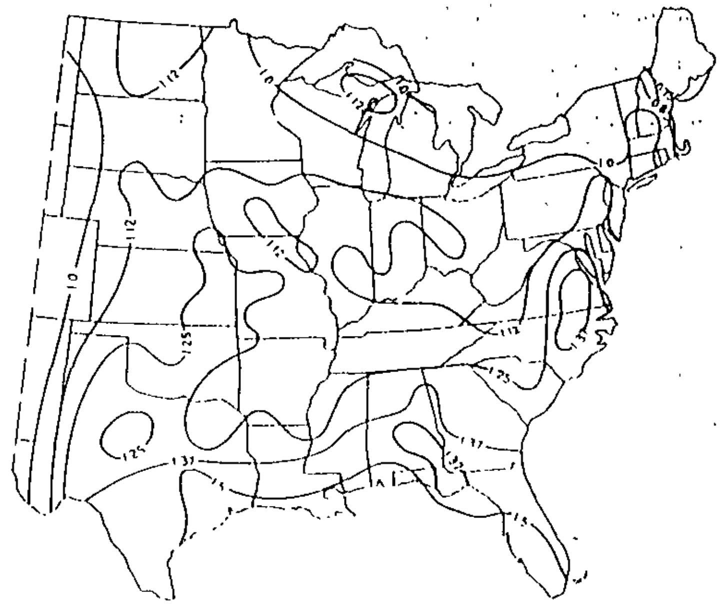


FIG. 7. Rainfall intensities in inches for 10-year frequency and 15-min duration.

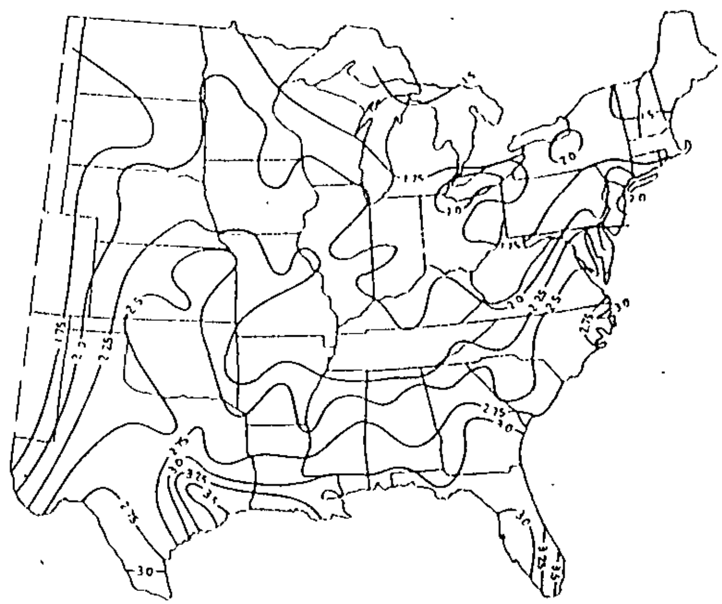


FIG. 8. Rainfall intensities in inches for 10-year frequency and 60-min duration.

For specific small areas it is more logical to relate the value of C to the actual type of surface. Coefficients commonly used are:[3]

Character of surface	Runoff coefficient
Streets:	
Asphaltic	0.70-0.95
Concrete	0.80-0.95
Brick	0.70-0.85
Drives and walks	0.75-0.85
Roofs	0.75-0.95
Lawns, sandy soil:	
Flat, 2%	0.05-0.10
Average, 2 to 7%	0.10-0.15
Steep, 7%	0.15-0.20
Lawns, heavy soil	
Flat, 2%	0.13-0.17
Average, 2 to 7%	0.18-0.22
Steep, 7%	0.25-0.35

When an area is made up of different types of surfaces, a common procedure is to use a weighted-average coefficient.

The coefficients in the previous tabulations are designed for use for storms of 5- to 10-year frequencies. For less frequent, higher intensity storms, the coefficients should be higher, because the infiltration and surface retention will be smaller proportions of the total precipitation. Likewise, for more frequent, lower intensity storms, the coefficients should be lower than indicated in the tables.

HYDRAULICS OF SEWERS

Although they are usually placed underground, most sewers are designed to flow with a free water surface. An advantage of the free-flow condition is that the depth will vary with the rate of flow in such a way as to keep the velocity reasonably high even at flow rates which are small in comparison with the full capacity of the pipe. This helps to maintain velocities sufficient to prevent the deposition of solids over a wide proportion of the flows which are likely to be encountered.

The capacity of a sewer pipe should be sufficient to carry the peak flow rate to be anticipated at the end of the design period, and the slope should be sufficient to provide for self-cleansing velocities during the early years of use. It is common practice to design sanitary sewers with slopes sufficient to provide for velocities of 2 fps when flowing full. Experience shows that with such slopes trouble from deposits is seldom encountered.

Although the flow in sewers is seldom steady or uniform, it is impracticable in most cases to take this into account, and each section of the sewer is usually designed with the assumption that the flow is steady and uniform.

Flow Formulas. Despite its complexity, Kutter's formula has been widely used in the past for the solution of problems involving open-channel flow in sewers because of the availability of charts and diagrams which facilitate its solution. The Manning equation is now being widely used in place of the Kutter formula.

The Kutter equation is

$$V = \left[\frac{1.81/n + 41.67 + 0.0028/S}{1 + n/\sqrt{r}\,(41.67 + 0.0028/S)}\right]\sqrt{rS}$$

in which V = mean velocity, fps.
r = hydraulic radius, ft
S = slope of energy gradient
n = coefficient of roughness

8

Manning's equation is

$$V = (1.486/n)r^{2/3}S^{1/2}$$

in which the nomenclature is the same as for the Kutter equation.

The roughness coefficient n varies from 0.010 for smooth surfaces to as high as 0.10 for rough natural channels and has the same numerical value in each of the above equations. It is common practice to use values of Kutter's or Manning's n of 0.013 for sewer design. This value makes some allowance for the future condition of the pipe as well as disturbances in the flow resulting from rough joints, interior coatings of grease or other matter, and eddying due to changes in pipe size, junctions, etc. Figure 9 is a diagram for the solution of the Manning equation applied to circular pipes flowing full, with n equal to 0.013. Experiments have indicated that the value of n is not the same at all depths and that maximum value is found to occur at a depth equal to about three-tenths of the diameter.

Pipes Flowing Partly Full. Figure 10, which is taken from the ASCE (WPCF) Manual previously referred to, gives the hydraulic elements of circular pipes flowing partly full. This figure shows the ratios of the values of the various elements to the values for the flowing-full condition. The cross-sectional area and the hydraulic radius are purely geometric functions and hence independent of n. The velocity and discharge for any particular ratio of depth to diameter depend upon whether n is assumed to be constant or variable with the depth. Velocity and discharge curves computed from both assumptions are shown. The variation in the value of n is based on extensive experiments by Wilcox and by Yarnell and Woodward.[11,12]

The use of Fig. 10 is illustrated by the following example:

Example. A 36-in. sewer pipe (n = 0.013) is laid on a slope of 2.00 ft per 1,000 ft. Find the depth of flow and the velocity when the flow rate is 20.0 cfs. Assume that n varies with the depth of flow.

Solution. From Fig. 9, the capacity of the pipe when flowing full is 30.0 cfs and the velocity is 4.23 fps. Then q/Q = 20.0/30.0 = 0.677. From Fig. 10, d/D = 0.67 and v/V = 0.93. Then depth = 0.67 × 3.0 = 2.00 ft and velocity = 0.93 × 4.23 = 3.93 fps.

Figure 10 also shows the relative velocities required to obtain equal cleansing of the pipe at all depths of flow. This is based on T. R. Camp's analyses of the theoretical work done by Shields on the movement of granular materials in open channels.[13,14] The diagram indicates that, if a sewer has self-cleansing velocities under flowing-full conditions, the velocity will also be self-cleansing for all flow conditions at depths greater than one-half the diameter. As previously stated, this is an important reason for designing sewers to flow as open channels.

Alternate Stages of Flow. The specific energy of the flow at any section in an open channel is defined as the energy of the flow expressed in feet (foot-pounds per pound) measured above the bottom of the channel. Figure 11 represents the flow profile at a section of a channel flowing with a free surface. The specific energy for a point in a stream line at a distance y above the bottom of the channel is given by the expression

$$H = y + h + v^2/2g = d + v^2/2g = d + Q^2/2gA^2$$

where H is the specific energy head, h is the pressure head above the stream line, v is the velocity at the point, Q is the flow rate, and A is the cross-sectional area. Since the velocity is a function of y, the specific energy is also a function of y. However, the above equation can be considered, with very little error, as being representative of the flow as a whole provided that v is taken as the average velocity for the cross section.

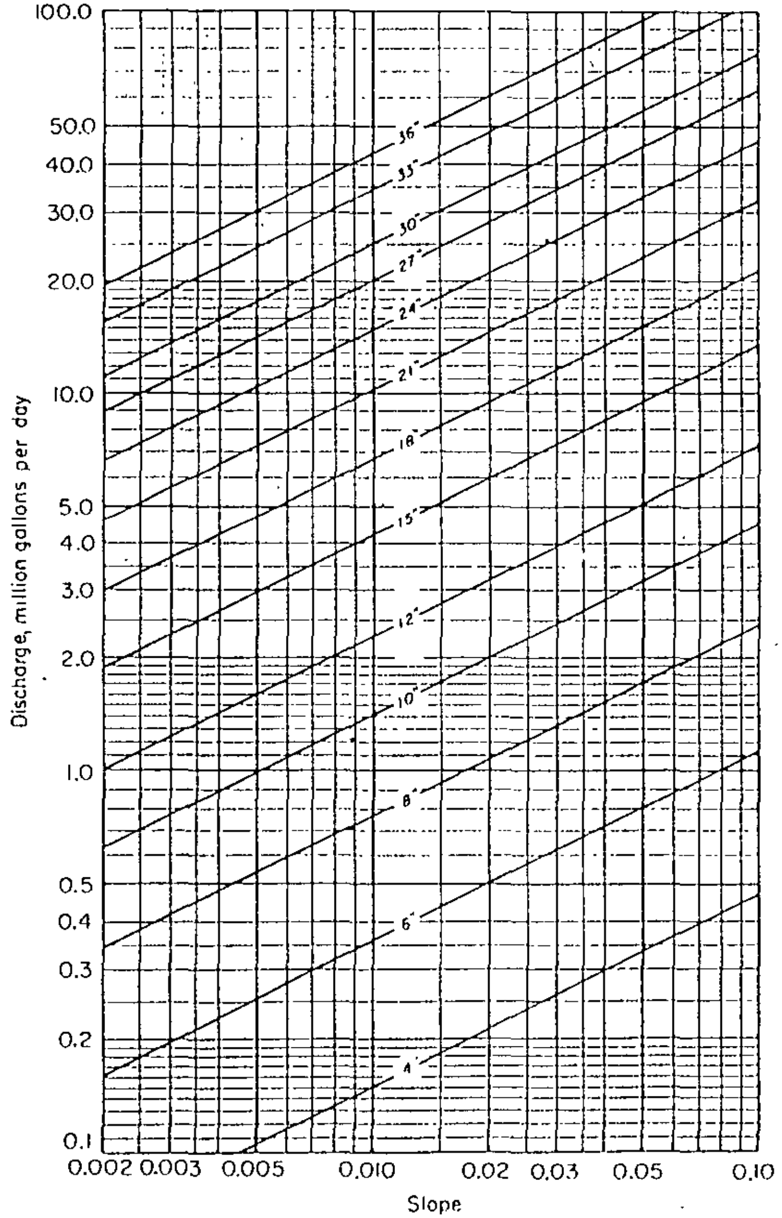


FIG. 9. Discharge of circular pipes (running full) based on the Manning formula

$$Q = A(1.486/n)r^{2/3}S^{1/2}$$

where Q = flow rate, A = cross-sectional area, r = hydraulic radius, S = slope of energy gradient, n = coefficient of roughness.

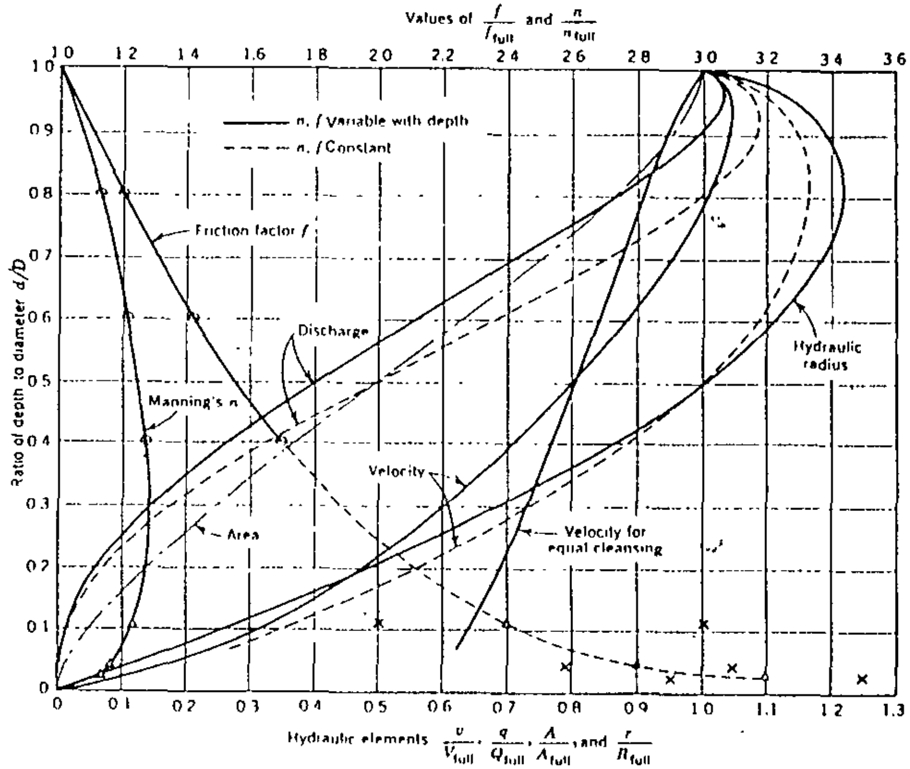


FIG. 10 Hydraulic elements of circular sewers.

In Fig. 11, the slope of the channel bottom is designated as S_0 and the slope of the energy gradient as S. For the general case of nonuniform flow (where the depth is increasing or decreasing in the direction of flow) the values of S and S_0 will be different; for uniform flow they will have the same value. The Manning equation can be applied to the conditions at any section provided that S in the equation is taken to be the slope of the energy gradient at the section.

It is evident that the cross-sectional area of the flow in the specific-energy equation is a function of the shape of the section and the depth. For a circular pipe, A is a complicated function of the depth d and the diameter D. Figure 12, which was prepared by Thomas R. Camp, is a graphical solution of the specific-energy equation for circular pipes.[15] It is noted from Fig. 12 that, for given values of Q, D, and H, there are two possible values of d which will satisfy the equation. The larger of the two values is designated as "upper stage flow," "tranquil flow," or "subcritical flow"; the lower value is designated as "lower stage flow," "shooting

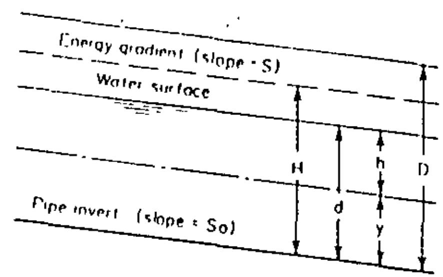


FIG. 11. Open-channel flow.

flow," or "supercritical flow." The "critical depth" is the depth at which the value of the function $Q^2/2gHD^4$ is a maximum or at which the value of H is a minimum for given values of Q and D. It is important to note that the relationships given in Fig. 12 are purely algebraic and are in no way related to the basic relationship between Q and S as given by the Kutter or Manning equations

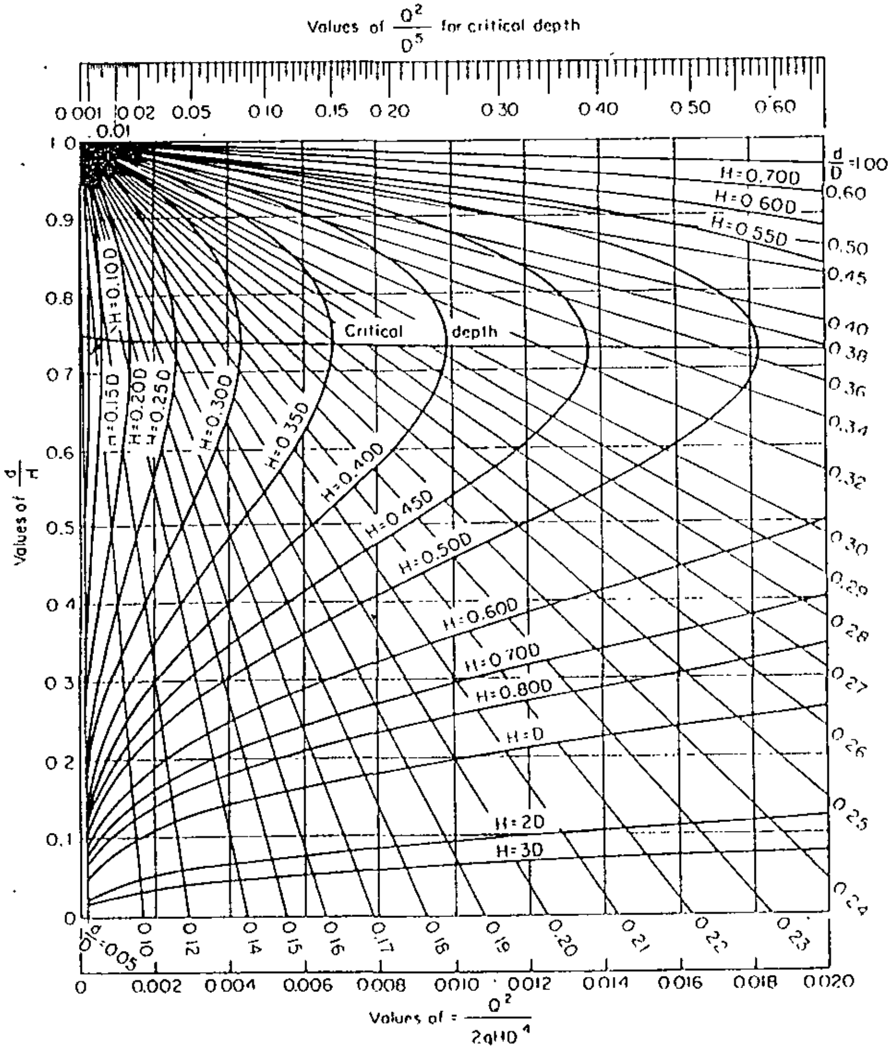


FIG. 12a. Energy relationships for free surface flow in pipes. $Q^2/2gHD^4$ from 0.0 to 0.02.

If a channel is long enough, or if the downstream conditions are just right, the flow in a channel may be uniform. Under these conditions, the depth will be constant along the channel and the values of S_0, S, and the slope of the water surface will be equal. The depth of flow for uniform flow conditions may be designated as the normal depth d_n. This is the depth of flow which makes the rate of loss of energy S equal to the bottom slope S_0. Perfectly uniform flow rarely occurs

because the depth at any cross section is dependent not only upon S_0 but also upon the distance of the section from the channel outlet or from the nearest downstream or upstream channel junction, at which point the depth may be controlled by factors which are independent of S_0. A value of d_n can be computed for any channel by the

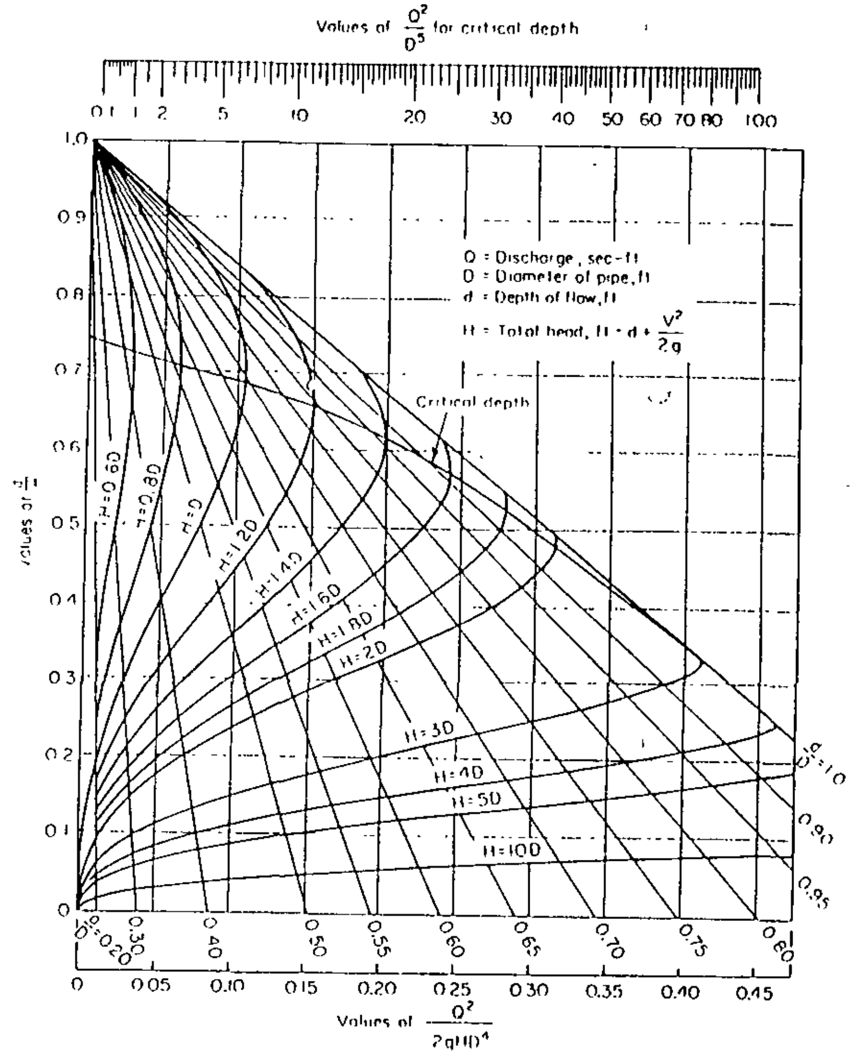


FIG. 12b. Energy relationships for free surface flow in pipes. $Q^2/2gHD^4$ from 0.0 to 0.50.

Manning equation by assuming that S is equal to S_0. However, whether or not the flow will ever occur at the depth d_n in any stretch of a channel will depend upon the hydraulic conditions downstream or upstream of the stretch. A value of S_0 which makes d_n greater than the critical depth d_c may be designated as a "mild slope," and a value which produces a d_n less than the critical depth may be designated as a "steep slope." The slope S_0 which makes d_n equal to the critical depth may be designated as the critical slope S_c.

The significance of the critical depth may be illustrated by reference to Fig. 13. The flow conditions for each of the cases shown are explained briefly below:

1. A channel with a mild slope discharges freely into the atmosphere. Critical depth will occur at the outlet. The depth will increase at successive sections upstream until the normal depth is reached, beyond which the flow will be uniform.

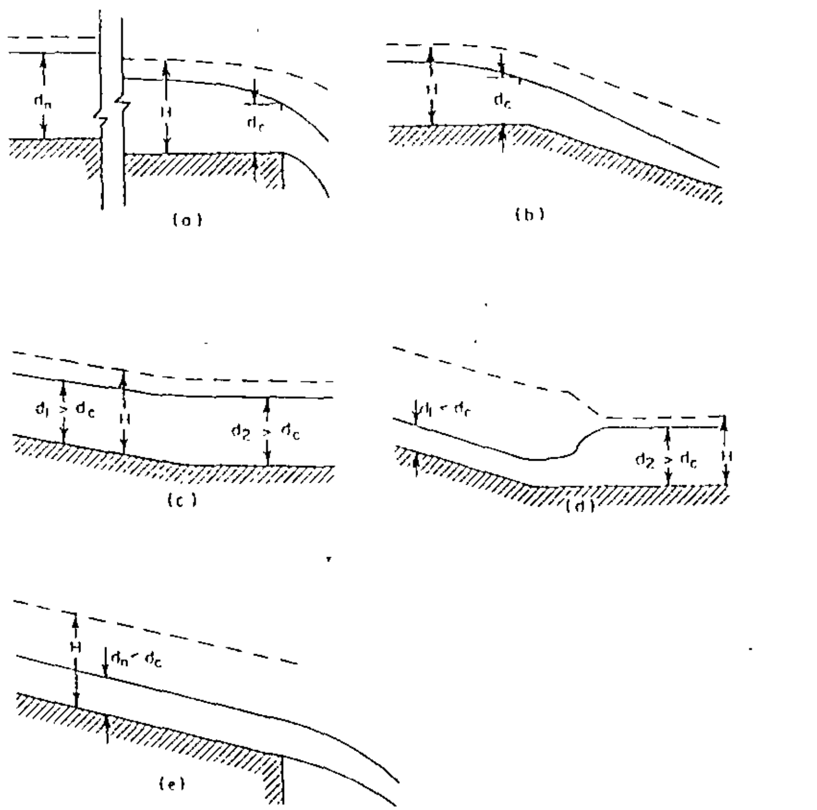


FIG. 13 Examples of nonuniform flow profiles. (*a*) Free discharge from a mild slope. (*b*) Discharge from a mild to a steep slope. (*c*) Discharge from one mild slope to another. (*d*) Discharge from a steep slope to a mild slope. (*e*) Free discharge from a steep slope.

In many cases the length of the channel will be much less than the distance required to develop normal depth. The significance of this is that for a short stretch of pipe, its actual capacity to carry flow without surcharge may be much greater than would be calculated by assuming uniform flow with S equal to S_0.

2. A channel changes slope from mild to steep. The conditions upstream of the junction will be the same as for paragraph 1. The depth downstream of the junction will decrease and approach d_n.

3. A channel changes slope from a mild slope to another mild slope. Upstream of the junction there will be a gradual decrease in the depth from section to section.

4. A channel changes slope from steep to mild. In this case, the change from the upstream supercritical flow to the downstream subcritical flow will take place suddenly in a "hydraulic jump." The position of the jump may be either upstream or downstream of the junction, depending upon the relative values of the various parameters which control the flow pattern.

5. A steep slope discharges into the atmosphere. In this case the flow will be at the normal supercritical flow, provided that the upstream control has permitted normal depth to be developed (see case 2).

The foregoing discussion on nonuniform flow is presented to show the reader the importance of understanding these principles, particularly when dealing with small systems where few pipes may be involved. For detailed presentation of nonuniform flow, the reader is referred to textbooks on the subject.[16–18]

Hydraulics of Sewer Transitions. In extensive sewer systems, most of the pipes will have mild slopes and the flows will be subcritical. Extra energy losses occur at all transitions where changes occur in size, slope, or direction of the pipe and at junctions where several pipes come together. If the transitions are properly designed to allow for these energy losses, the condition of uniform flow may be approximated in the individual lines and the flows will never be at depths greater than the depths computed on the assumption of uniform flow. However, if the transitions are not properly designed, pipes may at times flow at depths greater than the computed depths and surcharge may occur under peak flow conditions. The hydraulic principles involved in transition design are illustrated in Fig. 14, which shows a transition where a pipe flows into a larger pipe laid on a flatter grade. Due to the turbulence created by the flow expansion there will be a head loss h_t. In order to prevent the upstream pipe from flowing at a depth greater than its normal depth d_1, the relative elevations of the pipes must be such that the energy gradient of the downstream pipe is lower than that of the upstream pipe by the amount of h_t. This requires that the invert of the downstream be placed below that of the upstream pipe by the amount h_{ID}. The relationship between the various vertical dimensions is given by the equation

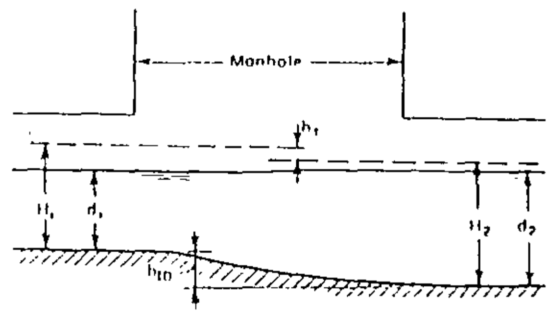


FIG. 14. Flow profile at junction.

$$h_{ID} = H_2 - H_1 + h_t$$

The equation assumes that the loss is concentrated at the center of the transition. As actually constructed, the transition will take place within a manhole, and the channel section within the manhole is made to provide for a gradual transition between the two pipes. If the computed invert drop h_{ID} is negative, it is usually taken as zero and the pipe inverts are placed at the same elevation.

The energy loss h_t is usually small, but it can assume fairly high values when high velocities are involved. Data on the magnitudes of h_t are scarce, but such data as are available indicate that h_t can be represented as a fraction of the change in velocity heads in accordance with the equation

$$h_t = K\Delta(v^2/2g)$$

Based on studies of open-channel transitions for subcritical flow by Julian Hinds, the values of K for smooth transitions might be taken as low as 0.10 for increasing velocity transitions and 0.20 for decreasing velocity transitions.[19] Increased transition losses occur when a sewer line changes direction and at junctions where one or more branch sewers join a main sewer. Reliable information on the transition head losses in such cases is almost entirely lacking. The hydraulic design of junctions may be considered as the design of two or more transitions, one for each path of flow. The exit sewer is common to all paths, and its invert must be placed at the lowest computed elevation. Because of the lack of information on the transition losses, allowances are usually made in accordance with the judgment of the designer. An arbitrary procedure which is commonly adopted is to allow about twice as much loss along flow paths in junctions as compared with the allowances for simple transitions involving the same velocities.

SEWER PIPE

Pipe Materials. The materials, listed alphabetically, of which street sewer pipes are most commonly constructed are asbestos cement, cast iron, concrete, and vitrified clay. Cast iron may be used for pressure sewers, for piping in and around buildings, and where its use may be indicated by structural requirements. The other materials are the most commonly used for sewers flowing under the usual free-surface conditions. Bituminized-fiber pipes are commonly used in collecting storm water from building downspouts, for foundation drains, and in leaching systems for residential sewage disposal. Plastic pipe may be used for the conveyance of highly acidic industrial wastes.

Asbestos-cement Sewer Pipe. Asbestos-cement sewer pipe is available in the sizes and classes given in the following table:

Class	Nominal size, diameter, in										
1500	6	8	10	12	14	16					
2400	6	8	10	12	14	16	18	20	24		
3300	6	8	10	12	14	16	18	20	24	30	
4000	. .	. .	10	12	14	16	18	20	24	30	36
5000	. .	. .	10	12	14	16	18	20	24	30	36

Standard lengths are 10 and 13 ft for the 6- and 8-in.-diameter pipe and 13 ft for the larger diameter pipe. The class designations refer to the crushing strengths as determined by the three-edge bearing method of testing. Joints are made with rubber rings or gaskets and couplings of the same material as the pipe. Advantages claimed for asbestos-cement pipe are ease of handling due to light weight, tight joints which resist infiltration and exfiltration as well as root penetration, and resistance to corrosion in most soil conditions. Corrosion may result from acid sewage and in highly acidic or highly sulfate-alkaline soils.

Standard specifications covering asbestos-cement sewer pipe are:

1. Federal Specification SS-P-3316, Pipe, Asbestos-cement, Sewer, Nonpressure
2. Tentative Specifications and Methods of Test for Asbestos-cement Nonpressure Sewer Pipe, ASTM Designation C428

Table 4. Concrete Sewer Pipe (ASTM C-14-59)

Internal diam, in	Min laying length, ft	Standard pipe		Extra-strength pipe	
		Min barrel thickness, in.	Min strength (sand bearing), lb/lin ft	Min barrel thickness, in.	Min strength (sand bearing), lb/lin ft
4	2½	⅝	1,500	¾	3,000
6	2½	⅝	1,650	¾	3,000
8	2½	¾	1,950	⅞	3,000
10	3	⅞	2,100	1	3,000
12	3	1	2,250	1⅜	3,375
15	3	1¼	2,620	1⅝	4,175
18	3	1½	3,000	2	4,950
21	3	1¾	3,300	2¼	5,775
24	3	2⅛	3,600	3	5,000

Concrete Sewer Pipe. Unreinforced concrete sewer pipe is available in standard sizes from 4 to 24 in. in diameter and in two strength classes. Standard specifications covering unreinforced concrete pipe intended for use in the conveyance of sewage, industrial wastes, and storm water are ASTM Designation C14. Table 4 gives the principal dimensional and strength requirements of these specifications.

Reinforced concrete sewer pipe, in sizes from 12 to 108 in. ID, are covered by ASTM Specifications C76. Pipe manufactured under these specifications are of five strength classifications. Table 5 gives the available sizes and strength requirements.

A number of different joint types are available depending upon the degree of watertightness required. The concrete pipe industry furnishes all the necessary bends, wyes, tees, and specials necessary for any sanitary sewerage project, the usual procedure being to make each such piece of pipe in accordance with the requirements of the project.[20]

Advantages of concrete pipe are the wide ranges in sizes, laying lengths, and strengths. A disadvantage of concrete pipe for sewers is that it is subject to corrosion under acid conditions. If flow velocities are insufficient to prevent the deposition of organic solids, septic conditions may result. Hydrogen sulfide gas produced by the anaerobic decomposition of organic matter becomes oxidized to produce sulfuric acid, which damages the pipe. This condition can usually be prevented by designing the sewers so that self-cleansing velocities will occur most of the time. Protective linings can be used to prevent corrosion where the sewage may be excessively acid.

Table 5. Reinforced-concrete Pipe (ASTM C76-59T)

Class	Size range, diam, in.	Sand-bearing strength, lb/ft per ft diam	
		To produce 0.01-in. crack	Ultimate
I	60-108	1,200	1,800
II	12-108	1,500	2,250
III	12-108	2,025	3,000
IV	12-84	3,000	4,500
V	12-72	4,500	5,625

Vitrified-clay Sewer Pipe. Vitrified-clay pipe is manufactured in standard sizes from 4 to 36 in. in diameter and in two strength classifications. ASTM Specifications C13 and C200 cover "standard-strength" and "extra-strength" clay pipe. The crushing strengths and dimensions are given in Tables 6 to 8.

A wide variety of vitrified clay fittings is available as shown by Fig. 15. Detailed dimensions of tee branches are given in Tables 9 to 12. Dimensions of the other types of fittings are given in the "Clay Pipe Engineering Manual."[21]

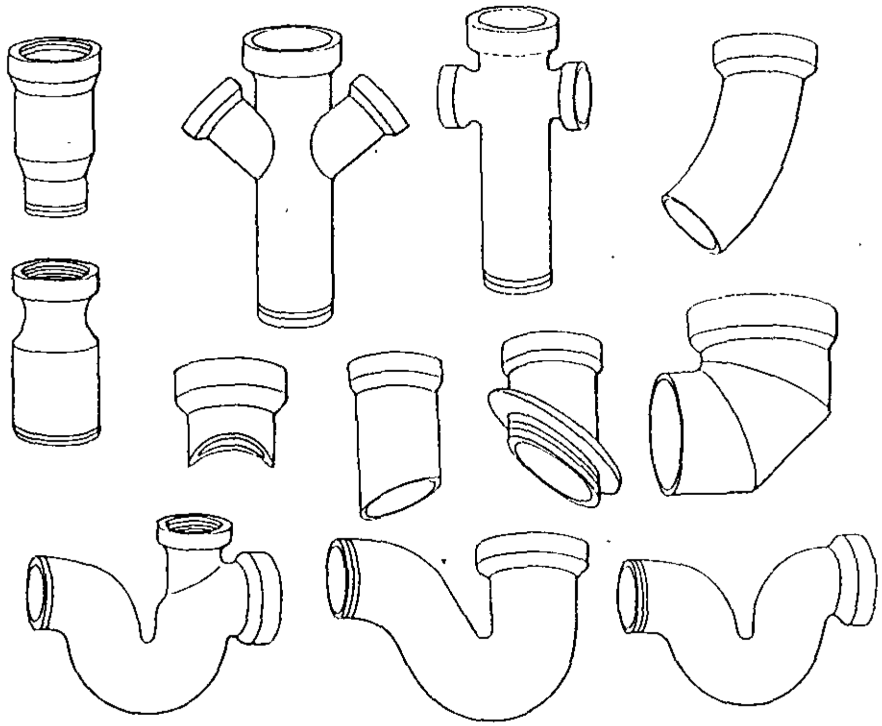


FIG. 15. Clay sewer pipe special fittings.

Vitrified-clay pipe is resistant to corrosion from most acids, making it advantageous when handling septic sewage or wastes with high acid content. Joints are commonly made up with bituminous compounds, of which there are many different types available. Recent developmental work by clay-pipe manufacturers has resulted in the marketing of joints employing resilient plastic materials which limit joint leakage under deflection and high ground-water conditions. Three types of these joints are covered under ASTM Specification C425, Vitrified Clay Pipe Joints Using Materials Having Resilient Properties.

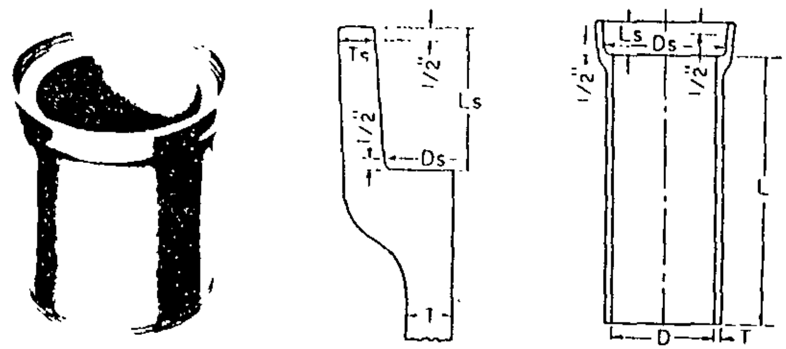
APPURTENANCES AND SPECIAL STRUCTURES

Essential to all sewerage systems are the appurtenant structures such as service connections, manholes, junction chambers, storm-water inlets, diversion chambers, etc. The design of such structures is not covered in detail in this chapter, but typical designs for the most commonly used appurtenances will be presented briefly.

Table 6. Crushing Strengths for Clay Sewer Pipe

Size, in.	Crushing strength, min, lb/lin ft			
	Standard-strength		Extra-strength	
	Three-edge bearing	Sand bearing	Three-edge bearing	Sand bearing
4	1,000	1,500		
6	1,100	1,650	2,000	3,000
8	1,300	1,950	2,000	3,000
10	1,400	2,100	2,000	3,000
12	1,500	2,250	2,250	3,375
15	1,750	2,625	2,750	4,125
18	2,000	3,000	3,300	4,950
21	2,200	3,300	3,850	5,775
24	2,400	3,600	4,400	6,600
27	2,750	4,125	4,700	7,050
30	3,200	4,800	5,000	7,500
33	3,500	5,250	5,500	8,250
36	3,900	5,850	6,000	9,000

Table 7. Standard-strength Vitrified-clay Pipe Conforming to ASTM Specifications C13



Size, in.	Laying length L		Max difference in length of two opposite sides, in	Outside diameter of barrel, in.		Inside diameter of socket at 1/2 in. above base, in. DS
	Min*	Limit of minus variation, in. per ft of length†		Min	Max	Min
4	2	1/4	5/16	4 7/8	5 1/8	5 3/4
6	2	1/4	3/8	7 1/16	7 9/16	8 3/16
8	2	1/4	7/16	9 1/4	9 3/4	10 5/8
10	2	1/4	7/16	11 1/8	12	12 3/4
12	2	1/4	7/16	13 3/4	14 5/16	15 1/8
15	3	1/4	1/2	17 3/16	17 13/16	18 5/8
18	3	1/4	1/2	20 5/8	21 7/16	22 1/4
21	3	1/4	9/16	24 1/8	25	25 5/8
24	3	3/8	9/16	27 1/8	28 3/8	29 3/8
27	3	3/8	9/16	31	32 1/8	33
30	3	3/8	5/8	34 3/8	35 5/8	36 1/8
33	3	3/8	5/8	37 5/8	38 15/16	39 7/8
36	3	3/8	11/16	40 3/4	42 1/4	43 1/4

Table 7. (Continued)

Size, in.	Depth of socket, in. *LS*		Thickness of barrel, in. *T*		Thickness of socket at ½ in. from outer end, in. *TS*	
	Nominal	Min	Nominal	Min	Nominal	Min
4	1 3/4	1 1/2	1/2	7/16	7/16	3/8
6	2 1/4	2	5/8	9/16	1/2	7/16
8	2 1/2	2 1/4	3/4	11/16	9/16	1/2
10	2 5/8	2 3/8	7/8	13/16	5/8	9/16
12	2 3/4	2 1/2	1	15/16	3/4	11/16
15	2 7/8	2 5/8	1 1/4	1 1/8	15/16	7/8
18	3	2 3/4	1 1/2	1 3/8	1 1/8	1 1/16
21	3 1/4	3	1 3/4	1 5/8	1 5/16	1 3/16
24	3 3/8	3 1/8	2	1 7/8	1 1/2	1 3/8
27	3 1/2	3 1/4	2 1/4	2 1/8	1 11/16	1 9/16
30	3 5/8	3 3/8	2 1/2	2 3/8	1 7/8	1 3/4
33	3 3/4	3 1/2	2 5/8	2 1/2	2	1 13/16
36	4	3 3/4	2 3/4	2 5/8	2 1/16	1 7/8

* There shall be no maximum length. Shorter lengths may be used for closures and specials.
† There is no limit for plus variation.

Building Service Connections. Figure 16 shows typical details of service connections to a sanitary sewer laid in a relatively shallow trench; Fig. 17 shows a typical connection to a deep sewer. It is noted that the connection shown in Fig. 16 makes use of either a wye branch or a tee branch in the main sewer line.

Junction Chambers and Manholes. Figure 18 shows a typical design for a junction chamber and manhole for relatively small sewers. For junctions of large sewers, a special underground structure will ordinarily be required, and the entrance to it will be provided for by a manhole located at one side. Such chambers and manholes are required at every sewer junction and at every point where the sewer

Table 8. Dimensions of Extra-strength Clay Pipe¶

Nominal size, in.	Laying length *L*		Max difference in length of two opposite sides, in.	Outside diameter of barrel, in ‡,§		Inside diameter of socket at ¼ in. above base, in *DS*
	Min, ft*	Limit of minus variation, in. per ft of length†		Min	Max	Min
6	2	1/4	3/8	7 1/16	7 5/16	8 3/16
8	2	1/4	7/16	9 1/4	9 3/4	10 1/2
10	2	1/4	7/16	11 1/2	12	12 3/4
12	2	1/4	7/16	13 3/4	14 5/16	15 1/8
15	3	1/4	1/2	17 3/16	17 13/16	18 5/8
18	3	1/4	1/2	20 5/8	21 3/16	22 1/4
21	3	1/4	9/16	24 1/8	25	25 7/8
24	3	3/8	9/16	27 1/2	28 1/2	29 3/8
27	3	3/8	5/8	31	32 1/8	33
30	3	3/8	5/8	34 3/8	35 5/8	36 1/2
33	3	3/8	5/8	37 5/8	38 15/16	39 7/8
36	3	3/8	11/16	40 3/4	42 1/4	43 1/4

Table 8. (Continued)

Nominal size, in.	Depth of socket, in. *LS*		Thickness of barrel, in. *T*		Thickness of socket at ½ in. from outer end, in. *TS*	
	Nominal	Min	Nominal	Min	Nominal	Min
6	2 1/4	2	11/16	9/16	1/2	7/16
8	2 1/2	2 1/4	7/8	3/4	9/16	1/2
10	2 5/8	2 3/8	1	7/8	5/8	9/16
12	2 3/4	2 1/2	1 3/16	1 1/16	3/4	11/16
15	2 7/8	2 5/8	1 1/2	1 3/8	15/16	7/8
18	3	2 3/4	1 7/8	1 3/4	1 1/8	1 1/16
21	3 1/4	3	2 1/4	2	1 5/16	1 3/16
24	3 3/8	3 1/8	2 1/2	2 1/4	1 1/2	1 3/8
27	3 1/2	3 1/4	2 3/4	2 1/2	1 11/16	1 9/16
30	3 5/8	3 3/8	3	2 3/4	1 7/8	1 3/4
33	3 3/4	3 1/2	3 1/4	3	2	1 13/16
36	4	3 3/4	3 1/2	3 1/4	2 1/16	1 7/8

* There shall be no maximum length. Shorter lengths may be used for closures and specials.
† There is no limit for plus variation.
‡ The average actual inside diameters of pipe having the nominal thickness of barrel shown in Table 8 may be smaller than the nominal sizes.
§ The outside diameter of the barrel may be greater than the maximum figures stated in Table 8 provided the other dimensions are varied accordingly within the specification tolerances.
¶ Dimensions *L*, *DS*, *LS*, *T*, and *TS* refer to the sketch of Table 7.

changes in size, slope, direction, or elevation. It is general practice to install sewers in straight lines between manholes, except that for the larger sizes (36 in. and above) they may be laid on curves. Manholes are usually installed at the upper end of every lateral sewer and in straight-line sewers so that the spacing will not exceed

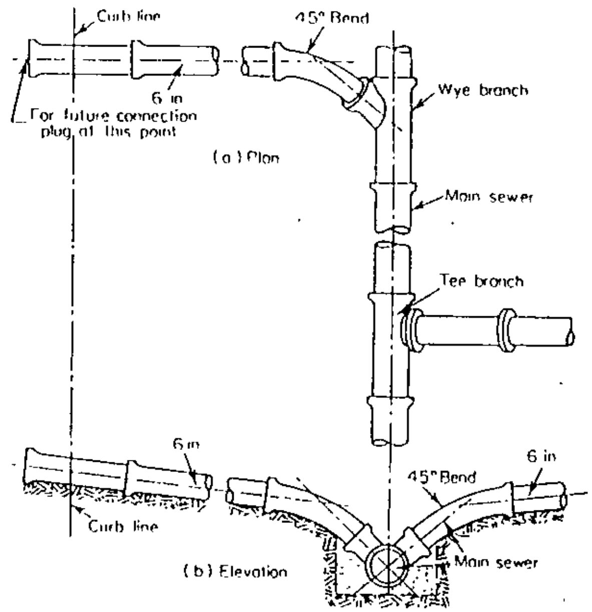
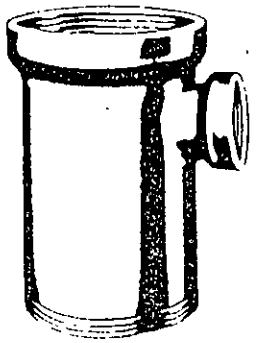


FIG. 16 Typical service connections to a shallow sewer.

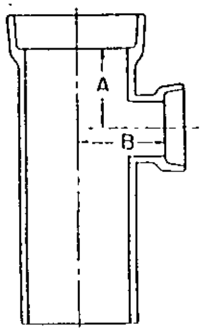
Table 11. Extra-strength Clay-pipe Fittings



"T" branches

are made with the spur set on the barrel at an angle of 90°.

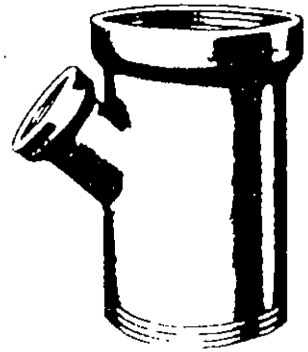
The spur is molded on the barrel before the fitting is dried, burned or glazed, making the whole an integral glazed unit.



Barrel diameter, in.	Nominal laying length, ft	Spur diameter, in.	*A*, in.	*B*, in	Barrel diameter, in.	Nominal laying length, ft	Spur diameter, in.	*A*, in.	*B*, in
4	1	4	5	4¼	21	3, 4	6	7½	14¼
6	1½	4	5¼	5½	21	3, 4	8	8¾	14¾
6	1½	6	6¼	5½	21	3, 4	12	11	15
					21	3, 4	15	12¾	15½
8	2, 3	4	5½	6½	21	3, 4	18	14¼	15¾
8	2, 3	6	6½	6¾					
8	2, 3	8	7¾	7	24	3, 4	6	8	16
					24	3, 4	8	9	16½
10	2, 3	4	6	7	24	3, 4	12	11¼	16¾
10	2, 3	6	6¾	7¾	24	3, 4	15	13	17⅛
10	2, 3	8	7¾	8¼	24	3, 4	18	14½	17¼
10	2, 3	10	9	8½					
					27	3, 4	6	8¼	17¾
12	2, 3	4	6¼	8¾	27	3, 4	8	9¼	18¼
12	2, 3	6	6¾	9	27	3, 4	12	11½	18½
12	2, 3	8	8	9½	27	3, 4	18	15	18¾
12	2, 3	10	9¼	9¾					
12	2, 3	12	10½	10	30	3, 4	6	8½	19½
					30	3, 4	8	9½	20
15	3, 4	4	6½	10½	30	3, 4	12	12	20¼
15	3, 4	6	7	10¼	30	3, 4	18	15½	21
15	3, 4	8	8¼	11¼					
15	3, 4	10	9½	11¼	33	3, 4	6	8¾	21⅜
15	3, 4	12	10½	11½	33	3, 4	12	12½	21⅞
15	3, 4	15	12¼	11¾	33	3, 4	18	16	22⅛
18	3, 4	6	7¼	12½	36	3, 4	6	9¼	22¾
18	3, 4	8	8¼	13	36	3, 4	8	10½	23¼
18	3, 4	12	10½	13¼	36	3, 4	12	12½	23½
18	3, 4	15	12½	13¼	36	3, 4	18	16	24½
18	3, 4	18	13¾	13½					

NOTE: Dimensions *A* and *B* are approximate only. Dimensions not shown are the same as for straight pipe.

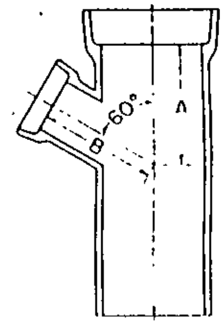
Table 12. Extra-strength Clay-pipe Fittings



"Y" branches

are made with the spur set on the barrel at an angle of 60°.

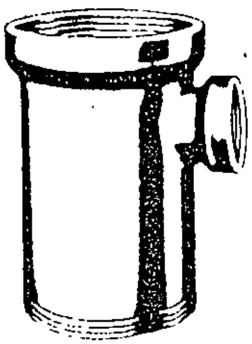
The spur is molded on the barrel before the fitting is dried, burned or glazed, making the whole an integral glazed unit



Barrel diameter, in.	Nominal laying length, ft	Spur diameter, in.	*A*, in.	*B*, in.	Barrel diameter, in.	Nominal laying length, ft	Spur diameter, in.	*A*, in	*B*, in.
4	1	4	8	6¾	21	3, 4	6	17¾	18¼
6	1½	4	8¾	7½	21	3, 4	8	18¾	19½
6	1½	6	9¾	8¼	21	3, 4	12	22¾	20¾
					21	3, 4	15	23¼	22
8	2, 3	4	9¼	8¾					
8	2, 3	6	11¼	9¾	24	3, 4	6	18¼	20¼
8	2, 3	8	12¼	11	24	3, 4	8	20¼	21½
					24	3, 4	12	23¼	22¾
10	2, 3	4	11	10¾	24	3, 4	15	23¾	24
10	2, 3	6	13	11¼					
10	2, 3	8	14	12	27	3, 4	6	21½	22½
10	2, 3	10	15	13	27	3, 4	8	22½	23¼
					27	3, 4	12	24½	24½
12	2, 3	4	11¼	12¼					
12	2, 3	6	12¼	12¾	30	3, 4	6	22½	24
12	2, 3	8	14¼	13½	30	3, 4	8	23½	25¼
12	2, 3	10	15¼	14¼	30	3, 4	12	25½	26½
12	2, 3	12	17¼	15					
					33	3, 4	6	24¼	26½
15	3, 4	4	12¼	13¾	33	3, 4	12	27¼	28½
15	3, 4	6	14¼	14½					
15	3, 4	8	16¼	15	36	3, 4	6	25	28¾
15	3, 4	10	18¼	15⅜	36	3, 4	8	26	29½
15	3, 4	12	19¼	16½	36	3, 4	12	27	30½
18	3, 4	6	16	16½					
18	3, 4	8	18	17¼					
18	3, 4	12	21	18¾					

NOTE: Dimensions *A* and *B* are approximate only. Dimensions not shown are the same as for straight pipe.

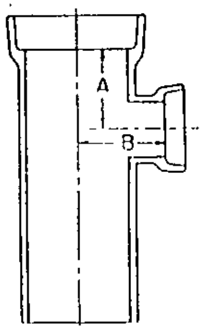
Table 11. Extra-strength Clay-pipe Fittings



"T" branches

are made with the spur set on the barrel at an angle of 90°

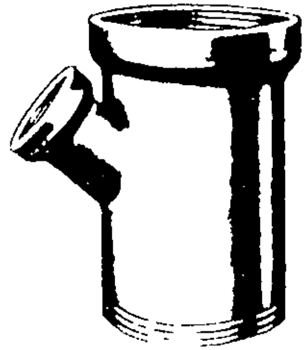
The spur is molded on the barrel before the fitting is dried, burned or glazed, making the whole an integral glazed unit.



Barrel diameter, in.	Nominal laying length, ft	Spur diameter, in.	A, in.	B, in	Barrel diameter, in.	Nominal laying length, ft	Spur diameter, in.	A, in	B, in
4	1	4	5	4¼	21	3, 4	6	7½	14¼
6	1½	4	5¼	5½	21	3, 4	8	8¾	14¾
6	1½	6	6¼	5½	21	3, 4	12	11	15
					21	3, 4	15	12¾	15½
8	2, 3	4	5½	6½	21	3, 4	18	14¼	15¾
8	2, 3	6	6½	6¾					
8	2, 3	8	7¾	7	24	3, 4	6	8	16
					24	3, 4	8	9	16½
10	2, 3	4	6	7	24	3, 4	12	11¼	16¾
10	2, 3	6	6¾	7¾	24	3, 4	15	13	17½
10	2, 3	8	7¾	8¼	24	3, 4	18	14¼	17¼
10	2, 3	10	9	8½					
					27	3, 4	6	8¼	17¾
12	2, 3	4	6¼	8¾	27	3, 4	8	9¼	18¼
12	2, 3	6	6¾	9	27	3, 4	12	11½	18½
12	2, 3	8	8	9½	27	3, 4	18	15	18¾
12	2, 3	10	9¼	9¾					
12	2, 3	12	10½	10	30	3, 4	6	8½	19½
					30	3, 4	8	9½	20
15	3, 4	4	6½	10½	30	3, 4	12	12	20¼
15	3, 4	6	7	10¾	30	3, 4	18	15½	21
15	3, 4	8	8¼	11¼					
15	3, 4	10	9½	11¼	33	3, 4	6	8¾	21½
15	3, 4	12	10½	11½	33	3, 4	12	12½	21¾
15	3, 4	15	12¼	11¾	33	3, 4	18	16	22¼
18	3, 4	6	7¼	12½	36	3, 4	6	9¼	22¾
18	3, 4	8	8¼	13	36	3, 4	8	10½	23¼
18	3, 4	12	10½	13¼	36	3, 4	12	12½	23½
18	3, 4	15	12½	13¼	36	3, 4	18	16	24½
18	3, 4	18	13¼	13½					

NOTE: Dimensions *A* and *B* are approximate only. Dimensions not shown are the same as for straight pipe.

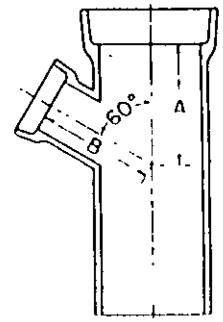
Table 12. Extra-strength Clay-pipe Fittings



"Y" branches

are made with the spur set on the barrel at an angle of 60°.

The spur is molded on the barrel before the fitting is dried, burned or glazed, making the whole an integral glazed unit.



Barrel diameter, in.	Nominal laying length, ft	Spur diameter, in.	A, in.	B, in.	Barrel diameter, in.	Nominal laying length, ft	Spur diameter, in.	A, in.	B, in.
4	1	4	8	6¾	21	3, 4	6	17¾	18¼
6	1½	4	8¾	7½	21	3, 4	8	18¾	19½
6	1½	6	9¾	8¾	21	3, 4	12	22¾	20¾
					21	3, 4	15	23¼	22
8	2, 3	4	9¼	8¾					
8	2, 3	6	11¼	9¾	24	3, 4	6	18¼	20¼
8	2, 3	8	12¼	11	24	3, 4	8	20¼	21½
					24	3, 4	12	23¼	22¼
10	2, 3	4	11	10¾	24	3, 4	15	23¾	24
10	2, 3	6	13	11¾					
10	2, 3	8	14	12	27	3, 4	6	21½	22½
10	2, 3	10	15	13	27	3, 4	8	22½	23¼
					27	3, 4	12	24½	24½
12	2, 3	4	11¼	12¼					
12	2, 3	6	12¾	12¾	30	3, 4	6	22½	24
12	2, 3	8	14¼	13½	30	3, 4	8	23½	25¼
12	2, 3	10	15¼	14¼	30	3, 4	12	25½	26½
12	2, 3	12	17¼	15					
					33	3, 4	6	24¼	26½
15	3, 4	4	12¼	13¾	33	3, 4	12	27¼	28½
15	3, 4	6	14¼	14¼					
15	3, 4	8	16¼	15	36	3, 4	6	25	28¾
15	3, 4	10	18¼	15¾	36	3, 4	8	26	29½
15	3, 4	12	19¼	16½	36	3, 4	12	27	30½
18	3, 4	6	16	16½					
18	3, 4	8	18	17¼					
18	3, 4	12	21	18¾					

NOTE: Dimensions *A* and *B* are approximate only. Dimensions not shown are the same as for straight pipe.

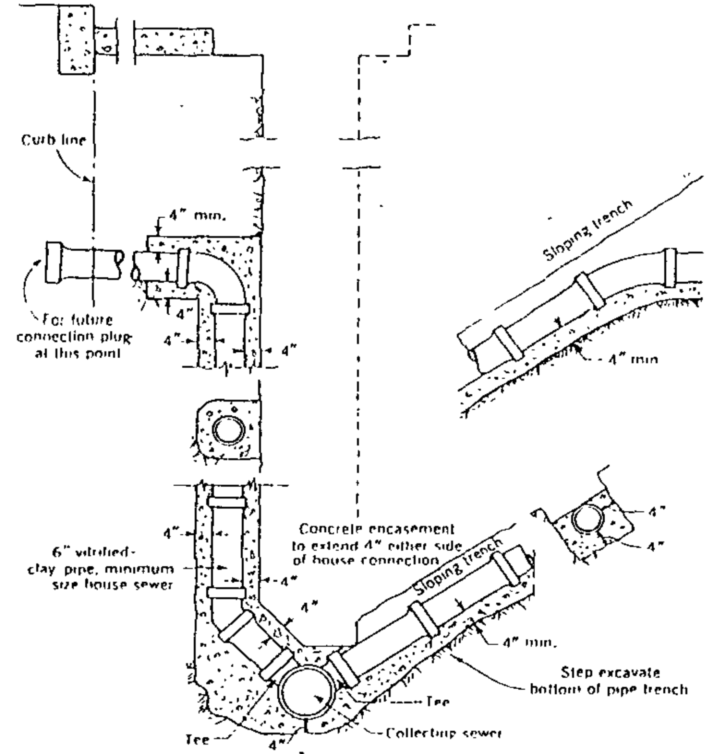


Fig. 17. Typical service connection to a deep sewer.

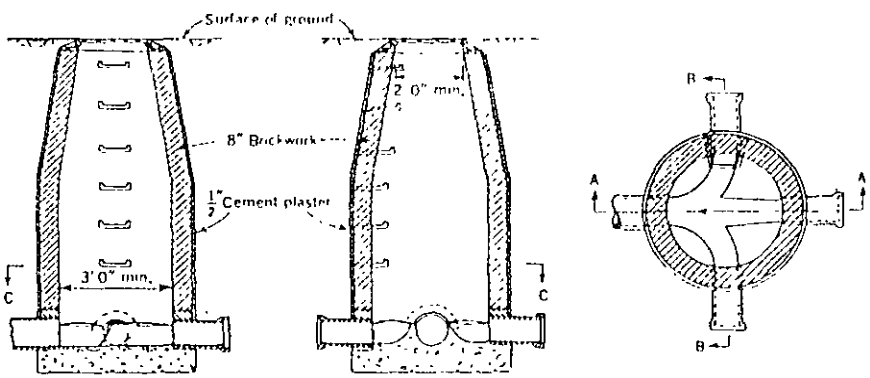


FIG. 18. Junction chamber and manhole for small sewers.

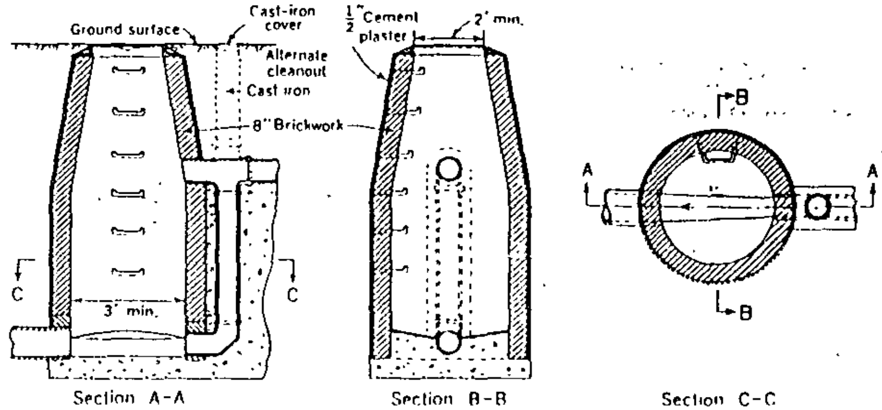


FIG. 19. Drop manhole.

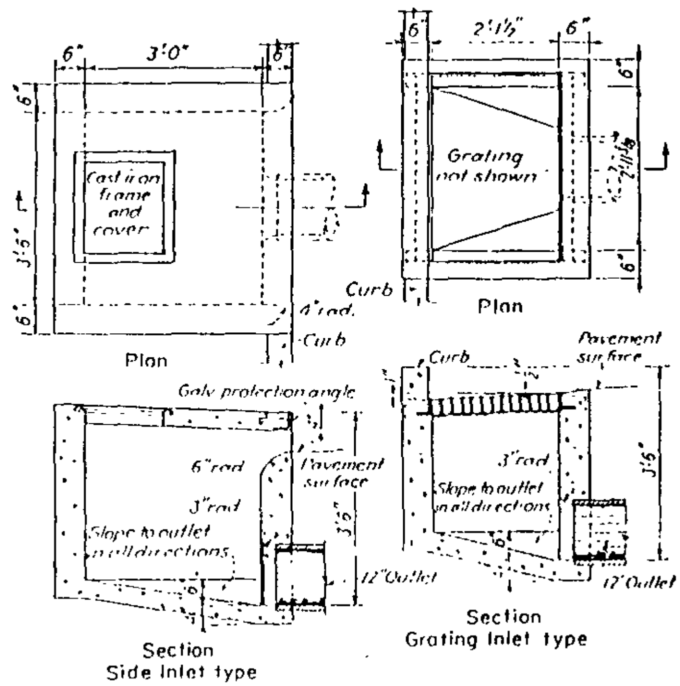


FIG. 20. Street inlets.

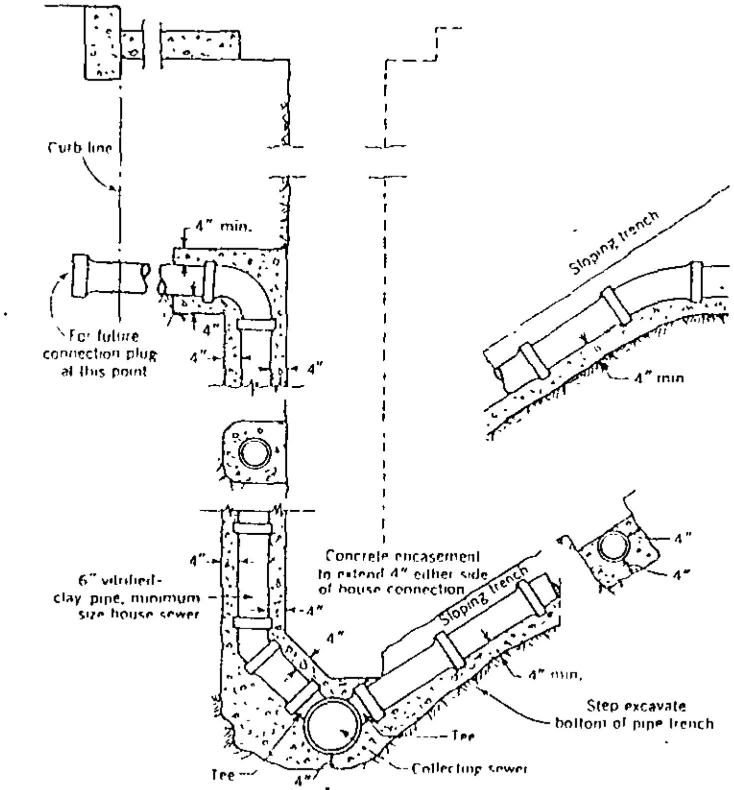


Fig. 17. Typical service connection to a deep sewer.

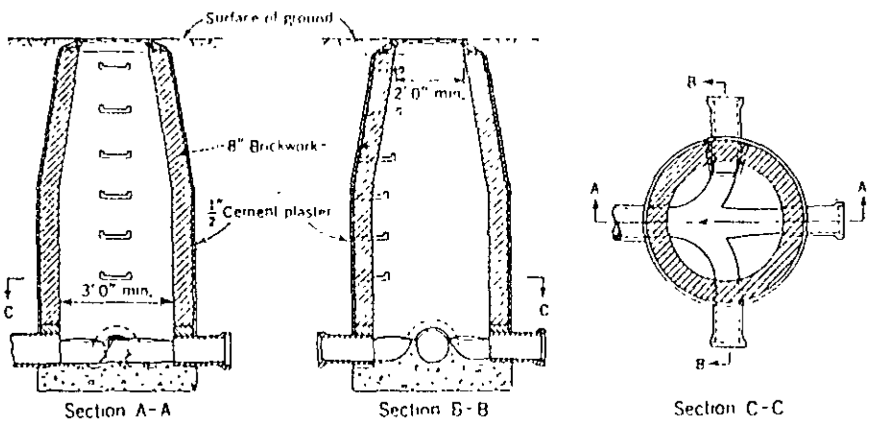


Fig. 18. Junction chamber and manhole for small sewers.

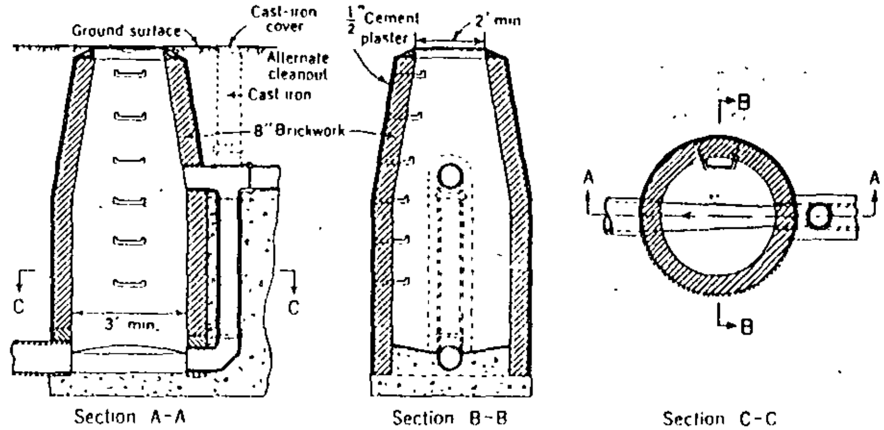


Fig. 19. Drop manhole.

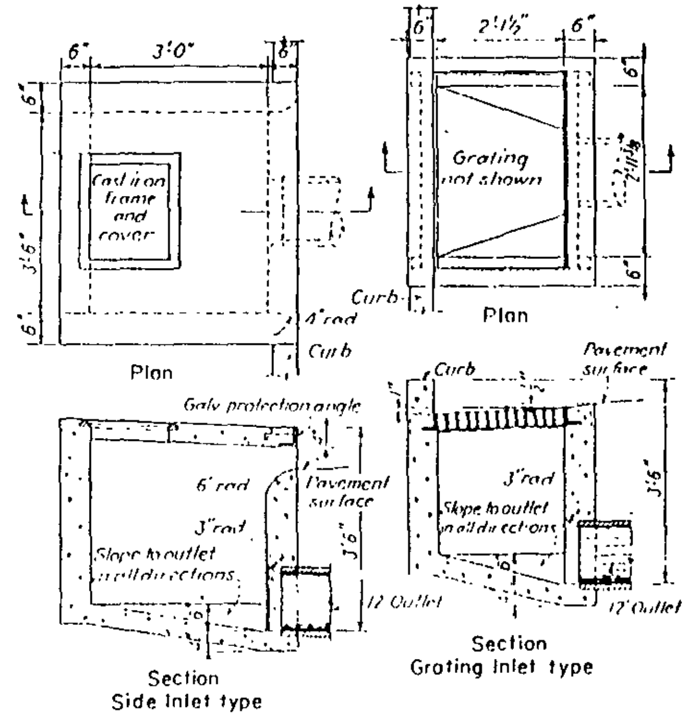


Fig. 20. Street inlets.

about 300 ft. Figure 19 shows typical details of a "drop manhole" at a point where a sewer takes an abrupt drop in grade.

Storm-water Inlets. Storm-water inlets which carry storm water from the streets to the storm sewers are located upstream of the crosswalks at street intersections and at low points. The designs vary considerably, and most cities have adopted their own standard design details. There are three general types of inlets: (1) curb inlets, which have a vertical opening in the curb; (2) gutter inlets, in which a horizontal opening in the gutter is covered by a cast-iron grating; and (3) combination inlets, which combine both the above features. Many types and sizes of standard castings are available for the construction of inlets. Figure 20 shows typical inlet details.

Sewage Pumping. In many sewerage-system layouts it is necessary to provide for pumping at one or more points. The required pumping capacity will vary from a few gallons per minute for stations serving only a few laterals to many millions of gallons per day for stations serving large districts. The smaller stations are frequently built underground, either as built-in-place or as complete factory-assembled units. Pumping is usually done with nonclog centrifugal pumps, although pneumatic ejectors are sometimes used for the smaller installations. The design of sewage pumping stations requires careful attention to the special requirements which are imposed by the function being performed. For detailed discussion of these requirements, the reader is referred to the ASCE-WPCF Manual.[3]

STRUCTURAL REQUIREMENTS

Sewers must be installed so as to be able to withstand the loads imposed upon them by the weight of the earth and any superimposed loads. The supporting strength of a buried pipe depends upon the installation conditions as well as the structural properties of the pipe itself. Sewer pipes are classed as rigid pipes, which cannot deform materially without cracking. For rigid pipes in trenches, the load can be represented by the equation

$$W = CwB^2$$

where W = load, lb per foot of length
w = weight of the soil, lb/cu ft
B = width of trench, ft
C = coefficient whose value depends upon type of soil and ratio of depth of cover to trench width

Table 13 gives values for C and Table 14 gives values of w to be used in the equation.

If a pipe is placed on undisturbed ground and covered with a fill, the load can be estimated from the equation

$$W = CwD^2$$

where D is the pipe diameter. Table 15 gives values of C for the latter condition, which is known as the "projection condition." The load on a pipe placed in a trench will increase with the trench width until it equals the load for the projection condition. If there is doubt as to whether the "ditch condition" or the "projection condition" controls, the load should be calculated by both formulas and the minimum value used.

In addition to the load of the backfill, some allowance should be made for the superimposed loads caused by vehicles. It is usually safe to assume that H-20 wheel loads will be the greatest live loads to be supported. H-20 loads refer to trucks having a gross weight of 20 tons, 80 per cent of which is on the rear axle, each rear wheel carrying 8 tons. Table 16 gives the percentage of wheel loads that can be assumed to be transmitted to buried pipe.[21]

Table 13. Values of C for Use in Formula $W = CwB^2$ *

Ratio of depth to trench width	Sand and damp topsoil	Saturated topsoil	Damp clay	Saturated clay
0.5	0.46	0.46	0.47	0.47
1.0	0.85	0.86	0.88	0.90
1.5	1.18	1.21	1.24	1.28
2.0	1.46	1.50	1.56	1.62
2.5	1.70	1.76	1.84	1.92
3.0	1.90	1.98	2.08	2.20
3.5	2.08	2.17	2.30	2.44
4.0	2.22	2.33	2.49	2.66
4.5	2.34	2.47	2.65	2.87
5.0	2.45	2.59	2.80	3.03
5.5	2.54	2.69	2.93	3.19
6.0	2.61	2.78	3.04	3.33
6.5	2.68	2.86	3.14	3.46
7.0	2.73	2.93	3.22	3.57
7.5	2.78	2.98	3.30	3.67
8.0	2.81	3.03	3.37	3.76
8.5	2.85	3.07	3.42	3.85
9.0	2.88	3.11	3.48	3.92
9.5	2.90	3.14	3.52	3.98
10.0	2.92	3.17	3.56	4.04
11.0	2.95	3.21	3.63	4.14
12.0	2.97	3.24	3.68	4.22
13.0	2.99	3.27	3.72	4.29
14.0	3.00	3.28	3.75	4.34
15.0	3.01	3.30	3.77	4.38
Very great	3.03	3.33	3.85	4.55

* *Iowa State Univ. Eng. Expt. Sta. Bull. 47.*

Table 14. Weights of Ditch-filling Materials

Material	Lb/cu ft
Dry sand	100
Ordinary (damp) sand	115
Wet sand	120
Damp clay	120
Saturated clay	130
Saturated topsoil	115
Sand and damp topsoil	100

Table 15. Values of C for Projection Condition

Ratio, depth of cover/pipe diam	0.5	1.0	1.5	2.0	2.5	3.0	3.5	4.0
C	0.6	1.2	2.0	3.0	4.2	5.6	7.5	10.0

Table 16. Percentage of Wheel Loads Transmitted to Underground Pipes for Unpaved Roadway or Berm Areas *

(Tabulated figures show percentage of wheel load applied to 1 lin ft of pipe.)

Depth of backfill over top of pipe, ft	Trench width at top of pipe, ft						
	1	2	3	4	5	6	7
1	17.0	26.0	28.6	29.7	29.9	30.2	30.3
2	8.3	14.2	18.3	20.7	21.8	22.7	23.0
3	4.3	8.3	11.3	13.5	14.8	15.8	16.7
4	2.5	5.2	7.2	9.0	10.3	11.5	12.3
5	1.7	3.3	5.0	6.3	7.3	8.3	9.0
6	1.0	2.3	3.7	4.7	5.5	6.2	7.0

Live loads transmitted are practically negligible below 6 ft.
* These percentages include both live load and impact transmitted to the pipe.

Pipe Bedding Conditions. The supporting strength of a rigid pipe depends upon the type of bedding used in the installation of the pipe. Four general types of bedding conditions have been defined for ditch conduits:

Type 1, Impermissible Bedding. Little or no care is taken to shape the foundation to fit the lower part of the pipe or to fill and tamp around the pipe.

Type 2, Ordinary Bedding. The soil at the bottom of the trench is shaped to fit the lower part of the pipe with reasonable closeness for a width of at least 50 per cent of the pipe diameter; and the remainder of the pipe is covered to a height of at least 6 in. above its top by granular material which is hand placed and tamped.

Type 3, First-class Bedding. The pipe is carefully bedded on fine granular material in an earth foundation carefully shaped to fit the bottom part of the pipe for a width at least 60 per cent of the diameter; the remainder of the pipe is entirely surrounded to a height at least 1.0 ft above the top by granular materials placed by hand in layers not exceeding 6 in. and thoroughly tamped.

Type 4, Concrete Cradle Bedding. The lower part of the pipe is embedded in concrete.

The load factors, or the ratios of the supporting strength to the crushing load as determined by the three-edge bearing method, for the various types of bedding are generally taken as follows:

Impermissible bedding	1.1
Ordinary bedding	1.5
First-class bedding	1.9
Concrete cradle bedding	2.2-3.4

The factors for the concrete cradle bedding depend upon the amount and quality of the concrete that is used. The value of 2.2 will generally apply when the concrete extends from about one-quarter of the pipe diameter (with minimum of 6 in.) below the pipe to the height where the lower 120 deg sector radii intersect the outside of the pipe. If the concrete is carried up to cover the entire bottom half of the pipe, the load factor may be as high as 3.4. If the entire pipe is encased in concrete with a minimum of $0.25D$ (or 4 in.) both above and below, the load factor may be as high as 4.5.

Safety Factor. The specified minimum strength by the three-edge bearing method for a rigid pipe should be divided by an appropriate safety factor in order to obtain the working strength. Some engineers use safety factors as low as 1.0 to 1.2 for reinforced-concrete pipe culverts. For street sewers a safety factor of 1.5 is recommended by the ASCE-WPCF Manual.

DISPOSAL OF SEWAGE AND STORM WATER

Storm water can ordinarily be disposed of by discharge into any natural drainage channel. Sanitary sewage and industrial waste waters containing objectionable constituents must be disposed of in accordance with the requirements of the local health authorities. The most satisfactory method of disposal of sanitary sewage is to convey it to an adequate public sewerage system. In areas which do not have public sewerage systems, individual disposal systems must be provided. These will vary in size from septic-tank systems for private residences to large treatment plants handling the wastes from large institutions and industries. The design of such systems should usually be done only by those who are experienced in this work and who are familiar with the conditions peculiar to the local area.

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19

FLOW OF SLUDGES AND SLURRIES

William H. Kapfer*

The terms *sludge* and *slurry* are used more or less interchangeably in this chapter to describe those fluid systems which generally consist of more than one phase, such as a solid suspended in a liquid or two immiscible liquids of different densities, or a material of pastelike consistency which behaves under some conditions essentially as a fluid. Through common usage "slurry" refers to a relatively thin or watery, and hence dilute, suspension, whereas "sludge" denotes a mud, or heavy phase, or concentrated suspension, or simply a very viscous fluid. No attempt is made here to define either term arbitrarily; either word implies the existence of a substance which can be made to flow through a piping system but which is not a simple, homogeneous Newtonian fluid such as water.

It is noted particularly that a sludge is not necessarily restricted to a suspension of solids in a liquid but may include certain other systems such as a very viscous liquid dispersed in another liquid or in a gelatinous mass. The transportation of such sludges or slurries in pipelines generally poses a more difficult design problem than that for ordinary simple homogeneous fluids because of several complicating factors. Many of the problems arise from the high viscosity, from the non-Newtonian character, or from the nonhomogeneity of a fluid system and the tendency of suspended material to segregate and settle.

The tendency to settle will vary, not only for different systems, b particular flow conditions. Particle density, shape, and size, as tribution, concentration, and composition, influence the settling Thus, differences can be expected in behavior for fiberlike particles a granular particles or between spherical-shaped and rodlike or fl

* Associate Professor of Chemical Engineering, New York Universit

given in Table 6. An allowance for overhead and contractors' profit should be made in arriving at the probable total cost of a complete piping installation erected in place.

Building Heating. The cost of a two-pipe steam-heating system for the boiler and turbine houses of a large power plant is detailed in Table 7. The cost includes bleeder connections from house-service turbines with a live-steam pressure-reducing station for standby, all supply and return headers and branch circuits, vacuum pumps, etc., as well as the radiation itself. The costs of material and labor are direct costs which do not include engineering, drafting, contractor's profit, or other overhead.

Plumbing. The cost of plumbing and drains such as floor and roof drains is given in Table 8.

Table 8. Cost of Power-plant Plumbing, Floor Drains, and Roof Drains

Average cost per kilowatt of capacity	*Percentage of material*	*Percentage of labor*
$0 36	50	50

For the underground pipes outside buildings, including city water lines, sewers, drains, service water, etc., there should be allowed approximately $0.25 per kilowatt of capacity.

The labor costs on which all the above figures are based are as follows:

Foremen	$5.25 per hour
Pipe fitters	4.25 per hour
Hanger men	3.90 per hour
Pipe coverers.. ...	4.25 per hour
Helpers.. ..	3.20 per hour
Common labor .. .	2.75 per hour

The combined cost for plumbing, heating, and ventilating will be approximately $1.70 per kilowatt for a 200- to 400-mw power station containing several units.

The cost of cast-iron fittings for 125- and 250-lb pressure was between 22 and 30 cents per pound.

The division of the material and labor costs among the pipe and fittings, the hangers, and the insulation is of value in making estimates. This subdivision is given in Table 9.

Table 9. Division of Material and Labor Costs for Piping—Average for Entire Steam Power Plant

Item	Material, %	Labor, %
Pipe and fittings	90	67
Hangers	6	11
Insulation	4	9
Fabrication		8
Handling and miscellaneous		5
Total	100	100

21

WATER-SUPPLY PIPING

Frank M. Kamarck*

Water-distribution systems which serve populated areas are classified broadly as being of the loop, gridiron, or tree types. Within the broad concept, there may be a combining of all three types used as the building blocks for the overall system.

In the *loop system*, large feeder mains that surround areas many city blocks square serve smaller cross-feed lines connected at each end into the main loop.

In the *gridiron* (or grid) system, the piping is laid out in checkerboard fashion with piping usually decreasing in size as the distance increases from the source of supply.

In the *tree system*, there is a single trunk main, reducing in size with increasing distance from its source of supply; branch lines are supplied from the trunk.

The grid and loop systems provide better reliability because of their multiple paths. Grid and loop systems are often backed up with feeder pipes leading directly from the pumping station to remote distribution centers serving to bolster the supply to meet increased demands with growth of population.

The requirements of distribution systems are subject to the requirements of local ordinances and state laws. The following is an average sampling of these pressure and flow requirements:

In residential areas, normal operating conditions: 35 psig
In residential areas, peak conditions: 20 psig
Minimum pipe size for fire service: 6 in. ips
Minimum capacity at each fire hydrant: 600 gpm
Minimum storage capacity: 3 days of maximum use
Maximum line velocity: 10 fps (pressure requirements will govern)
Fire-hydrant spacing: 500 ft or less

* Staff Engineer, Burns and Roe, Inc., Oradell, N.J.

Satisfactory pressure for distribution systems ranges from 30 to 100 psi. Pressures below 30 psi cannot supply water successfully to three- and four-story buildings without booster pumps. Pressures over 100 psi require heavier distribution pipes, produce more leakage, and are too high for use in plumbing fixtures. Pressures in the range of 50 to 75 psi are most generally satisfactory. In cities where there are differences in surface elevation in excess of 100 ft, it is often advisable to zone the system according to elevation in order to avoid excessive pressure at the lower elevations. The different zones have independent reservoirs or supplies from the pumping station and are usually designated as "high service" and "low service."

The water demand for extinguishing fires varies among different districts of a city. The requirements for each city as a whole can be predicated on population, which is the usual basis for estimating water requirements other than for fire fighting. In an estimate of the quantity of water required for extinguishing fires, an allowance should be made for probable losses from broken connections and hydrants which may be left open. Experience with the largest fires that the country has experienced indicates that 20,000 gpm usually should suffice, even in big cities, for fire fighting.

The selection of pipe sizes in distribution networks is influenced more by the necessity of maintaining adequate water pressure than by the economics of pumping costs.

The prime objective of the distribution network is that it supply a sufficient quantity of water to all parts of the system at pressures adequate for the requirements of the consumers, at all times and under all conditions of their demands. In order to accomplish this, reserve capacity is necessary in the form of storage, pumping, and distribution facilities.

Network Analysis of Distribution Systems. The complexity of the analysis required for a well-designed system is in many ways comparable to that of utility power networks. The importance of determining the basic conditions as accurately and as completely as possible cannot be overemphasized, including the range of variation for all conditions. The result of the calculations for the network cannot be any better than the data used.

The data will cover the pressure and flow requirements under the varying rates to be expected in the respective parts of the system; the topography of the service areas; the pumping arrangements and pump characteristics; the location, quantity, and elevation of storage; and the condition of the pipe system with regard to pressure drop and carrying characteristics.

There are several procedures that may be used for the analysis of flow in complex networks:

The Electric Network Analyzer Method (See "Hydraulic Analysis of Water Distribution Systems by Means of an Electric Network Analyzer" by T. R. Camp and H. L. Hazen, *J. New Engl. Water Works Assoc.*, December, 1934)

The Hardy-Cross Method (See "Analysis of Flow in Networks of Conduits or Conductors" by Hardy Cross, *Univ. Illinois Eng. Expt. Sta. Bull.* 286, November, 13, 1936)

The Graphical Method (See "Solution of Transmission Problems of Water Systems" by E. H. Aldrich, *Trans. ASCE.*, 1938)

Electronic Computer Analysis of Hydraulic Networks (See "Pipeline Network Analysis by Electronic Digital Computer" by L. N. Hoag and G. Weinberg, *J. Am. Water Works Assoc.*, May, 1957)

The solution of the flow problem in and head losses of a complex distribution network of water conduits is an extremely tedious and time-consuming task. However, for simpler networks, a trial-and-error method is practicable. In using this method, the pressure drop tables herein and the following relationships will

shorten the work considerably. In the turbulent region (almost all flows will be in this higher flow region, Reynolds number above 2,000), the pressure drop h varies as the 1.85 power of the flow rate Q, that is

$$h_1/h_2 = (Q_1/Q_2)^{1.85}$$

In the viscous-flow region (low flow, Reynolds number below 2,000), the pressure drop varies directly as the flow, or $h_1/h_2 = Q_1/Q_2$. For the same flow, pressure drop varies approximately as the fifth power of the inside diameter D, so that

$$h_1/h_2 = (D_1/D_2)^5$$

In Fig. 1, pressure at $A = P_1$ and pressure at $B = P_2$. Pressure drop between A and $B = P_1 - P_2$ in each of the pipeline branches between A and B.

The flow quantity in each branch will vary in accordance with the characteristics of the branch (internal diameter, length, roughness, and number of fittings) in order to result in the same pressure drop, $P_1 - P_2$. A procedure for the determination of flow rates in the individual branches is given below.

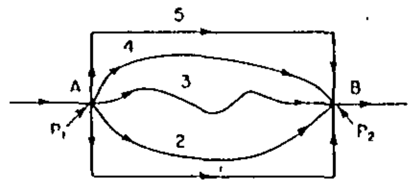


Fig. 1

Make an initial set of flow assumptions by dividing up the flow entering point A, and obtain the friction heads for each branch between A and B. Since all the friction heads between two junction points A and B must be equal, adjust the flows based on the size of the friction heads from this first trial. Obtain a new set of friction heads based on these adjusted flows. After a few trials, adjusting the flows each time, the friction heads for all the branches between A and B can be made equal, or very nearly so, indicating that the proper flows have been obtained in every branch between A and B.

A detailed discussion of and methods for calculating pressure drop in piping are given in Chap. 3. Approximate friction losses for cold water are given in Table 1.

In complex distribution systems, the number of variables is so great and creates so many simultaneous equations that direct solution algebraically of a network is impractical by hand calculation. The equations arise from the three fundamental relationships.

1. The algebraic sum of the rates of discharge toward any junction point is zero; that is, the sum of flows into a junction point must equal sum of flows out of the junction point.
2. The algebraic sum of the head losses around any closed circuit is zero.
3. The head loss is directly proportional to some power of the discharge.

The use of an electronic computer programmed for this analysis provides the highest accuracy in the solution of this problem in the shortest time and least relative cost.

Illustrative Example. For example, the typical computer service charge at this time is about \$100 for an average 20-loop problem. Computer service is available with program capacity of 500 loops and 1,000 lines based on a modified Hardy-Cross method. A typical method[4,*] of setting up the problem for an electronic computer is as follows:

* Superscript numerals refer to bibliographical references which are listed at the end of this chapter.

Table 1. Friction Loss for Water in Feet per 100 Feet of Pipe

6-in. nominal wrought iron or steel, Schedule 40, ID = 6.065 in.			12-in. nominal wrought iron or steel, Schedule 40, ID = 11.938 in.		
Discharge, gpm	Velocity, fps	Friction, ft per 100 ft of pipe	Discharge, gpm	Velocity, fps	Friction, ft per 100 ft of pipe
10	0.111	0.00146	100	0.287	0.00325
20	0.222	0.00487	120	0.344	0.00448
30	0.333	0.00988	140	0.401	0.00590
40	0.444	0.0164	160	0.459	0.00747
50	0.555	0.0244	180	0.516	0.00920
60	0.666	0.0337	200	0.573	0.0111
70	0.777	0.0445	220	0.631	0.0132
80	0.888	0.0564	240	0.688	0.0155
90	0.999	0.0698	260	0.745	0.0180
100	1.11	0.0843	280	0.802	0.0206
120	1.33	0.118	300	0.860	0.0233
140	1.55	0.155	350	1.00	0.0306
160	1.78	0.198	400	1.15	0.0391
180	2.00	0.246	450	1.29	0.0485
200	2.22	0.299	500	1.43	0.0587
220	2.44	0.357	550	1.58	0.0698
240	2.66	0.419	600	1.72	0.0820
260	2.89	0.487	650	1.86	0.0950
280	3.11	0.560	700	2.01	0.109
300	3.33	0.637	750	2.15	0.124
320	3.55	0.719	800	2.29	0.140
340	3.78	0.806	850	2.44	0.156
360	4.00	0.898	900	2.58	0.173
380	4.22	0.991	950	2.72	0.191
400	4.44	1.09	1,000	2.87	0.210
420	4.66	1.20	1,100	3.15	0.251
440	4.89	1.31	1,200	3.44	0.296
460	5.11	1.42	1,300	3.73	0.344
480	5.33	1.54	1,400	4.01	0.395
500	5.55	1.66	1,500	4.30	0.450
550	6.11	1.99	1,600	4.59	0.509
600	6.66	2.34	1,700	4.87	0.572
650	7.22	2.73	1,800	5.16	0.636
700	7.77	3.13	1,900	5.45	0.704
750	8.33	3.57	2,000	5.73	0.776
800	8.88	4.03	2,200	6.31	0.930
850	9.44	4.53	2,400	6.88	1.093
900	9.99	5.05	2,600	7.45	1.28
950	10.5	5.60	2,800	8.03	1.47
1,000	11.1	6.17	3,000	8.60	1.68
1,100	12.2	7.41	3,200	9.17	1.90
1,200	13.3	8.76	3,400	9.75	2.13
1,300	14.4	10.2	3,600	10.3	2.37
1,400	15.5	11.8	3,800	10.9	2.63
1,500	16.7	13.5	4,000	11.5	2.92
1,600	17.8	15.4	4,500	12.9	3.65
1,700	18.9	17.3	5,000	14.3	4.47
1,800	20.0	19.4	5,500	15.8	5.38
1,900	21.1	21.6	6,000	17.2	6.39
2,000	22.2	23.8	6,500	18.6	7.47
2,100	23.3	26.2	7,000	20.1	8.63
2,200	24.4	28.8	7,500	21.5	9.88
2,300	25.5	31.4	8,000	22.9	11.20
2,400	26.6	34.2	8,500	24.4	12.6
2,500	27.8	37.0	9,000	25.8	14.1
2,600	28.9	39.9	9,500	27.2	15.7
2,700	30.0	42.9	10,000	28.7	17.4
2,800	31.1	46.1	11,000	31.5	21.0
2,900	32.2	49.4	12,000	34.4	24.8
3,000	33.3	52.8	13,000	37.3	28.9
3,200	35.5	59.9	14,000	40.1	33.5
3,400	37.8	67.4	15,000	43.0	38.4
3,600	40.0	75.5	16,000	45.9	43.7
3,800	42.2	84.1	17,000	48.7	49.2
4,000	44.4	93.1	18,000	51.6	55.2
			19,000	54.5	61.5
			20,000	57.3	68.1

Table 1. (Continued)

18-in. OD Steel, Schedule 40, ID = 16.876 in.			24-in. OD Steel, Schedule 40, ID = 22.626 in.		
Discharge, gpm	Velocity, fps	Friction, ft per 100 ft of pipe	Discharge, gpm	Velocity, fps	Friction, ft per 100 ft of pipe
300	0.430	0.00437	300	0.239	0.00107
400	0.574	0.00730	400	0.319	0.00178
500	0.717	0.0109	500	0.399	0.00267
600	0.861	0.0152	600	0.479	0.00371
700	1.00	0.0201	700	0.559	0.00490
800	1.15	0.0256	800	0.638	0.00621
900	1.29	0.0318	900	0.718	0.00767
1,000	1.43	0.0386	1,000	0.798	0.00928
1,200	1.72	0.0541	1,200	0.958	0.0129
1,400	2.01	0.0719	1,400	1.12	0.0171
1,600	2.30	0.092	1,600	1.28	0.0219
1,800	2.58	0.114	1,800	1.44	0.0272
2,000	2.87	0.139	2,000	1.60	0.0330
2,500	3.59	0.211	2,500	1.99	0.0499
3,000	4.30	0.297	3,000	2.39	0.0700
3,500	5.02	0.397	3,500	2.79	0.0934
4,000	5.74	0.511	4,000	3.19	0.120
4,500	6.45	0.639	4,500	3.59	0.149
5,000	7.17	0.781	5,000	3.99	0.181
6,000	8.61	1.11	6,000	4.79	0.257
7,000	10.0	1.49	7,000	5.59	0.343
8,000	11.5	1.93	8,000	6.38	0.441
9,000	12.9	2.42	9,000	7.18	0.551
10,000	14.3	2.97	10,000	7.98	0.671
12,000	17.2	4.21	12,000	9.58	0.959
14,000	20.1	5.69	14,000	11.2	1.29
16,000	22.9	7.41	16,000	12.8	1.67
18,000	25.8	9.31	18,000	14.4	2.10
20,000	28.7	11.5	20,000	16.0	2.58
22,000	31.6	13.9	22,000	17.6	3.10
24,000	34.4	16.5	24,000	19.2	3.67
26,000	37.3	19.2	26,000	20.7	4.29
28,000	40.2	22.2	28,000	22.3	4.96
30,000	43.0	25.5	30,000	23.9	5.68
32,000	45.9	29.0	32,000	25.5	6.42
34,000	48.8	32.8	34,000	27.1	7.22
36,000	51.6	36.8	36,000	28.7	8.08
38,000	54.5	40.8	38,000	30.3	9.00
40,000	57.4	45.0	40,000	31.9	9.98
42,000	60.2	49.7	42,000	33.5	11.0
44,000	63.1	54.5	44,000	35.1	12.1
46,000	66.0	59.5	46,000	36.7	13.2
48,000	68.9	64.8	48,000	38.3	14.3
50,000	71.7	70.2	50,000	39.9	15.5
55,000	78.9	84.8	55,000	43.9	18.7
60,000	86.1	101	60,000	47.9	22.3
65,000	93.2	118	65,000	51.9	26.2
70,000	100.4	136	70,000	55.9	30.4
			75,000	59.8	34.8
			80,000	63.8	39.4
			85,000	67.8	44.4
			90,000	71.8	49.7
			95,000	75.8	55.5
			100,000	79.8	61.5
			110,000	87.8	74.0
			120,000	95.8	88.0
			130,000	103.7	103
			140,000	112	119
			150,000	120	137

Note

1 gal, U.S. = 0.1337 cu ft

1 cu ft = 7.48 gal, U.S.

For water at 60 F

1 ft head = 0.433 psi

1 psi = 2.301 ft head

Table I. (Continued)

30-in. OD Steel, Schedule 20, ID — 29 000 in			36-in. ID Steel		
Discharge, gpm	Velocity, fps	Friction, ft per 100 ft of pipe	Discharge, gpm	Velocity, fps	Friction, ft per 100 ft of pipe
400	0.191	0.000540	1,000	0.315	0.000988
500	0.243	0.000805	1,200	0.378	0.00137
600	0.291	0.001115	1,400	0.441	0.00181
700	0.340	0.00147	1,600	0.504	0.00231
800	0.389	0.00187	1,800	0.567	0.00285
900	0.437	0.00231	2,000	0.630	0.00344
1,000	0.486	0.00280	2,500	0.788	0.00517
1,200	0.583	0.00390	3,000	0.946	0.00721
1,400	0.680	0.00514	3,500	1.103	0.00957
1,600	0.777	0.00652	4,000	1.26	0.0122
1,800	0.874	0.00814	4,500	1.41	0.0152
2,000	0.971	0.00986	5,000	1.58	0.0185
2,500	1.21	0.0148	6,000	1.89	0.0260
3,000	1.46	0.0206	7,000	2.21	0.0345
3,500	1.70	0.0276	8,000	2.52	0.0442
4,000	1.94	0.0354	9,000	2.84	0.0551
4,500	2.19	0.0440	10,000	3.15	0.0670
5,000	2.43	0.0535	12,000	3.78	0.0942
6,000	2.91	0.0750	14,000	4.41	0.126
7,000	3.40	0.100	16,000	5.01	0.162
8,000	3.89	0.129	18,000	5.67	0.203
9,000	4.37	0.161	20,000	6.30	0.248
10,000	4.86	0.196	25,000	7.88	0.378
12,000	5.83	0.277	30,000	9.46	0.540
14,000	6.80	0.371	35,000	11.03	0.724
16,000	7.77	0.478	40,000	12.6	0.941
18,000	8.74	0.598	45,000	14.1	1.18
20,000	9.71	0.732	50,000	15.8	1.45
25,000	12.1	1.13	60,000	18.9	2.07
30,000	14.6	1.61	70,000	22.1	2.81
35,000	17.0	2.17	80,000	25.2	3.66
40,000	19.4	2.83	90,000	28.4	4.59
45,000	21.9	3.56	100,000	31.5	5.64
50,000	24.3	4.38	120,000	37.8	8.05
60,000	29.1	6.23	140,000	44.1	10.9
70,000	34.0	8.43	160,000	50.4	14.2
80,000	38.9	11.0	180,000	56.7	17.9
90,000	43.7	13.8	200,000	63.0	22.1
100,000	48.6	17.0	250,000	78.8	34.4
110,000	53.4	20.6	300,000	94.6	49.4
120,000	58.3	24.5	350,000	110	67.0
130,000	63.1	28.7	400,000	126	87.3
140,000	68.0	33.3			
150,000	72.9	38.2			
160,000	77.7	43.3			
170,000	82.6	48.8			
180,000	87.4	54.7			
190,000	92.3	60.8			
200,000	97.1	67.1			
210,000	102	73.8			
220,000	107	81.0			
230,000	112	88.6			
240,000	117	96.7			
250,000	121	106			

Table I. (Continued)

42-in. ID Steel			48-in. ID Steel		
Discharge, gpm	Velocity, fps	Friction, ft per 100 ft of pipe	Discharge, gpm	Velocity, fps	Friction, ft per 100 ft of pipe
1,000	0.232	0.000471	1,500	0.266	0.000508
1,500	0.347	0.000977	2,000	0.355	0.000855
2,000	0.463	0.00164	2,500	0.443	0.00129
2,500	0.579	0.00246	3,000	0.532	0.00180
3,000	0.695	0.00343	3,500	0.621	0.00238
3,500	0.811	0.00454	4,000	0.709	0.00304
4,000	0.926	0.00580	4,500	0.798	0.00378
4,500	1.042	0.00720	5,000	0.887	0.00458
5,000	1.16	0.00874	6,000	1.064	0.00636
6,000	1.39	0.0122	7,000	1.24	0.00844
7,000	1.62	0.0162	8,000	1.42	0.0108
8,000	1.85	0.0208	9,000	1.60	0.0134
9,000	2.08	0.0258	10,000	1.77	0.0163
10,000	2.32	0.0314	12,000	2.13	0.0229
12,000	2.78	0.0441	14,000	2.48	0.0305
14,000	3.24	0.0591	16,000	2.84	0.0391
16,000	3.71	0.0758	18,000	3.19	0.0488
18,000	4.17	0.0944	20,000	3.55	0.0598
20,000	4.63	0.115	25,000	4.43	0.0910
25,000	5.79	0.176	30,000	5.32	0.128
30,000	6.95	0.250	35,000	6.21	0.172
35,000	8.11	0.314	40,000	7.09	0.222
40,000	9.26	0.433	45,000	7.98	0.278
45,000	10.42	0.545	50,000	8.87	0.341
50,000	11.6	0.668	60,000	10.64	0.484
60,000	13.9	0.946	70,000	12.4	0.652
70,000	16.2	1.27	80,000	14.2	0.849
80,000	18.5	1.66	90,000	16.0	1.06
90,000	20.8	2.08	100,000	17.7	1.30
100,000	23.2	2.57	120,000	21.3	1.87
120,000	27.8	3.67	140,000	24.8	2.51
140,000	32.4	4.98	160,000	28.4	3.26
160,000	37.1	6.46	180,000	31.9	4.11
180,000	41.7	8.12	200,000	35.5	5.05
200,000	46.3	10.00	250,000	44.3	7.88
250,000	57.9	15.6	300,000	53.2	11.3
300,000	69.5	22.1	350,000	62.1	15.3
350,000	81.1	30.4	400,000	70.9	20.0
400,000	92.6	39.6	450,000	79.8	25.2
450,000	104.2	50.1	500,000	88.7	31.1
500,000	116	61.7	550,000	97.5	37.6
			600,000	106.4	44.7

Table I. (Continued)

54-in. ID Steel			60-in. ID Steel		
Discharge, gpm	Velocity, fps	Friction, ft per 100 ft of pipe	Discharge, gpm	Velocity, fps	Friction, ft per 100 ft of pipe
2,000	0.280	0.000488	2,000	0.227	0.000293
2,500	0.350	0.000733	2,500	0.284	0.000440
3,000	0.420	0.00102	3,000	0.340	0.000612
3,500	0.490	0.00134	3,500	0.397	0.000810
4,000	0.560	0.00172	4,000	0.454	0.00103
4,500	0.630	0.00213	4,500	0.511	0.00128
5,000	0.700	0.00257	5,000	0.567	0.00155
6,000	0.840	0.00358	6,000	0.681	0.00216
7,000	0.981	0.00476	7,000	0.794	0.00285
8,000	1.121	0.00610	8,000	0.908	0.00365
9,000	1.26	0.00760	9,000	1.021	0.00454
10,000	1.40	0.00920	10,000	1.13	0.00550
12,000	1.68	0.0129	12,000	1.36	0.00766
14,000	1.96	0.0171	14,000	1.59	0.0102
16,000	2.24	0.0219	16,000	1.82	0.0131
18,000	2.52	0.0273	18,000	2.04	0.0163
20,000	2.80	0.0333	20,000	2.27	0.0198
25,000	3.50	0.0504	25,000	2.84	0.0301
30,000	4.20	0.0713	30,000	3.40	0.0424
35,000	4.90	0.0958	35,000	3.97	0.0567
40,000	5.60	0.124	40,000	4.54	0.0730
45,000	6.30	0.155	45,000	5.11	0.0916
50,000	7.00	0.189	50,000	5.67	0.112
60,000	8.40	0.267	60,000	6.81	0.158
70,000	9.81	0.358	70,000	7.94	0.213
80,000	11.21	0.465	80,000	9.08	0.275
90,000	12.6	0.586	90,000	10.21	0.344
100,000	14.0	0.715	100,000	11.3	0.420
120,000	16.8	1.02	120,000	13.6	0.600
140,000	19.6	1.38	140,000	15.9	0.806
160,000	22.4	1.80	160,000	18.2	1.04
180,000	25.2	2.26	180,000	20.4	1.32
200,000	28.0	2.77	200,000	22.7	1.62
250,000	35.0	4.32	250,000	28.4	2.52
300,000	42.0	6.19	300,000	34.0	3.60
350,000	49.0	8.40	350,000	39.7	4.88
400,000	56.0	11.00	400,000	45.4	6.34
450,000	63.0	13.9	450,000	51.1	8.01
500,000	70.0	17.0	500,000	56.7	9.87
550,000	77.0	20.6	550,000	62.4	11.9
600,000	84.0	24.5	600,000	68.1	14.1
650,000	98.1	28.7	650,000	73.8	16.6
700,000	112.1	33.2	700,000	79.4	19.2
			750,000	85.1	22.0
			800,000	90.8	25.0

Table I. (Continued)

72-in. ID Steel			84-in. ID Steel		
Discharge gpm	Velocity fps	Friction, ft per 100 ft of pipe	Discharge, gpm	Velocity, fps	Friction, ft per 100 ft of pipe
2,000	0.158	0.000123	3,000	0.174	0.000121
2,500	0.197	0.000183	4,000	0.232	0.000203
3,000	0.237	0.000254	5,000	0.289	0.000306
3,500	0.276	0.000336	6,000	0.347	0.000425
4,000	0.316	0.000427	7,000	0.405	0.000562
4,500	0.355	0.000530	8,000	0.463	0.000717
5,000	0.394	0.000640	9,000	0.521	0.000891
6,000	0.473	0.000890	10,000	0.579	0.00108
7,000	0.552	0.00118	12,000	0.695	0.00150
8,000	0.631	0.00150	14,000	0.811	0.00199
9,000	0.710	0.00186	16,000	0.926	0.00255
10,000	0.789	0.00227	18,000	1.042	0.00316
12,000	0.947	0.00313	20,000	1.16	0.00384
14,000	1.104	0.00418	25,000	1.45	0.00579
16,000	1.26	0.00538	30,000	1.74	0.00810
18,000	1.42	0.00673	35,000	2.03	0.0108
20,000	1.58	0.00822	40,000	2.32	0.0139
25,000	1.97	0.0124	45,000	2.61	0.0174
30,000	2.37	0.0173	50,000	2.89	0.0212
35,000	2.76	0.0231	60,000	3.47	0.0298
40,000	3.16	0.0297	70,000	4.05	0.0398
45,000	3.55	0.0370	80,000	4.63	0.0513
50,000	3.94	0.0450	90,000	5.21	0.0640
60,000	4.73	0.0637	100,000	5.79	0.0781
70,000	5.52	0.0850	120,000	6.95	0.111
80,000	6.31	0.110	140,000	8.11	0.149
90,000	7.10	0.138	160,000	9.26	0.193
100,000	7.89	0.168	180,000	10.42	0.242
120,000	9.47	0.237	200,000	11.6	0.297
140,000	11.04	0.321	250,000	14.5	0.458
160,000	12.6	0.414	300,000	17.4	0.649
180,000	14.2	0.522	350,000	20.3	0.880
200,000	15.8	0.642	400,000	23.2	1.14
250,000	19.7	1.00	450,000	26.1	1.44
300,000	23.7	1.42	500,000	28.9	1.78
350,000	27.6	1.92	550,000	31.8	2.14
400,000	31.6	2.50	600,000	34.7	2.54
450,000	35.5	3.16	650,000	37.6	2.97
500,000	39.4	3.88	700,000	40.5	3.43
550,000	43.4	4.69	750,000	43.4	3.93
600,000	47.3	5.56	800,000	46.3	4.47
650,000	51.3	6.52	850,000	49.2	5.04
700,000	55.2	7.56	900,000	52.1	5.64
750,000	59.2	8.67	950,000	55.0	6.29
800,000	63.1	9.83	1,000,000	57.9	6.95
850,000	67.0	11.1			
900,000	71.0	12.4			

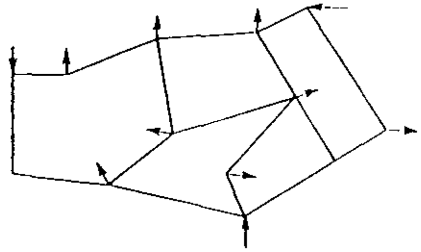


FIG. 2

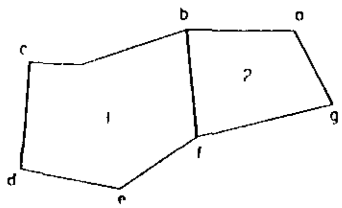


FIG. 3

1. Make a skeleton drawing of the network. Indicate by appropriate arrows the points of constant flow input or output, constant head input or output (see Fig. 2).

2. Number all loops in the system in arbitrary sequence. Do not include "loops around loops." For example, in Fig. 3 there are two loops, not three. The large loop (*abcdefg*) is not numbered. The two basic loops (*abfg* and *bcdef*) are numbered.

3. Number each line. A line has two ends. An end may be a point at which water is drawn from or added to the system, one at which pipe characteristics change, or a tee joint. For example, in Fig. 4, the point *x* is the meeting of three lines, not two; point *y* is the meeting of two lines where an 8-in. pipe joins a 10-in. pipe; point *z* is simply a bend in the single pipe and is not the end of any line, although it could have been specified as one, if desired. Figure 4 shows the complete numbering of the system shown in Fig. 2. Note that each line is numbered once and only once, even though it may be in more than one loop. Also note that the numbering is serial; that is, if there are *n* branches, each of the numbers from 1 to *n* must be used in the numbering.

4. Assign a base direction: Put an arrow on each line in loop 1, indicating the clockwise direction (as shown in Fig. 5). Then put an arrow on each line in loop 2, indicating clockwise direction, except where a line which previously has been assigned a direction is encountered. Then the original assignment is not changed. In Fig. 5, line 4 is a member of loop 1 and also of loop 2 but has been given a base direction of loop 1. The line 4 assignment is not changed. This process is continued for every loop in the network, an arrow being assigned in a clockwise direction whenever it has not been assigned previously.

5. In water-distribution systems, the situation often is encountered where system pressure must be raised by the use of booster pumps in series with the supply pipeline. If the higher pressure area is connected to the remainder of the system at one point only, the two pressure-zone networks are hydraulically independent problems. If the pressure zones are connected at two or more points, the booster pumps must be included in the appropriate loops.

For all loops containing booster pumps, an unbalanced or residual head H_0 must be determined. This is done by algebraically summing the assumed constant head changes at the boosters in a clockwise direction.

Note that head *losses* are considered as positive in sign, so proceeding from the suction side of a pump to the discharge side gives a negative head loss.

Following the hydraulic analysis, a check should be made to assure that the pumping head assumptions are sufficiently accurate. The resulting flow-rate values should allow optimum hydraulic design of the booster-station installations.

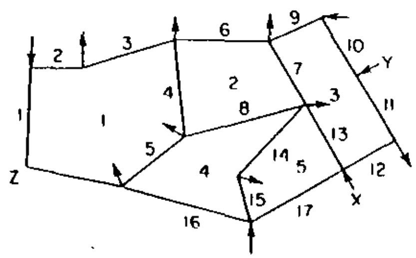


FIG. 4

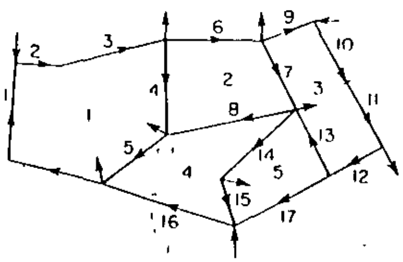


FIG. 5

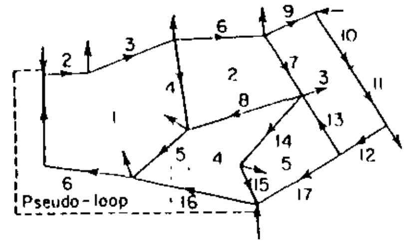


FIG. 6

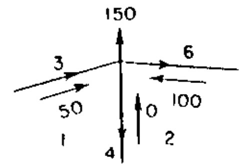


FIG. 7

6. Additional "pseudo-loops" must now be added to the list if there is more than one constant head input (see Fig. 6). If the number of such inputs is *m*, trace $(m - 1)$ paths between inputs in the same manner in which the loops were traced, making sure that each constant head input is used at the end of at least one of these "loops." If the direction of procedure is from the lower to the higher input in each path, H_0 will be the positive difference in the head loss between the two inputs. If booster pumps are encountered, the head change across such pumps must be algebraically added to the head difference between the inputs in order to obtain the H_0 for the "pseudo-loops."

When the listing of all the loops has been completed (including the consideration of booster pumps), the work should be carefully checked, preferably by a second person, since any errors will completely upset the calculations. Note that pseudo-loops do not introduce any new lines. Note also that each pseudo-loop must be assigned its own number.

7. The only remaining task is to supply initial flow values and pipe characteristics which the computer can use as starting values for the calculations. The only restriction on these values is that they satisfy the mass balance condition at each junction. That is, the sum of the flow *into* a junction must equal the sum of the flows *out* of the junction. For example, Fig. 7 shows the junction of lines 3, 4, and 6; flows of 50 gpm in line 3 and 100 gpm in line 6 would satisfy the condition.

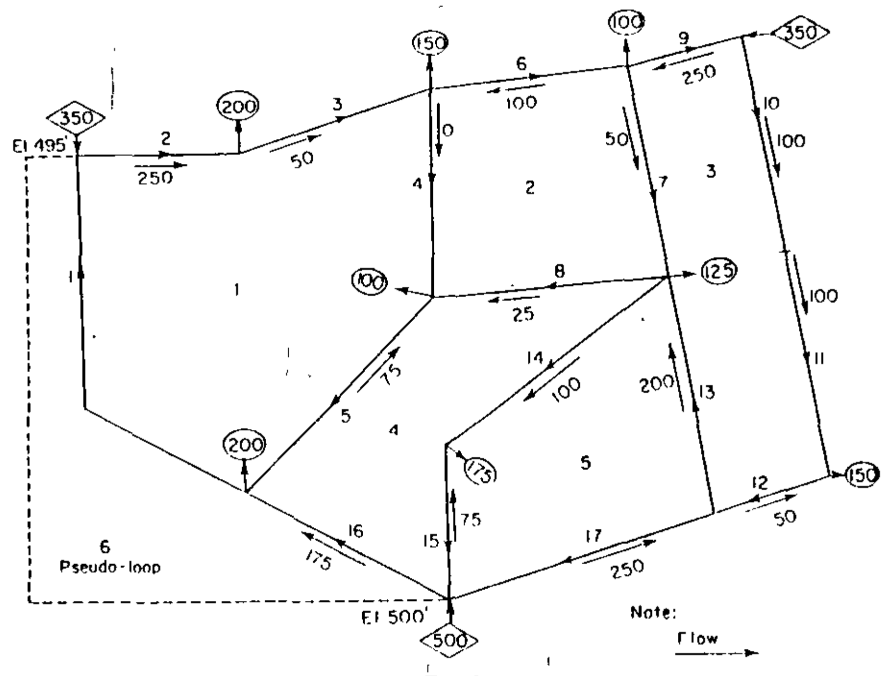


FIG. 8

Proceeding in this manner, balance every junction in the network, working toward the variable-flow (constant-head) inputs which can take up the slack. When all flows are specified, check the accuracy of the work by summing the inputs and outputs. If these sums are unequal, some computational error has been made and must be corrected. The complete schematic for this system is shown in Fig 8. This schematic includes the assumed starting values of the flows.

CIRCULATING COOLING-WATER SYSTEMS FOR POWER PLANTS OR INDUSTRIAL PLANTS

General Requirements. The complete problem for supplying the cooling water to an industrial plant can best be illustrated in the complexity of the main cooling- or circulating-water system for a steam-electric station. A steam power plant with a name-plate rating of 750,000 kw requires as much cooling water as the domestic water requirements of a city with four to five million population.

The circulating cooling-water system for a steam power plant takes water from some source, such as a river, lake, or cooling-tower basin, and provides continuous cooling to the condenser. The circulating water condenses the turbine exhaust steam, thereby lowering the turbine exhaust pressure so that additional energy can be extracted from the steam. The importance of a continued cooling-water supply is such that, if the cooling-water supply should fail, the resulting rise in exhaust steam back pressure will cause an automatic shutdown of the turbine-generator unit. Reliability, efficiency, and economy are the prime design considerations for a steam power-plant circulating cooling-water system. The availability of this cooling water plays an important role in the selection of the steam power-plant site.

A fundamental decision has to be made as to whether the circulating-water system will be designed on a single generating unit basis or to supply water for multiple units with one circulating-water system. This basic decision can be made only after considering all the special conditions for the particular plant. The obvious economic appeal of supplying the multiple unit by means of a single circulating system has to be balanced against the train of problems that follow in its wake. The large variation of flow and its related pressure drop, between full operation of all units and the operation of a single unit, may create such system complexity, for example, at the intake structure, that it will cancel out most of the economic advantage. The variation in the pumping head may be beyond the operating range of the circulating-water pumps. Also, the provisions needed for reliability of the system in order to minimize or avoid total shutdown in event of failure of some part may add substantially to the overall cost of the system.

Types of Systems. Many types of system arrangements are in common usage. Those encountered most frequently are defined and discussed briefly below.

A once-through or direct-condensing system is one in which water is drawn from a river, lake, or other large body of water and, after passing through the condenser, is returned to the same or a different body of water.

A recirculating system is one in which the same water is used repeatedly and is itself cooled between passes through the condenser. Means of cooling the water has to be provided in the form of cooling towers, spray ponds, or cooling ponds. Make-up water has to be provided to take care of water losses due to evaporation, leakage, and other causes.

A siphon system is one in which the siphon principle is employed to carry the water through elevated parts of the system, such as the condenser, in order to reduce the pumping power required. These elevated portions of the water system operate under a partial vacuum.

A pressure system is one in which the water flows under a positive head throughout. This system is generally used with recirculating systems, such as with cooling-tower installations.

Frequently, features of two or more of the above systems are combined in a single installation. For example, the once-through and siphon systems may be combined to produce the once-through siphon system which is shown in Fig 9. The cooling water enters the system through the intake structure (trash racks, traveling screen, fine screens) into the suction chamber and into the circulating pump suction. The water is discharged by the circulating pump into the tunnel or pipe leading to the condenser. The water passes through the condenser aided by siphon action and out through the discharge tunnel or piping to the seal well and discharge structure.

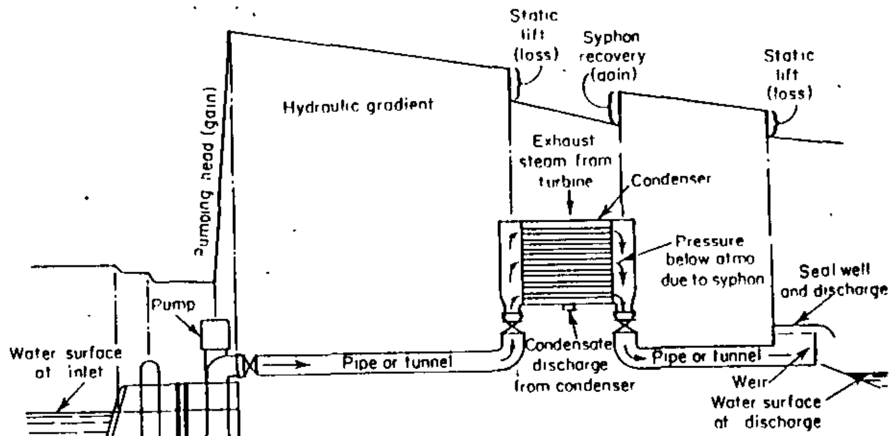


FIG. 9. Once-through siphon system. Intake structure: trash racks, traveling screens, fine screens, circulating water pumps, valves, service water and screen wash pumps, hoisting devices for pumps and screens, water-treatment equipment.

When topography of the plant area permits and the fluctuation of the water level is not excessive, consideration should be given to locating the circulating water pumps near the condenser. Water then enters the pump suction through a gravity canal or tunnel.

Figure 10 illustrates the recirculating pressure system with cooling tower. The cooling water starts its circuit from the cooling-tower basin by passing through the fine screens into the suction chamber and then into the circulating pump suction. The water is discharged by the circulating pump into the tunnel or piping leading to the condenser. The water passes through the condenser and out through the discharge tunnel or piping and discharges into the top of the cooling tower. Depending on plant area conditions, the circulating pump suction chamber may be located near the condenser with the water flowing by gravity from the cooling-tower basin.

Fundamentals of Design. In the design and layout of circulating-water systems, certain precautions must be observed in order to assure system reliability. The more important of these are discussed below.

In once-through systems, proper relative location of intake and discharge points has to be determined so that warm discharge water does not find its way into the system intake. Currents, eddies, or tides may carry the warm water upstream into the intake. Special means may be necessary to prevent this in some such form as a low dam, piping, canal, levee, etc In order to obtain the coldest water from the lowest stratum, the inlet may be lower than the screen and pump suction of the intake structure Each case has to be studied for its special conditions.

In recirculating systems with cooling towers, the location of the cooling tower has to be carefully studied so that the tower vapor cloud will disperse before reaching the power-plant area proper or creating a nuisance over adjoining facilities or properties.

The length of the system should be as short as practicable for the lowest pressure drop and pumping and installation costs and to decrease water hammer.

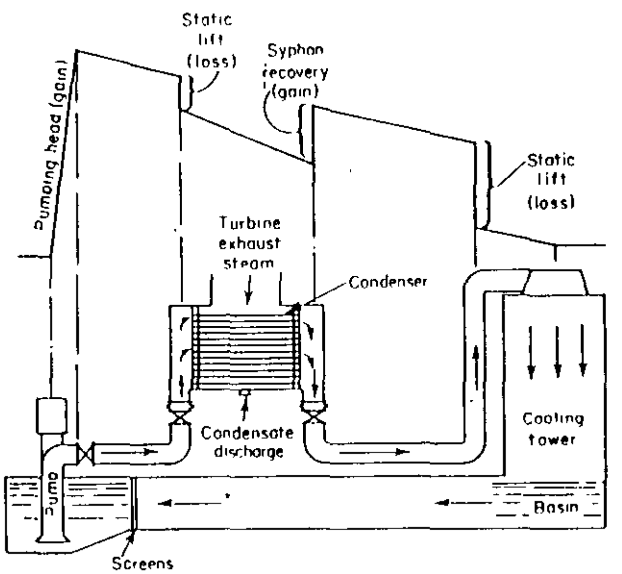


FIG. 10. Recirculating pressure system using cooling tower.

System elevations and profile should be set so as to obtain maximum siphon benefit and to eliminate or reduce the problems of air binding.

Items such as bends, tees, valves, reducers, and obstructions to smooth flow, all of which cause large pressure drops, should be used sparingly. Where these are necessary parts of the system, they should be designed for minimum pressure drop.

Preliminary Design, Flow Quantities. The following represent requirements averaged from actual installations for purposes of preliminary water-requirement estimates. There are many factors such as seasonal temperature of the water source, design of the actual condenser, and steam conditions of the thermal cycle that will create sharp deviations from these averages.

Once-through siphon system with a single-pass condenser
For 66,000 kw and below: 850 gpm per 1,000 kw
For 66,000 to 100,000 kw: 800 gpm per 1,000 kw
For 125,000 kw and above: 750 gpm per 1,000 kw

Once-through siphon system with a two-pass condenser
For 100,000 kw and below: 780 gpm per 1,000 kw
Fo 000 kw and above: 650 gpm per 1,000 kw

Recirculating pressure system, cooling tower, single-pass condenser
For 100,000 kw and above: 750 gpm per 1,000 kw

Recirculating pressure system, cooling tower, two-pass condenser
For 66,000 kw and below: 850 gpm per 1,000 kw
For 100,000 kw and above: 650 gpm per 1,000 kw

Preliminary Design, Pipe Sizing. For preliminary sizing of the circulating piping, a velocity of about 6 fps in the suction conduit to the pump and a velocity of about 8 to 10 fps in the discharge piping may be used. The final size selection must be made based on pressure drop. This final size selection entails an economic analysis, for the particular plant, of the cost of pumping the quantity required with various pressure drops, pumps, and pipe sizes Sometimes the problem of future plant expansion enters into this sizing.

The conditions that determine the piping wall thickness from pump discharge through the shutoff valve after the condenser for siphon systems, are the normal operating pressure, the pump shutoff head, the water-hammer surge pressure, and the external pressure on pipe due to burying conditions.

At pump and condenser discharges, the velocity should be lowered gradually by means of an expanding tube or diffuser with 14-deg or less total flare. At changes in size from pipe to tunnel or from tunnel to pipe, the use of gradual size change is advisable.

The pressure drop through bends can be reduced by proper spacing and specification. For example, one 90-deg bend has a lower pressure drop than that of two 45-deg bends. Spacing bends far enough apart may save pressure drop. For example, the total head loss in closely located bends may be twice that of the same bends spaced farther apart. Turning vanes properly used may cut bend losses considerably.

Intakes. Intakes from rivers are designed to bring water from the river with as small an entrance loss and disturbance as possible. Ordinarily, the cleaning screens and the circulating pumps are housed at the river's edge. Sometimes the pumps are at the plant and the water is brought from the cleaning screens by means of a gravity canal or conduit to the pumps This design saves pumping power and should be considered when the contour elevation is favorable from the river to the plant Allowance should be made for the head drop in the gravity canal and the river-level fluctuation. Also, consideration has to be given to possible contamination by leaves, branches and other debris between the screens and the pumps, and the necessity for auxiliary screening.

The intake structure usually consists of coarse screens (or trash racks), a stop log dam, traveling screens, screen wash pumps, and circulating-water pumps. There is frequently a gantry or bridge crane. Where large quantities of debris may be expected that will not be washed away by the river, power-operated raking may be necessary with means for disposing of the debris.

The smaller trash or foreign matter that passes through the coarse screens is picked up on the revolving screen. The usual speed of this screen is 10 fpm but may be of higher velocity for unusual trash conditions. These screen openings are made to sift out foreign matter from the water of size about one-half the diameter of the condenser tubes. The water velocity through the screen is kept below 2½ fps, since the rate of screen clogging rises rapidly with velocity. The amount of screen clogging is indicated by the difference of level on the two sides of the screen This can be measured with an air-bubbler system or with float gages. Generally, the screen wash pumps are started automatically when this difference is about 6 in When the screen-wash pump discharge pressure reaches a predetermined value, the screens are started automatically. The screen-wash pump requirement may run from 25 gpm at 40 psig to 35 gpm at 80 psig, depending on the trash characteristics of the river involved

Normally sluice gates or stop logs are provided before and after each traveling screen to permit its segregation and dewatering for repairs. Sometimes guides are provided for two sets of backup (or fine) screens after the traveling screens for the purpose of screening the water in event of breakdown and repair of the traveling screens. During such repair periods the fine screens are raised and cleaned alternately.

Consideration should be given to other problems that arise at the intake. Ice formation on the screens in winter may require recirculation of some of the warm discharge water. Various means may have to be considered to prevent silting, such as a settling area. The problem of obtaining the coldest layer of water requires a study of the relative location of intake and discharge.

The *intake structure design* involves the design of the suction chamber, and this in turn is governed by the selection of the pump, which may be of either the horizontal or vertical type.

The *horizontal pump* may not be practical where there is a large differential between high- and low-water river level. The horizontal pump must be set in a dry compartment, and its center line should be not more than 13 ft above the low-water level. The horizontal pump requires more space than a vertical pump and is sometimes more costly. A shutoff valve is frequently needed on the inlet side to cut off the water source in order to service the pump. However, the horizontal pump is more easily serviced and it is less sensitive to poor intake conditions. The horizontal pump has a lower starting horsepower and shutoff head and is usually a single- or double-suction single-stage, centrifugal-type pump.

The *vertical pump*, set in the water intake chamber, is more suitable for large river-level variations. The vertical pump adapts to a wider range of pump heads, requires a minimum of floor area, and is sometimes less costly than the comparable horizontal pump However, the vertical pump is sensitive to inlet conditions, generally has a higher shutoff head than a horizontal pump, and is more difficult to service. The vertical centrifugal pump generally used is a radial-flow, axial-flow, or a mixed-flow type. The radial-flow vertical pump has a bottom suction, and the discharge is on the horizontal center line of the hollow impeller: The axial-flow (or propeller) type of vertical pump has a bottom suction, and the water is raised vertically by the propeller and discharges at some specified elevation above the propeller. The mixed-flow vertical pump utilizes a hollow impeller of the radial type, but the water rises vertically in the pump casing before it is discharged.

The complexity of the intake structure is naturally affected by the number of circulating pumps necessary for the circulating-water system. Reliability points to the use of at least two pumps. The particular conditions for the plant will dictate the final choice, whether it will be two pumps at two-thirds capacity each, at one-half capacity each, or some other more favorable percentage each. The capacity selection has to be the subject of a careful analysis, taking into account such items as the condenser water requirement, the variation of pumping head, and the best efficiency range of the pumps

The water levels should be determined at both the suction and discharge ends of the circulating-water system. The levels of significance are those at high water, mean water, low water and at extreme low water. In addition, the coincidence of water level with the temperature of water and generating unit loads, especially at peak loads, should be analyzed. The calculation of the pump design operating point would be based on the maximum requirement of the condensing unit for sustained full load with the coincident water level and temperature

It is desirable to know the approximate operating points for several conditions

other than the design point in order to establish the capacity and head limits at which the pump will be expected to operate. Preliminary operating points can be established from assumed pump curves. There may possibly be operating conditions which cannot be permitted because the operating point falls outside the safe operating range of the pump selected. The condition for starting the system siphon must be part of this analysis

The intake piping to the suction of horizontal pumps should be designed so as to avoid air pockets Also, the water-flow velocity should be made uniform over the suction inlet area by placing bends as far as possible from the pump inlet.

The intake chambers for vertical pumps require careful design for good pump operation. The design must bring about a uniform and undisturbed flow of water to the pump without whirl. Most pump manufacturers have design suggestions for intake chambers for their particular pumps Each vertical-pump installation should be studied individually. There are no standard solutions to vertical-pump intake problems.

Discharge Structure. On the discharge end of the circulating-water system, an underwater (or sealed) discharge must be provided to prevent entry of air into the piping, which would otherwise break the siphon action at the condenser. One means of providing this seal is through the use of a seal well, that is, a basin with a water level controlled by an overflow weir. The seal-well water level regulates the height of the siphon recovery, and it is the final elevation to which the circulating pump delivers the water.

Beyond the seal well, the discharge into the river or other body of water must be done in such a way that the discharge velocity is dissipated without washing away banks, tearing up the bottom, undermining the discharge piping, or permitting uncontrolled recirculation to the intake

Air Binding. Air which accumulates in water piping can reduce the effective area for water flow and thus increase pumping cost through the resulting extra head loss

Air enters the piping system from several sources such as release of air from the water, air carried in through vortices into suction of pump, air leaking in through joints that may be under negative pressure, and the air originally present in the piping system before filling.

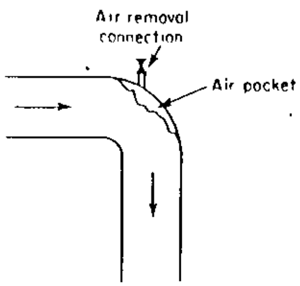
The water from the water source may be nearly completely saturated with air. If the temperature of this water is raised, for example, after passing through a condenser, and the pressure is lowered by the siphon action, the water will release most of its air. However, this air release is not instantaneous but proceeds on a time-rate release and is therefore dependent upon the length of time the water remains in the piping. Experience indicates that the actual air release in a circulating system of a conventional generating plant is probably in the order of 10 per cent of calculated theoretical release .

The problem associated with the formation of air pockets in the piping is dependent on the velocity of the water and on the configuration of the piping. The higher the water velocity, the less air binding will occur for the same piping configuration. The water with higher velocity has greater tendency to scrub out the air pockets, and its higher turbulence breaks up large air bubbles and entrains the resulting smaller bubbles.

On gentle downward slopes, a continuous air pocket may form along the top of the pipe for the entire slope In a sharper downward slope, several air pockets may form, each air pocket terminating in a hydraulic jump. Slopes may require a water velocity in excess of 10 fps to assure that the piping remains free of air. In a 90-deg downward drop, an air pocket may form in the upper portion of the bend and a velocity in excess of 7 fps may be required for air elimination Connections for air vents, as shown in Figs. 11 and 12, should be considered at the beginning of the slope and down along the slope.

Air Inlet Valves. In rolling or mountainous country, air and vacuum valves should be placed at the summits of water-supply lines to allow air to escape when they are being filled as well as to admit air to prevent forming a vacuum when they are being drained. It is good practice to install such valves at abrupt breaks in grade, where they serve also to permit the escape of air while the line is operating under pressure, thus preventing the accumulation of air that is released from solution in the water.

Correct sizes of vacuum valves can be determined only by computation of the probable rate of water drainage from the line, which depends in turn on its profile



Pipe with 90° drop

Fig. 11

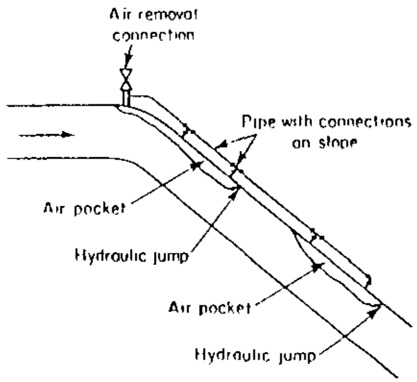


Fig. 12

and on the rate at which air is admitted to break the vacuum. The failure of the pipeline at a low point, for example, may empty part of the line and create a vacuum condition in the section of piping where there is a separation in the water column. The size of the vacuum valve should be computed on the supposition that air must enter at the same volumetric rate as that at which water leaves.

The strength of the pipe shell under this differential pressure will depend on the length-to-diameter ratio, the thickness-to-diameter ratio, and the modulus of elasticity of the pipe material. The presence of stiffener rings may exert a considerable influence on the strength of the pipe shell. However, if the distance between stiffener rings is great, they will have no appreciable effect on the section of the pipe shell midway between them, and the collapsing pressure will be the same as that of a pipe shell of infinite length. Parmakian[50] gives the following procedure for this calculation:

For cylindrical shells without stiffener rings:

$$P' = (50.2)(10^6)(t/D)^3$$

For cylindrical shells with stiffener rings:

$$P' = (73.4)(10^6)(t/D)^{5/2}(D/L)$$

where P' = collapsing pressure for pipe shell, psi
t = thickness, in.
D = inside diameter, in.
L = spacing of stiffener rings, in.

Pressure head at air inlet valve = $2.31(P_2 - P_1)$

where P_2 = [max]imum allowable internal pressure at air inlet valve, psi
P_1 = [bar]ometric pressure, psi

Cubic feet air per second that must be supplied:

$$Q = Wv = \pi D^2 V/(4)(144)$$

where Q = air that must be supplied at pressure P_2, cfs
W = weight of air flowing, lb/sec
v = specific volume of air at pressure P_2, cu ft/lb
$V = V_1 - V_2$ or $V = V_1 + V_2$

where V_1 = water velocity leaving location of air inlet valve computed for condition by Bernoulli energy equation, fps
V_2 = water velocity approaching location of air inlet valve, fps

If V_1 is larger than V_2, or if V_1 and V_2 are in opposite directions, an air inlet valve is required to supply enough air to the pipe interior to occupy the space created by breaking of the water column.

The air flow through the valve may be considered as an isentropic expansion, and the valve sized accordingly.

WATER HAMMER

An understanding of the physical picture of the water-hammer phenomenon can be helpful in dealing with the particular piping system under design.

The problem of water hammer in a pipeline consists of containing the pressure and dissipating the water flow energy. For example, in a pipeline with a supply pump, energy necessary to move the water through the piping is supplied by the pump. If a valve is suddenly closed at the end of the discharge line, the moving column of water is brought to a stop at the valve. The kinetic energy contained in the column of water, originally given to the water by the pump, is still present and must be dissipated. The column of water compresses, the pressure rises, and some of the kinetic energy is transformed to internal energy. The higher water pressure acts upon the pipe wall and does work in stretching it, but only a small percentage of energy will be lost in this. The pipe will obey the laws of vibration and return most of the energy to the water.

When the water-hammer pressure wave loads the pipe wall, the strain in the wall increases slightly faster than in strict proportion to stress within the elastic range, and on release of the loading, the reverse occurs. This results in a small area that is enclosed by the stress-strain curve in both directions, called the elastic hysteresis, and corresponds to a transformation of energy during the stress-strain cycle. If the pressure rise in the pipe is sufficient, the walls may be stressed into the plastic region and experience a permanent set. In this case the water gives up a large amount of energy with no recovery.

The energy or pressure wave which started at the closed valve stretches the diameter progressively to its source, the pump. The energy-wave front may meet a check valve at the pump and be reflected back toward its origin, continuing its dissipation of energy within the water and the pipe wall. This will continue until the kinetic energy is fully converted to internal energy.

When there is sudden failure of power at the pump motor, the only remaining energy is the kinetic energy of the rotating parts of pump and motor plus that of the entrained water. Since this is small when compared with that required to maintain flow against the discharge head, the reduction in pump speed is quite rapid. As the pump speed reduces, the flow of water in the discharge line at pump outlet is also reduced. Since this water has low energy, its pressure head is lower than that of water which was previously energized by the pump and delivered into the pipe water column. A depressuring action then occurs, starting at the pump and traveling up the pipeline.

Soon the pump speed is reduced to a point at which no water can be delivered against the existing discharge piping head. If there is no control valve present at the pump, the flow through the pump reverses, although the pump may be still rotating in a forward direction. The speed of the pump then drops more rapidly, stops, and reverses rotation. A short time later the pump, acting as a turbine driven by the higher head in the discharge line, reaches runaway speed in reverse. As the pump approaches runaway speed, the reverse flow through the pump reduces rapidly, so that the remaining energy in the water produces a pressure rise at the pump and reflects back along the length of the discharge line.

In order to analyze this problem, the items which have to be considered are the pump and motor inertia, the pump characteristics, and the water-hammer wave in the discharge line. The kinetic energy of the rotating system of the pump and motor is obtained from the inertia equation. The complete pump characteristic curves should be specified and obtained from the pump manufacturer. These curves should define the manner in which pump torque and speed vary with head and discharge flow throughout the range of operation as a pump, energy dissipator, and turbine.

Table 2. Water-hammer Velocity in Piping Systems

(a, wave velocity in feet per second)

D/t	Steel ($E = 28 \times 10^6$ psi)	Cast iron ($E = 16 \times 10^6$ psi)	Transite ($E = 3.4 \times 10^6$ psi)
20	4,300	4,100	3,000
40	4,000	3,600	2,300
60	3,800	3,350	2,000
80	3,600	3,100	1,750
100	3,400	2,900	1,600
150	3,100	2,500	1,300
200	2,800	2,250	1,150
250	2,600	2,050	
300	2,400		

The water-hammer effects are obtained from equations that define relations between head and flow in the discharge line during the transient flow condition which results from water-hammer wave action.

Water-column separation might occur at high points near the hydraulic gradient on long discharge lines. This condition can create high-pressure conditions at the moment of rejoining of the separated water columns.

When a check valve is to be used on the discharge side of the pump, its design and closing characteristics should be carefully considered. The discharge flow from the pump keeps the check valve open. But if pump power failure occurs, the check valve closes as reverse flow develops in the discharge line. The water-hammer analysis has to consider the partial reverse flow condition until the check valve closes completely and, after that point, the effect of the closed valve.

Table 2 gives the water-hammer wave velocity as a function of diameter-to-thickness ratios for three different piping materials encountered frequently in water-supply or -distribution systems. In this tabulation, a is the wave velocity in feet per second, D/t is the dimensionless ratio of diameter to thickness, and E is the elastic modulus.

If a valve is closed in the time of one wave cycle, (in the time a pressure wave goes to other end of pipeline and returns to the closing valve) or less, then the water hammer should be calculated on basis of instant valve closure.

To determine time for wave cycle, use

$$T = 2L/a$$

To determine water hammer for instantaneous valve closing, use

$$h = aV/g$$

where T = time for one wave cycle, sec
L = pipeline length, ft
h = water-hammer head above static head, ft
a = velocity of pressure wave, fps
V = water velocity at instant before valve closure, fps
g = 32.2 ft/sec²

To determine water hammer for slower valve closing, use

$$h_2 - h_1 = a(V_1 - V_2)/g$$

where h_2 = pressure after partial closing of valve, ft
h_1 = pressure before start of valve closing, ft
$h_2 - h_1$ = pressure rise due to water hammer, ft
V_1 = water velocity before start of valve closing, fps
V_2 = water velocity after partial closing of valve, fps

CORROSION

All water coming from wells, rivers, lakes, and ocean is an extremely dilute water solution of mineral salts and gases. The salts are mineral matter dissolved by water flowing over and through the earth layers. The salts are mainly sulfates, bicarbonates, chlorides of calcium, sodium, and magnesium. These minerals give water its hardness (destroying soap and preventing lather) and precipitate as a white lime-type scale. The dissolved gases are atmospheric oxygen and carbon dioxide, picked up by water-atmosphere contact, e.g., spray, raindrops, and ammonia from decaying vegetable matter.

The dissolved gases are the prime agents of chemical corrosion which act on the metals of piping systems. The oxygen attacks the iron or steel, and the process is accelerated by the carbon dioxide. The rate and extent of the chemical corrosion are influenced by the mineral salts dissolved in the water.

The water is corrosive when the Langelier index (calcium carbonate saturation index) is minus (−). The water analysis will generally, but not always, show a pH value below 7 (acidic).

The water is scaling when the Langelier index is plus (+). The water analysis will generally show a pH value above 7 (basic).

The Langelier index is obtained by subtracting the pH that the water must have at a particular temperature in order for calcium carbonate precipitation to start from the actual pH of the water. For example:

(pH actual) − (pH saturation) = Langelier index
(6.5) − (7.7) = −1.2 and the water is corrosive
(8.1) − (7.2) = +0.9 and the water is scaling

The precipitation of calcium carbonate as a scale or film thickness may be desirable as a means of protection against corrosion if the rate of build-up will be sufficiently low and allowance is made in diameter of the piping. But this calcium carbonate film would be undesirable on heat-transfer surfaces. Since temperature lowers the solubility of calcium carbonate and calcium sulfate, the Langelier index would be different for colder water coming to the condenser, for the water passing through condenser tubes, and for the warm water flowing away from the condenser. Therefore, the scaling tendency of the same water would be higher in the condenser and in the discharge piping from the condenser.

The *exterior of the buried pipe* will be subject to similar but aggravated chemical action due to water. In addition, the pipe exterior will be susceptible to attack by aerobic and anaerobic bacteria, galvanic action, and stray electric currents. The chemical action on the pipe exterior may be more intense because of concentration of oxygen, salts, and other chemicals leached out of the surrounding earth by ground water.

Some forms of anaerobic bacteria that thrive only in the absence of free oxygen obtain their oxygen by the chemical breakdown of oxygen compounds in the earth with the resultant production of substances, such as hydrogen sulfide, that will corrode the buried pipeline. There are also many types of aerobic bacteria that produce sulfuric acid, sulfate, ferric hydroxide, etc., all corrosive to steel or iron. Organic soil should be kept away from the vicinity of the pipeline to minimize possibility of this corrosion action.

Also, when iron or steel is in contact with a more cathodic material, for example copper or brass, a galvanic cell is set up, electrolysis results, and the corrosion rate of steel or iron rises. If iron or steel is in contact with a more anodic material, for example zinc, the zinc is the affected material and the corrosion rate of the steel or iron drops.

There is some natural resistance to chemical corrosion of the pipe materials in an uncoated state. The chemical-corrosion product, for example an oxide film, may build up sufficiently to slow down or prevent further corrosion.

The materials most commonly used for pipelines are steel, wrought iron, cast iron, concrete, and asbestos cement (transite). The natural coating characteristics of these materials are mentioned briefly below.

In *cast iron*, the rust (an iron oxide) builds up into a strong adhesive coating that finally forms a barrier sufficient to stop or slow down further corrosion. The higher silicon cast iron has the best characteristics in this respect.

In *wrought (or wrot) iron*, the surface rust is not so adhesive but the silicate fiber interlacing has a reinforcing effect, and after some penetration the barrier against further corrosion will tend to establish itself.

In *steel*, the rust powders and flakes off easily and will not build up into an adhesive and sufficiently protective coating.

In *concrete and asbestos-cement piping*, the corrosion is of a different form. These materials are subject to leaching of the free lime from the cement, deterioration in alkali soils, and attack by organic growth.

Cathodic Protection. If no protective coating is used, or if a low-cost, limited-life coating has been selected, cathodic protection should be considered as a means of limiting the main agent of corrosion, which is the electrochemical process. In this process, the moist earth is the electrolyte, two dissimilar materials are the anode and the cathode, and the pipe wall between completes the electric circuit. This process may be set in motion in a number of different ways, among which are dissimilar metals, galvanic action of a single metal due to dissimilar soils, variation in moisture and chemical content of soil, nonuniformity of metal caused by mill scale, surface scarring, welding, and even temperature differentials.

The current flows from the anode to the cathode and produces corrosion at a rate greater than that which would occur by normal chemical means. Corrosion is increased at the anode end and decreased at the cathode end. The anode is the point or area at which the current leaves the metal, and the cathode is the point at which the current enters the metal. The loss of metal at the anode end is in the range of 20 lb/amp per year.

The electrochemical galvanic series (Table 3) gives the relation between any two metals. The metal listed nearer the top of the table will be the anode that will waste away. T" metal nearer the bottom of the table will be the cathode and will be protected. The farther apart the metals are in the table, the greater the potential difference will be between them and the greater the corrosion rate of the anode end.

A typical example of the galvanic action of dissimilar metals is represented by the condenser with its steel shell, steel tube sheets, and its copper-alloy tubes. The steel is nearer the anode end than is the copper alloy, and as a consequence the corrosion of the steel tube sheets and shell is accelerated. Always, that metal which is higher in the galvanic series will waste away.

Cathodic protection is a means of diverting the electrochemical corrosion from the pipeline to wasting anodes.

There are two ways of providing cathodic protection. The least costly installation is the galvanic method based on the natural battery action between position of metals in the electrochemical table. An anode or wasting piece is deliberately used. This approach requires very careful analysis of all the varying conditions involved.

Table 3. Galvanic Series

Anode end (least noble, the wasting end)

Magnesium	Nickel (active)
Magnesium alloys	Brasses
Zinc	Copper
Aluminum	Bronzes
Aluminum alloys	Nickel-copper alloys
Cadmium	Nickel (passive)
Carbon steel	Stainless steel (passive)
Cast iron	Titanium
Stainless steel (active)	Silver solder
Soft solder	Silver
Tin	Graphite
Lead	Gold
	Platinum

Cathode end (most noble, the protected end)

The more costly cathodic protection is the impressed current method that requires an external source of electricity. The impressed current renders the piping cathodic to the surrounding soil by a controlled difference of potential.

In addition, in locations where there may be stray currents, the installation of removal wires at designated points so that the current may leave the pipeline should be considered. Or in other words, stray currents are utilized to provide cathodic protection for the pipeline.

Protective Coatings. Since corrosion of metal is a surface reaction, it is obvious that, if a protective coating which is continuous, impervious, chemically inert, and electrically insulating can be bonded to the interior and exterior of the piping, corrosion cannot take place in the pipe wall as long as the protective coating remains in place undamaged and without cracks or pinholes.

The basis of selection for the best coating differs somewhat for the interior and exterior of the pipe.

The coating on the interior of the pipe to perform its function properly would be selected for its chemical inertness, imperviousness, adhesiveness, adjustment to pipe deformation, and resistance to erosion of the flowing water.

The coating on the exterior of the pipe would be selected for its chemical inertness, electrical resistance, imperviousness, adhesiveness, adjustment to pipe deformation, and resistance to shear and compression due to varying earth conditions.

Galvanizing. The zinc used for galvanizing pipe is on the anodic (wasting) or electrochemical protective side of the steel, and it is wasted or changed to zinc compounds before the steel pipe will be attacked. Lead coating on pipe exteriors is not desirable for underground use because lead chemically corrodes in most soils and is cathodic in relation to iron and steel.

Coal-tar Enamel. Specification AWWA C203 covers the coal-tar enamel protective coatings for steel water pipe. This standard sets up the specifications for the materials involved, method of application to the inside and outside of the piping, the thickness required, protection of the coatings, testing, etc.

The type of enamel is specified as AWWA Coal-tar Enamel and is described in this standard with full characteristics and the ASTM tests required.

The 1961 "Paint Manual" of U.S. Department of Interior states:

Coal-tar enamel is especially appropriate for use on the interior surfaces of penstocks and outlet pipes. A long-life coating in penstocks is necessary because dewatering involves loss of revenue from electric power production. In outlet pipes where water flows at high velocities, there is special requirement for stability within the film, which is characteristic of coal-tar enamel. However, hand-applied coal-tar enamel coatings exhibiting drips and undue roughness from careless or inexperienced application, may be damaged by the high velocities. Therefore, for such exposure, excellent workmanship is mandatory. The glasslike surface obtained by the spinning process is especially suitable for this service. Except as limited by certain considerations discussed in the following paragraphs, coal-tar enamel is considered an excellent coating for the interior surfaces of all steel pipe and for the exterior surfaces of buried steel pipe.

The use of coal-tar enamel is limited by its susceptibility to damage from cold weather. The materials marketed are not expected to withstand temperatures under (-) 20 F without danger of cracking and disbonding. Although cases can be cited where they have stood up under such conditions, enamel is not usually considered a good risk where it will be subjected to such extreme cold before or after installation of the coated parts. Enamel coatings are adversely affected by prolonged pre-installation exposure in the open. Continued heat developed when coated or lined pipe is exposed to warm weather and sunlight without the moderating effect of water within the pipe induces hardening and loss of plasticity in the enamel. This embrittlement may be the cause of later damage when the pipe is handled. A heat-reflective coating of white-wash, red lead or aluminum paint is helpful on the outside of the pipe; but even so, prolonged exposure before installation is undesirable.

Although more expensive than shop application, coal-tar enamel is usually applied to the interior of large diameter pipe after installation for several reasons. Most shop lining equipment will not handle pipes much over 10 feet in diameter. Field application avoids pre-installation exposure and installation damage to the coating. If enamel is to be used for lining pipe that is too small to work in, the enamel may be applied in the field by portable centrifugal lining equipment just prior to installation. In practice, this is rarely done, however. Especially where the amount of pipe to be lined is small, the expense of setting up the field equipment is not warranted and shop coating is generally employed if prolonged pre-installation exposure can be avoided. Enamel should not be used for the interior of pipe under 27 inches in diameter where pipe sections must be joined by welding, because of the difficulty or impossibility of coating interior joints after assembly. This is not a consideration where sections are joined by mechanical couplings.

For buried steel pipe, AWWA specifications suggest a variety of exterior coating systems employing enamel, the choice of system depending on soil conditions. Soil forces which are destructive to coatings are caused by wetting and drying of the soil, rocks in the backfill, and chemical constituents. Earth often adheres tightly to coating materials and, on drying, exerts powerful stresses which shear and tear the coating from a pipe surface. The soil properties of liquid limit and plasticity index reflect the soil capacity to create such stresses.

Glass mat has an open structure which permits the hot enamel to penetrate and pass through the mat so as to embed it, producing a reinforcing effect much like that of steel in concrete. This makes it an excellent covering for pipe under severe soil conditions. When asbestos felt is covered by hot enamel, there is a tendency to volatilize the felt saturant and create vapor pockets in the coating, and thus weaken the coating. Asbestos felt is therefore not suitable as an inner wrapping. However, it is desirable as an outer wrapping for buried pipe, because of its value as a shield against soil stress and moderate deformation forces from rocks or clods in backfill material. Even in the rare instances when sandy, well-drained soil would permit elimination of outer wraps, use of asbestos felt can be justified since it reduces handling damage. For large diameter pipe, where the backfill

contains much rock or where highly corrosive conditions exist, pneumatically placed mortar is often used as a shield over the enamel coating.

Mechanical couplings are coated with coal-tar enamel and usually surrounded with a sand shield using No. 16 maximum size sand; or a double wrap of a glass-reinforced coal-tar enamel tape is sometimes used in lieu of the sand shield.

Supplemental protection can be provided for exterior surfaces of buried steel pipe by installing cathodic protection. This is not usually essential at the time the pipe is installed, because potential measurements between soil and pipe can be made subsequently to determine how much, if any, additional protection is needed. However, it is important that the pipe sections be connected or "bonded" electrically at the time of installation if cathodic protection is contemplated at a later date.

Hot-applied Coal-tar tapes—Glass-reinforced coal-tar tapes are used by the Bureau of Reclamation for the exterior coating of welded field joints and also for small quantities of small diameter steel pipe. This tape is applied with the aid of a wide-flame heating torch over properly primed steel surfaces.

Cold-applied Plastic Tapes—Spirally wound vinyl, polyethylene and polyvinyl chloride-butyl rubber tapes, in thickness of 10 to 20 mils, have been used for the protection of the exterior surfaces of straight sections of steel pipe and for the coating of welded joints of shop-coated pipe sections in the field. Difficulty has been experienced in obtaining water-tight seals at the lap when these tapes are applied over pipe fittings. The use of effective primers and a double wrap of the tape has been found to afford maximum protection. A felt wrap held in place by steel bands is desirable when puncture of the tapes may be anticipated, as indicated by the presence of sharp-edged rocks in the backfill.

Cement Mortar—Cement mortar is used extensively by the Bureau of Reclamation as a protective interior lining for steel pipe in new construction, and it has also been used in rehabilitating old pipe lines in place.

Asphalt Coatings.—An asphalt hot-dip coating is sometimes used in Bureau of Reclamation work for small diameter (under 24 inches) steel pipe and corrugated metal pipes in distribution systems. Asphalt is not generally considered to be as effective as coal-tar coating for protection of steel pipe against corrosion, but it is an inexpensive shop treatment not as susceptible to damage from exposure as coal-tar enamel.

The cold-applied plastic tapes are easily installed and have toughness, flexibility, inertness, dielectric properties, and resistance to bacterial action. However, these tapes lose their adhesiveness at temperatures below 34 F.

Cold-applied coatings such as asphalt emulsions, asphalt, or coal tar in volatile solvents have not to date given long enough or sufficient protection when applied to underground piping.

MATERIALS

The following publications and standards of the AWWA (American Water Works Association) contain the complete material specification requirements for water piping:

Excavation, No. R612, Trench Excavation and Backfilling

Grounding, No. 270, Grounding of Electric Circuits on Water Pipes, Stray Current Problems

Hydraulic, No. R805, Water Hammer Allowances in Pipe Design

Plastic Pipe, Nos. R281 and R1015, Developments in Plastic Pipe

Steel Pipe, No. R601, New Developments in Tests of Coatings and Wrappings

Valves, No. 372, Selection of Valves for Water Works Service

American Standard for Vertical Turbine Pumps, A101-60 (ASA B58.1)

AWWA Standard for Cast-iron Pressure Fittings, C100-55

American Standard Practice Manual for the Computation of Strength and Thickness of Cast-iron Pipe, 111 (ASA A21.1).

American Standard for Cast-iron Pit Cast Pipe for Water or Other Liquids, C102-53 (ASA A21.2)

American Standard for Cement-mortar Lining for Cast Iron Pipe and Fittings, C104-53 (ASA A21 4)

American Standard for Cast Iron Pipe Centrifugally Cast in Metal Molds, for Water or Other Liquids, C106-53 (ASA A21.6)

American Standard for Cast Iron Pipe Centrifugally Cast in Sand-lined Molds, for Water or Other Liquids, C108-53 (ASA A21 8)

American Standard for Short-body Cast Iron Fittings, 3 Inch to 12 Inch for 250-psi Water Pressure Plus Water Hammer, C110-52 (ASA A21 10)

American Standard for a Mechanical Joint for Cast Iron Pressure Pipe and Fittings, C111-53 (ASA A21 11)

AWWA Standard for Fabricated Electrically Welded Steel Pipe, C201-60T

AWWA Standard for Mill-type Steel Water Pipe, C202-60T

AWWA Standard for Coal-tar Enamel Protective Coatings for Steel Water Pipe, C203-57

AWWA Standard for Cement-mortar Protective Coatings for Steel Water Pipe of Sizes 30 Inches and Over, C205-41

AWWA Standard for Field Welding of Steel Water Pipe Joints, C206-57

AWWA Standard for Steel Pipe Flanges, C207-55

AWWA Standard for Dimensions for Steel Water Pipe Fittings, C208-59

AWWA Standard for Reinforced Concrete Water Pipe—Steel Cylinder Type, Not Prestressed, C300-57

AWWA Standard for Reinforced Concrete Water Pipe—Steel Cylinder Type, Prestressed, C301-58

AWWA Standard for Reinforced Concrete Water Pipe—Noncylinder Type, Not Prestressed, C302-57

AWWA Standard for Asbestos-cement Water Pipe, C400-53T

AWWA Standard for Gate Valves for Ordinary Water Works Service, C500-61

AWWA Standard for Rubber Seated Butterfly Valves, C504-58

AWWA Standard for Metal-seated Butterfly Valves, C505-58

AWWA Standard for Installation of Cast-iron Water Mains, C600-54T

AWWA Standard for Disinfecting Water Mains, C601-54

AWWA Standard for Cement-mortar Lining of Water Pipelines in Place—Sizes 16 Inches and Over, C602-55

AWWA Standard for Steel Tanks, Standpipes, Reservoirs and Elevated Tanks for Water Storage, D100-59, D5.2-59, D102-55T

Table 4. List of Material Specifications

Material	Specification
Copper pipe	ASTM B42
Red brass pipe	ASTM B43
Cast iron, bell and spigot	FSB WW-P-421
Cast iron, pit cast	ASA A21 2
Cast iron, centrifugally cast in metal molds	ASA A21.6
Cast iron, centrifugally cast in sandlined molds	ASA A21 8
Welded wrought-iron pipe	ASTM A72
Welded and seamless steel pipe	ASTM A53
Seamless carbon-steel pipe	ASTM A106
Black and galvanized welded and seamless steel pipe	ASTM A120
Electric-fusion-welded steel pipe (30 in and over)	ASTM A134
Electric-resistance-welded steel pipe	ASTM A135
Electric-fusion-welded steel pipe (4 to 30 in.)	ASTM A139
Seamless and welded austenitic stainless steel pipe	ASTM A312
Spiral-welded steel or iron pipe	ASTM A211
pipe	API 5L

Table 5. Properties of Large-diameter Water-supply Piping

ID, in.	Thickness, in	Pipe weight per foot bare, based on 5% overweight lb	Water contents per foot*	Moment of inertia about pipe axis, in.⁴	Section modulus, in.³	Radius of gyration, in.	Maximum safe span unstiffened pipe, simply supported 120 deg bearing, stress = 5,000 psi, ft
30	3/16	65	306 lb 36 72 gal	2,025	133	10 673	34
	1/4	87		2,717	178	10 695	38
	5/16	108		3,418	223	10 718	41
	3/8	130		4,127	268	10 740	44
	7/16	152		4,845	313	10.762	47
	1/2	173		5,572	359	10 785	50
36	3/16	78	441 lb 52 88 gal	3,489	191	12 794	34
	1/4	104		4,676	256	12 817	39
	5/16	130		5,876	320	12 839	42
	3/8	156		7,088	385	12 861	45
	7/16	182		8,312	450	12.884	48
	1/2	208		9,549	516	12.906	51
	9/16	233		10,799	581	12.928	53
	5/8	259		12,061	647	12.951	54
42	3/16	90	600 lb 71 97 gal	5,528	260	14.916	35
	1/4	120		7,404	348	14.938	40
	5/16	150		9,296	436	14 960	43
	3/8	180		11,206	524	14 982	46
	7/16	210		13,132	612	15 005	50
	1/2	240		15,074	701	15 027	52
	9/16	270		17,034	790	15 049	54
	5/8	300		19,011	879	15 072	55
48	1/4	134	784 lb 94 gal	11,028	454	17 059	40
	5/16	167		13,838	569	17 081	43
	3/8	201		16,671	683	17.104	46
	7/16	234		19,526	799	17 126	50
	1/2	268		22,402	914	17 148	52
	9/16	302		25,301	1,030	17 171	55
	5/8	335		28,222	1,146	17 193	57
	11/16	369		31,165	1,262	17.215	59
	3/4	402		34,130	1,379	17.238	62
54	1/4	153	992 lb 119 gal	15,675	575	19.180	40
	5/16	191		19,661	719	19.203	44
	3/8	230		23,676	864	19.225	47
	7/16	267		27,718	1,010	19.247	51
	1/2	306		31,787	1,155	19.269	53
	9/16	344		35,884	1,301	19 292	56
	5/8	383		40,009	1,448	19 314	58
	11/16	420		44,163	1,595	19 336	60
	3/4	459		48,345	1,742	19 359	63

* 1 U S gallon at 60 F = 8 337 lb

Table 5. (Continued)

ID, in.	Thickness, in.	Pipe weight per foot bare, based on 5% over-weight lb	Water contents per foot*	Moment of inertia about pipe axis, in.⁴	Section modulus, in.³	Radius of gyration, in	Maximum safe span unstiffened pipe, simply supported 120 deg bearing, stress = 5,000 psi, ft
	1/4	170		21,472	709	21.302	40
	5/16	212		26,923	888	21.324	44
	3/8	255		32,409	1,066	21.346	48
	7/16	297		37,929	1,246	21.368	51
60	1/2	340	1,225 lb 146.88 gal	43,483	1,425	21.391	54
	9/16	383		49,071	1,605	21.413	57
	5/8	425		54,693	1,785	21.435	59
	11/16	468		60,350	1,966	21.458	61
	3/4	510		66,042	2,147	21.480	64
	1/4	187		28,547	858	23.423	40
	5/16	234		35,786	1,074	23.445	44
	3/8	280		43,064	1,290	23.468	48
	7/16	327		50,384	1,506	23.490	51
	1/2	374		57,745	1,723	23.512	54
	9/16	420		65,147	1,941	23.534	57
66	5/8	466	1,482 lb 177.72 gal	72,591	2,158	23.557	59
	11/16	513		80,077	2,377	23.579	62
	3/4	560		87,604	2,595	23.601	64
	13/16	607		95,174	2,814	23.624	66
	7/8	654		102,785	3,034	23.646	67
	15/16	701		110,439	3,254	23.668	69
	1	748		118,135	3,474	23.692	71
	1/4	204		37,027	1,021	25.544	41
	5/16	255		46,405	1,277	25.567	45
	3/8	306		55,831	1,534	25.589	49
	7/16	357		65,304	1,792	25.611	52
	1/2	409		74,828	2,050	25.633	55
	9/16	459		84,400	2,308	25.655	58
72	5/8	510	1,764 lb 211.5 gal	94,022	2,567	25.678	60
	11/16	561		103,693	2,826	25.700	62
	3/4	612		113,414	3,086	25.722	65
	13/16	663		123,183	3,346	25.744	67
	7/8	715		133,004	3,606	25.767	69
	15/16	766		142,874	3,868	25.789	70
	1	818		152,794	4,129	25.812	72
	5/16	276		58,940	1,499	27.688	45
	3/8	332		70,898	1,800	27.710	49
	7/16	387		82,912	2,102	27.732	52
	1/2	442		94,986	2,404	27.755	55
	9/16	497		107,116	2,707	27.777	58
78	5/8	552	2,070 lb 248.2 gal	119,303	3,010	27.800	60
	11/16	607		131,547	3,314	27.821	63
	3/4	663		143,850	3,618	27.844	65
	13/16	718		156,211	3,923	27.866	67
	7/8	774		168,631	4,228	27.888	69
	15/16	830		181,110	4,534	27.911	71
	1	885		193,645	4,841	27.933	73

Table 5. (Continued)

ID, in.	Thickness, in.	Pipe weight per foot bare, based on 5% over-weight lb	Water contents per foot*	Moment of inertia about pipe axis, in.⁴	Section modulus, in.³	Radius of gyration, in.	Maximum safe span unstiffened pipe, simply supported 120 deg bearing, stress = 5,000 psi, ft
	5/16	300		73,552	1,738	29.809	45
	3/8	360		88,459	2,087	29.831	49
	7/16	420		103,431	2,437	29.854	53
	1/2	480		118,471	2,787	29.876	56
	9/16	540		133,577	3,138	29.898	59
84	5/8	600	2,401 lb 287.9 gal	148,749	3,489	29.920	61
	11/16	660		163,989	3,841	29.943	64
	3/4	720		179,298	4,194	29.965	66
	13/16	780		194,670	4,547	29.987	68
	7/8	840		210,111	4,900	30.009	70
	15/16	900		225,621	5,254	30.032	72
	1	960		241,198	5,609	30.054	73
	5/16	316		90,395	1,994	31.930	45
	3/8	379		108,702	2,395	31.953	49
	7/16	440		127,084	2,796	31.975	53
	1/2	504		145,540	3,198	31.997	56
	9/16	567		164,074	3,601	32.019	59
90	5/8	630	2,756 lb 330.5 gal	182,683	4,004	32.042	61
	11/16	693		201,371	4,407	32.064	64
	3/4	756		220,133	4,811	32.086	66
	13/16	819		238,975	5,216	32.108	68
	7/8	882		257,891	5,621	32.131	70
	15/16	945		276,887	6,027	32.153	72
	1	1,008		295,958	6,413	32.175	74
	5/16	334		109,636	2,269	34.052	46
	3/8	402		131,819	2,724	34.074	50
	7/16	468		154,092	3,181	34.096	53
	1/2	536		176,449	3,638	34.118	56
	9/16	604		198,891	4,095	34.141	59
96	5/8	670	3,136 lb 376 gal	221,423	4,553	34.163	62
	11/16	738		244,040	5,012	34.185	64
	3/4	804		266,746	5,471	34.207	67
	13/16	871		289,535	5,931	34.230	69
	7/8	938		312,417	6,392	34.252	71
	15/16	1,005		335,386	6,853	34.274	73
	1	1,072		358,441	7,315	34.297	75

For example, Specification AWWA C201-50 covers Electric Fusion Welded Steel Water Pipe of Sizes 30 Inches and Over and requires that steel plate shall conform to specifications for low- and intermediate-tensile-strength carbon-steel plates of structural quality, ASTM Designation A283, Grade B, of latest revision or as otherwise specified by the purchaser.

DESIGN FOR STRUCTURAL STRENGTH

Pipe-wall Thickness. The Code for Pressure Piping, ASA B31.1 specifies the following formula for determining the thickness of piping for water service of materials other than cast iron:

$$t_m = PD/(2S + 2yP) + C$$

where t_m = minimum pipe wall thickness, in.
P = maximum internal service pressure, psig
D = outside diameter of pipe, in.
S = allowable stress in material due to internal pressure, at the operating temperature, psi (see material listing in Code)
C = allowance for threading, grooving, mechanical strength, and/or corrosion, in.
y = 0.4 for ferritic steels and austenitic steels for temperatures to 900 F

The Pipe Fabrication Institute publishes a Technical Bulletin TB1, "Pressure Temperature Ratings of Plain End Pipe Used in Power Plant Piping Systems." This covers sizes ½ through 24 in. for pipe of carbon steel and alloys. The tables give thicknesses with their pressure ratings that are based on the Code for Pressure Piping.

Regarding cast iron, the code for Pressure Piping states:

> The thickness of cast iron pipe conveying liquids may be determined by selection from American Standards ASA A21 2, A21.6, or A21.8, or from Federal Specification WW-P-421, using the class of pipe for the pressure next higher than the maximum internal service pressure in pounds per square inch These thicknesses include allowances for foundry tolerances and water hammer. Where the thickness for liquid service is calculated, the methods of ASA A21 1 shall be followed

In selecting the proper cast-iron pipe, it is essential that the trench-laying conditions be known and specified. The AWWA gives four trench-laying conditions in AWWA Standard C101 (ASA A21.1) for water piping:

1. Flat-bottom trench, without blocks, untamped backfill
2. Flat-bottom trench, without blocks, tamped backfill
3. Pipe laid on blocks, untamped backfill
4. Pipe laid on blocks, tamped backfill

The piping thicknesses given in the AWWA Cast Iron Standards are for depths of cover 3½, 5, and 8 ft, and for working pressures of 50, 100, 150, 200, 250, 300, and 350 psi. The thicknesses include allowance for foundry practice, corrosion, and either water hammer or truck load. The AWWA Standard C101 (ASA A21.1) should be consulted for water-hammer and truck-loading allowances.

Loading on Pipe Due to Depth of Cover and Live Loads. Tables 6 and 7 provide a guide to the loading that may be expected for various depths of bury and live loads on the surface over the pipe The AWWA Standards provide tables of allowable loads for cast-iron and concrete piping. The allowable loads on buried steel pipes up to about 24 in. in diameter can be computed based on the ability of the pipe to sustain a vertical load by ring action alone. Pipes over 24 in. diameter, depending on wall thickness, will not carry heavy loads with ring action alone without undue deflection Side support from earth is required. But the earth on the sides of the pipe must be compacted to a high density, approaching the density defined in Specification ASTM D698, in order to provide a predictable amount of side support. Note that piping with more than 8 ft of cover is not appreciably affected by surface live loads. Table 8 is based on ring strength alone

Table 7 is convenient for a determination of the amount of pipe loading which might be received on a buried line owing to the load transmitted to it from an overhead moving vehicle. To illustrate the use of this table, the following example is given.

Table 6. Dead Load from Earth Cover on Underground Pipes

(Loads are shown in pounds per linear foot of pipe)

Depth of cover, ft	Nominal pipe diameter, in.																		
	3	4	6	8	10	12	15	18	21	24	27	30	33	36	39	42	48	54	60
2	145	180	240	290	340	390	450	500	560	610	700	750	820	875	940	1,000	1,140	1,280	1,380
3	220	270	370	460	550	630	750	860	950	1,040	1,120	1,200	1,300	1,400	1,480	1,580	1,740	1,970	2,080
4	300	370	520	650	780	920	1,080	1,230	1,400	1,520	1,630	1,750	1,850	2,000	2,100	2,220	2,500	2,730	2,980
5	380	470	660	830	1,000	1,160	1,420	1,610	1,810	2,010	2,200	2,340	2,500	2,630	2,800	2,950	3,250	3,600	3,820
6	460	570	800	1,000	1,200	1,430	1,710	2,000	2,230	2,500	2,700	2,950	3,180	3,350	3,500	3,650	4,030	4,420	4,700
7	540	670	950	1,180	1,420	1,700	2,050	2,400	2,700	3,050	3,300	3,570	3,900	4,100	4,300	4,440	4,900	5,450	5,780
8	620	780	1,080	1,370	1,620	1,960	2,400	2,780	3,200	3,550	3,900	4,200	4,500	4,800	5,050	5,300	5,900	6,430	6,880

These values apply to both rigid and flexible pipes buried in ditches or covered by embankment. They are based on maximum conditions of trench width and 120 lb/cu ft soil material, using the Marston formula, and should be used only as approximations. (See American Water Works Association Standard AWWA H1 for charts and tables based on full-scale tests for various conditions.)

Table 7. Percentage of Wheel Loads Transmitted to Underground Pipes

(Values show percentage of wheel load applied to one linear foot of pipe)

Depth of cover, ft	Nominal pipe diameter, in.																		
	3	4	6	8	10	12	15	18	21	24	27	30	33	36	39	42	48	54	60
1	15.2	18.6	25.6	30.0	34.6	40.0	45.2	49.6	52.8	54.4	56.0	57.2	58.0	58.8	59.6	59.7	60.0	60.3	60.6
2	7.0	8.5	11.4	14.0	16.6	19.2	23.0	26.4	30.0	31.2	33.6	35.6	37.4	39.0	40.0	40.9	42.6	43.6	44.6
3	3.5	4.0	5.8	7.2	8.6	10.4	12.8	15.0	17.2	18.6	20.4	22.2	23.6	25.0	25.8	26.0	26.6	28.0	29.4
4	2.0	2.4	3.4	4.2	5.0	6.2	7.8	9.2	10.6	11.6	13.0	14.4	15.8	17.0	17.6	17.7	18.0	19.7	21.4
5	1.4	1.8	2.4	2.8	3.4	4.2	5.2	6.2	7.2	7.8	8.8	9.8	10.6	11.6	12.2	12.4	12.7	14.0	15.3
6	0.9	1.2	1.6	2.0	2.2	2.8	3.6	4.2	5.0	5.6	6.2	7.0	7.6	8.4	8.6	8.8	9.3	10.7	12.0
7	0.3	0.5	1.0	1.4	1.6	2.0	2.6	3.2	3.8	4.2	4.6	5.2	5.8	6.4	6.5	6.6	6.7	7.7	8.7
8	0.2	0.4	0.8	1.0	1.2	1.6	2.0	2.4	2.8	3.2	3.6	4.0	4.4	4.6	5.0	5.1	5.3	6.0	6.7

The values include an impact factor of 2.0 and are based on "Underground Conduits—An Appraisal of Modern Research," M. G. Spangler, *ASCE Paper* 2337, 1947. The values apply to one vehicle with wheels at least 6 ft apart measured along the axle. The wheel load (as in the example) is ½ of the axle load. The wheel load may be on dual tires but is still considered one wheel. (See American Water Works Association Standard AWWA H1 for charts and tables based on full-scale tests for various conditions.)

Table 8. Allowable Loads for Steel Pipe

Nominal pipe diam, in	Wall thickness, in or schedule	Allowable load, lb/lin ft	Nominal pipe diam, in	Wall thickness, in, or schedule	Allowable load, lb/lin ft
4	0.237 Sch 40	10,000	20	0.250 Sch 10	1,900
5	0.258 Sch 40	10,000	20	0.375 Sch 20	6,250
6	0.280 Sch 40	10,000	20	0.500 Sch 30	14,600
8	0.250 Sch 20	8,300	24	0.250 Sch 10	1,250
8	0.322 Sch 40	9,900	24	0.375 Sch 20	4,250
10	0.250 Sch 20	7,300	24	0.500	10,000
10	0.307 Std	14,600	30	0.250	930
10	0.365 Sch 40	18,700	30	0.375	3,000
12	0.250 Sch 20	5,100	30	0.500 Sch 20	6,500
12	0.330 Std	13,700	36	0.250	850
12	0.375 Sch 40	17,500	36	0.375	2,250
14	0.250 Sch 10	3,800	36	0.500	4,900
14	0.312 Sch 20	7,500	42	0.375	1,750
14	0.375 Sch 30	13,100	42	0.500	3,900
16	0.250 Sch 10	3,000	48	0.500	3,000
16	0.312 Sch 20	5,800	48	0.625	5,500
16	0.375 Sch 30	10,000	54	0.500	2,800
18	0.250 Sch 10	2,440	54	0.625	4,500
18	0.375	7,700	60	0.500	2,500
18	0.437 Sch 30	12,700	60	0.625	4,000

Example. Find the total load per foot on a 30-in. pipe with 6 ft of cover and a 20,000-lb wheel load (i.e., one-half of a 40,000-lb axle load).

Dead load (from Table 6)	2,950 lb/ft
Live load (from Table 7) is 7.0 per cent of 20,000	1,400 lb/ft
Total load	4,350 lb/ft

These values are based on an approximate 2 per cent deflection. Data are taken from "Design and Deflection Control of Buried Steel Pipe Supporting Earth Loads and Live Loads," Russell E. Barnard, *Proc. ASTM*, vol. 57, 1957. The table applies to all types of steel pipe—welded, seamless, spiral welded—and to stainless or other steels which have a modulus of elasticity of about 30 × 10^6 psi, regardless of yield or ultimate strength, since deflection within the elastic limit is dependent on modulus of elasticity, not strength.

Anchors. For piping with joints such as bell and spigot on cast-iron pipe, bell and spigot with O-ring gasket, Dresser or Smith-Blair type on steel pipe, and any other type of pipe joint depending on friction or packing, anchorage has to be provided to prevent joint pull-out due to internal pressure.

AWWA C600, Standard Specifications for Installation of Cast-iron Water Mains, states in Sections 12.2 and 12.3:

All plugs, caps, tees and bends deflecting 22-½ degrees or more on mains 8 inches in diameter or larger shall be provided with a reaction backing, or movement shall be prevented by attaching suitable metal rods or clamps. . . .

Reaction backing shall be concrete . . . of not less than 2,000 psi at 28 days. Backing shall be placed between solid ground and the fitting to be anchored. . . .

Anchors should be considered at all direction changes and at dead ends of the piping. The anchors should bear on and against undisturbed soil. If backfill is ever used against the face of the anchor, it should be well compacted by wetting and tamping in thin layers. Any soil weakened by rain or snow or overexcavation beneath the anchor should be replaced by concrete. The resistance of the soil to movement of the anchor is a combination of passive resistance against the face of the anchor and the resistance to sliding along the base. Accordingly, values of cohesion and friction angle of the soil involved should be known or at least estimated intelligently.

For large and critical anchors, the soil properties should be determined by penetration and laboratory tests on undisturbed samples. The use of piles should be considered when the size and cost of the anchor are appreciable.

If there is the possibility of shock pressures due to water hammer, the anchors, should be designed to withstand these

Anchor loadings may be determined as follows: resultant pressure thrust on anchor = (internal pressure) × (largest internal pipe cross-section area) × (fitting factor).

Fitting	Factor
90-deg ell	1.41
Caps, plugs, tees	1.00
45-deg ell	0.77
22½-deg ell	0.39
11¼-deg ell	0.20

In addition, *centrifugal thrust* is present, but will be low for the usual water-line velocities

$$\text{Centrifugal thrust} = (2AWV^2/g)\sin(\theta/2)$$

where A = inside area of pipe, sq ft
W = density of fluid, lb/cu ft
V = velocity, fps
θ = change in direction
g = 32.2 ft/sec² gravity acceleration

Another important case where anchors must be considered for reasons other than joint design is that in which the temperature of the water flowing in the pipeline is appreciably higher than the temperature prevailing during construction of the pipeline.

If the temperature of the water is 200 F, for example, and the temperature of the water must be maintained, then the design of the piping would follow that of underground steam lines and would involve insulation, watertight protective conduits or tunnels, etc.

But if the requirements of design are such that the loss of heat from the water is not important, for example, waste drains from process, cooling water after heat pickup, atomic-plant waste, etc., it is economically desirable to treat the piping as cold-water piping. When this is done, the thermal expansion in the piping must be controlled or its effects dissipated.

Expansion joints of bellows or slip type, ball joints, or gasketed friction joints may be utilized to take up expansion, provided leakage and failure possibility are evaluated and found acceptable.

In between the anchored points, the expansion of solidly joined piping (welded) must be absorbed through either bending and torsion of cantilever legs or through bowing of the line. The possibility of suppressing the expansion and containing it in the pipeline through compressive stress is not very feasible. The force available for this is the starting friction between the earth and the pipe, and this is some percentage of the loading on the pipe. A portion of the expansion can be suppressed in this way through friction, but the remaining force can be very large and will appear at the anchor points.

The magnitude of the thermal thrust may be seen from the following example: The modulus of elasticity is the tangent of the stress-strain curve in the elastic range and is defined as

$$E = (F/a) \div (e/l)$$

and from this

$$F = Ea(e/l)$$

where F = force, lb
E = modulus of elasticity (for the pipe material at temperature), psi
a = metal cross-section area, sq in.
e = total expansion, in.
l = length, in.
e/l = expansion, in./in.

Expansion of carbon steel for temperature rise from 70 F to	Expansion in./in.
100 F	0.00018
150 F	0.00050
200 F	0.00083

For a pipe 24 in. OD by ½ in. thick with a thermal growth of 0.00083 in./in. for a temperature change from 70 F at installation to 200 F operating,

$E = 27.7 \times 10^6$ at 200 F   Metal area = 36.91 sq in.

$F = (27.7 \times 10^6)(36.91)(0.00083) = 843{,}500$ lb thermal force thrust

Even if it were possible to suppress this thermal force, the compressive stress in the pipe wall would be excessive, as can be seen by dividing the force by the cross-sectional area. In addition, the possibility of buckling due to column action would require investigation.

A feasible approach is to take care of the expansion by some form of bending of the pipe. The piping would have to be run in a manner to provide bending legs at right angles. For smaller temperature differentials, pipe lengths laid at a small-angle zig-zag might be sufficient.

In addition, screened sand should be used around the pipe to lower the sliding friction and provide adjustment area for the pipe. A polyvinyl chloride type of sheeting may be advisable to contain the sand area and prevent washout.

Expansion during Construction. Water piping is normally not designed for expansion, so that expansion of piping during construction may be easily overlooked.

The pipeline is joined together in a continuous string in a trench. The piping expands under the direct heat and radiation of the sun and contracts during the cooling of the night. The length differential between night and high sun at noon may be several inches, depending on the exposed length of the piping string. For example, for 150 F surface metal temperature, the growth could be 3 in. for 500 ft. As long as the final closure weld is not made, the piping freely grows, resisted only by the sliding friction at the bottom of the trench. But once the final closure weld is made, thermal thrust will occur and may cause damage either to a portion of the pipeline or to the rigid end connections. The ideal approach would be to test the line and bury it before the final closure weld is made.

In addition, on very large diameter lines, the differential between the top of the pipe exposed to direct sun radiation and the bottom hidden in the cooler earth may create some bowing of the line.

EARTHQUAKE

The damage caused directly by an earthquake may be only a small percentage of the total resulting destruction. The major loss will result from the fires started by the earthquake damage if the water services for fire fighting are badly disrupted. In the 1906 San Francisco earthquake, the broken water-service mains prevented any effective fire fighting and the major damage came from the unchecked fires that swept the city for days

The physical picture of the earthquake will vary with the intensity of the seismic disturbance, the distance from its epicenter, the soil characteristics, the contour of the area, and the type of readjustment taking place deep in the earth. The piping engineer will have to form some sort of a picture of the type of seismic action possible in the area under consideration, establish the seismic design conditions, and decide on the degree of seismic protection justified by the demands of reliability and economics for the pipeline and its branch components. The following is quoted from "Earthquake Damage and Earthquake Insurance" by John R Freeman, McGraw-Hill Book Company, 1932:

> The earthquake shock at the site of a structure is caused by the impact from the passage of an elastic wave which has originated deep in the earth from an earth-slip, probably along a "fault-line" or line of cleavage between vast fault-blocks, commonly many miles in depth, breadth and thickness, into which the outer formation of the earth to a depth of perhaps 40 miles, has been cracked by shrinkage stress and unknown causes during millions of years past.
>
> This elastic earthquake wave proceeds outward in all directions from its origin, and reaches the site of the structure under consideration, first as a longitudinal oscillation of alternate compression and expansion in the direction of transmission, followed quickly by a supplementary transverse oscillation at right angles to the direction of transmission.
>
> This wave system while passing through the earth is subject to reflection, or to partial absorption, at obstacles such as an abrupt change in geological formation, not far beneath the earth surface, and therefore may reach the building site in a much confused form.
>
> Although deep in the bed-rock these waves seldom exceed a small fraction of an inch in amplitude of motion, they may become magnified in mobile earth (such as alluvial deposits) near the earth surface, so that the violence of the wave motion near the earth's surface may vary largely at localities less than a mile apart. Moreover, the character of the wave motion may become changed from the harmonic form commonly assumed and into the form of oscillation an elastic body is ordinarily thrown by a sharp blow.

The earthquake acts as a giant vibrator upon the soil near the surface of the earth. Where the soil is hard cohesive, the surface movement will be less than where the soil is soft cohesive or cohesionless This vibratory action will compact and cause settlement of earth fills. Where there are water-saturated soils, the water forced by consolidation of the soil that had previously been retained in spaces between soil particles will provide temporary flotation of surface areas; and compression zones are created that act to heave some surface areas.

Separation of surface area may occur at sharp demarcation of soil characteristics, since movement of the adjacent soil areas could have been different. On hillside slopes, shear of the soil layer can occur, since the low elasticity of the soil cannot restore the soil to its former position during the back-and-forth surface oscillation.

Buildings will possess movements that may vary considerably from those of the soil areas surrounding them, and the fill areas next to the walls may settle under the vibratory action. Tanks with their water loads may shift, tend to rotate, and their shells ovalize in addition to the possible differential movement of the tank as a whole as compared with the surrounding soil.

Earthquake code provisions that speak of a percentage of gravity acceleration design requirement, for example, $0.2g$ for Zone 3 type of earthquake, to take care of seismic effects cannot realistically be applied to buried piping because the underground piping rides with the ground. Instead, provisions for seismic protection have to be considered on a differential movement basis in the following areas: at pipeline changes in direction, at pipeline crossings of sharp demarcations of soil characteristics, at pipeline entrances to fill around buildings and at building walls, at points of attachment to tanks or other aboveground equipment.

A sampling of some of the earthquake damage experienced by pipelines may be helpful:

Line pulled apart.
Line pulled apart, then telescoped.
Line crinkled and ruptured.
Line weld broken.
Breaks in cast-iron piping and fittings.

Adjoining areas of differing soils displaced and pulled apart piping mains.

Subsidence of filled ground due to earthquake caused breakage of water, sewer, and other buried lines.

Pipeline breakage at tanks has been noted where lower courses had failed and also in large-diameter tanks where flexibility of piping arrangement was not present. Failures were predominant in cast-iron fittings.

Passing through Fill and into Buildings. A sleeve should be used through the wall and should be of sufficiently larger diameter to provide the estimated seismic movement for the pipeline. In addition, consideration should be given to the use of a corrugated culvert (split type is available) around the pipe extending from the building wall to undisturbed earth. The wall end of the corrugated culvert should be supported independently of the pipe. The culvert will protect the pipe from the earthquake-compacting settlement, and the wall sleeve will allow horizontal movement in addition to vertical adjustment. However, if flexibility inside the building is limited, it may be necessary to consider horizontal flexibility outside the building so that the piping can move toward or away from the building. This may take the form of a right-angle leg for flexibility or movable joints.

Connection of Tanks. In tanks filled with water, a wave action is set up by the earthquake that may cause appreciable distortion in the tank. The shell configuration at the top becomes oval as the result of wave action, but the tank remains essentially circular at the bottom owing to restraint of the flooring and foundation friction. This difference in configuration causes the base of the shell to depress on the major axis, or direction of the wave, and lift on the minor axis. This action causes shell nozzle circular movement, including a lifting and lowering.

In addition, a differential may develop between the distorting of the tank shell and the movement of the adjacent earth Flexibility in the piping attached to the tank must be allowed in all directions to deal with these complex movements between the tank connections and the point at which the piping enters the earth. Cast-iron fittings and valves should not be used. Corrugated culvert around the piping, as it goes underground, might prove useful to obtain sufficient flexibility. Some form of flexible joint, such as a Barco-type joint, may aid in obtaining sufficient flexibility.

In crossing over from one type of soil to another where marked difference in seismic behavior is suspected, the main object is to obtain flexibility in all directions. An ample loop enclosed in an oversized corrugated culvert is one approach to obtaining flexibility.

Where there is a change in direction of the underground piping, the soil may move axially along one leg of the pipe and broadside against the other leg as movement differentials develop If sand is used around the pipe for at least 50 ft along both legs, the pipe has a better chance for adjustment The sand should be protected against dispersal and washout with some form of sheeting, such as polyvinyl chloride.

IRRIGATION PIPING

In semiarid regions where the natural rainfall is insufficient for growing crops to best advantage, irrigation frequently is resorted to if a supply of water is available from a river or lake. This can be accomplished by gravity flow from ditches or furrows or by pressure sprinkling from pipes.

Large amounts of lightweight steel pipe are used for irrigation purposes in the western United States and elsewhere. Since the advent of fusion welding, most of this pipe is fabricated from strip or sheet steel using straight- or spiral-welded seams. Where used aboveground, such pipe usually is joined with drive joints or with compression couplings of one sort or another such as Dresser or Victaulic or those having lever-clamping devices instead of bolts to facilitate breaking joints for moving the pipe about. Twenty-foot lengths are usual. Portable lightweight pipe for irrigation purposes frequently is galvanized as a protection against rusting.

Overhead Sprinkling Systems. Sprinkling, or overhead irrigation, may be carried on by the use of permanent systems with underground supply pipes and permanently located sprinklers or by the use of portable pipe. Overhead irrigation by sprinklers became practicable on a large and economical scale with the development of portable systems using pipe light enough to be moved around readily, yet strong enough to withstand rough field service and equipped with couplings that work easily and quickly without the use of tools or at most with just a wrench. Portable systems are said to have many advantages over permanently installed overhead systems. The initial investment is considerably less, there is nothing in the field to interfere with cultivating and planting, and the outfits can be moved quickly from one field to another as the need arises. The following recommendations for installing and operating aboveground portable sprinkling systems are taken from the "Handbook of Water Control":[60]

Experience in the field has shown that generally, with two men, a 1000-ft line can be moved to its next position parallel to the former position and 60 ft from it in from 30 to 35 min, ready for another application

Often in the field, special fittings such as ells, wyes, tees, etc , may be used to advantage to meet special operating conditions.

When one single line is used it is necessary to stop irrigating long enough to move the line to a new set-up. If an alternate line is provided, no time is lost in moving.

Revolving sprinklers give best results when operating at minimum pressures of from 20 to 30 psi, depending upon their size, the smaller sprinklers requiring less pressure than the large ones The diameter of coverage of sprinklers varies with the size of the sprinker and the operating pressure employed.

It has been found, in order to obtain and insure satisfactory coverage, that when small sprinklers discharging less than 7 gpm are used they should be spaced every 20 ft along the line. Sprinklers discharging 7 gpm or more give adequate coverage and distribution when spaced every 40 ft (at every second pipe joint) along the line.

Lines carrying small sprinklers operating under minimum pressures of 20 psi should be moved a maximum distance of 40 ft between set-ups.

Lines carrying sprinklers operating under a minimum pressure of 25 psi should not be moved more than 50 ft between set-ups

Lines carrying sprinklers operating under a minimum pressure of 30 psi should not be moved more than 60 ft between set-ups.

Manufacturers of sprinklers designed for agricultural service adjust their sprinklers to turn slowly—about 1 rpm. If they turn too fast, the effective diameter of the coverage circle is materially reduced.

The nozzle sizes of the sprinklers can be varied, depending on the amount of water and the pressure available The rate of application of water depends on the character of the soil. Some types of soil will take water faster than others, and on such soils water can be applied at a higher rate

Units of Water Measurement. In irrigation and hydroelectric work as well as in hydraulic mining, the units for *rate* of flow are the cubic foot per second (or second foot) for larger quantities and the miner's inch for smaller quantities. The cubic foot per minute also is used to some extent in hydraulic mining. The

miner's inch was developed in the early mining days and is still used extensively for other purposes as well. It is an awkward term, but its use often is necessary on account of custom and law. A miner's inch is the rate at which water discharges through 1 sq in of opening under a prescribed head (approximately 6 in.), and the number of miner's inches is equal to the area of the opening in square inches. The value of the miner's inch varies in different localities, ranging from $\frac{1}{50}$ to 1/38.4 cfs (see Table 9).

The units for *volume* of water delivered are the cubic foot, the acre-foot, and the acre-inch, which is one-twelfth the acre-foot. For large volumes the acre-foot is the unit recommended. This is the volume required to cover 1 acre to a depth of 1 ft, which equals 43,560 cu ft. One cubic foot per second flowing steadily for 24 hr approximately equals 2 acre-feet.

Table 9. Conversion of Units of Flow Used in Measuring Water

(From "Handbook of Water Control"[60])

Cubic feet per second	Gallons per minute	Million gallons per day	Miner's inches: Arizona, California, Montana, Nevada, Oregon	Miner's inches: Idaho, Kansas, Nebraska, New Mexico, North Dakota, South Dakota, Utah	Miner's inches: Colorado	Acre-inches per hour	Acre-feet per 24 hours
1	448.8	0.646	40	50	38.4	0.992	1.983
0.00223	1	0.001440	0.0891	0.1114	0.0856	0.0022	0.00442
1.547	694.4	1	61.89	77.36	59.44	1.535	3.07
0.025	11.25	0.0162	1	1.25	0.960	0.0248	0.0496
0.020	9.00	0.01296	0.80	1	0.768	0.0198	0.0397
0.026	11.69	0.0168	1.042	1.302	1	0.0258	0.0516
1.01	452.42	0.651	40.32	50.40	38.71	1	2.00
0.504	226.3	0.3258	20.17	25.21	19.36	0.5	1

The interrelation of the various units of measurement for rate of flow and volume of water delivered is shown in Table 9.

Measuring Devices. The miner's inch is measured through a *miner's inch box* which is a special form of free-flowing orifice consisting of an opening in a plank under a head of from 3 to 9 in The arrangement of the opening, which may be from 1 to 4 in high; the thickness of the plank, which may be from 1 to 3 in.; and the point from which the effective head is measured, as well as the head itself, are largely matters of local custom and of state law. Owing to the uncertainties of this method of measurement, the miner's inch usually is construed now as some fraction of 1 cfs as shown in Table 9.

A number of commercial *meters* are on the market which are designed for measuring irrigation water. Usually they regulate the flow of water and may or may not be arranged to register the total quantity passed The measuring element can be a weir, flume, orifice, venturi meter, or current meter used in connection with a registering device of some sort. Or the registering device may be omitted and the metering element used merely to limit the rate at which water can be taken A typical installation of a *metering orifice box* is shown in Fig. 13.

A special form of submerged orifice called a *meter gate* is used extensively for measuring water for irrigation purposes. A meter gate consists of a sluice gate to which are fitted two measuring wells in which the elevation of water on the upstream and downstream sides of the device can be read in inches [illegible] rule The

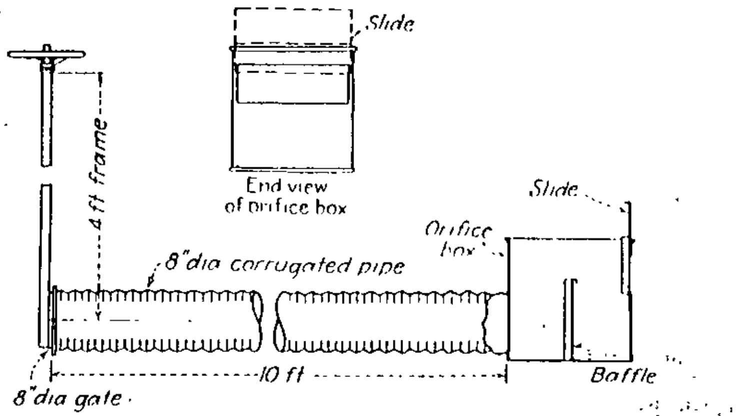


FIG. 13. Typical installation of a metering orifice box.

difference between the two readings gives the available head across the meter gate from which the rate of flow can be computed or taken from a table to correspond with the amount of opening. A typical meter gate installation is shown in Fig. 14.

When the installation of a meter gate is contemplated and the site selected, sufficient excavation is made to place the outlet pipe level with a clear opening inlet and outlet and set low enough to ensure the proper submergence of the outlet. Submergence should not be less than 6 in. under lowest conditions. If full submergence is not maintained, no readings can be taken in the downstream measuring well because of the surging of the water surface. Also, excavation should be made to give full contraction at the entrance if possible. The clearance between the inside of the gate and the side walls and bottom of the canal on the upstream side of the meter gate should be not less than 6 in.

After the gate has been installed the measuring wells are placed in position and connected by 3/4-in. pipes, one to the upstream side of the gate and the other to the main pipe 12 in. below the gate. A notch is made with a hack saw in the stem of the gate at a point flush with the top of the handwheel when the handwheel is without slack on the stem and the bottom of the gate slide is exactly level with the bottom of the inside of the gate seat. This is called the point of "zero gate opening." This means that the gate is not tightly closed. The stem will be the width of the gate seat higher than it is in the fully closed position. The amount of any gate opening is obtained by measuring the distance from the hack saw mark to the top of the handwheel.

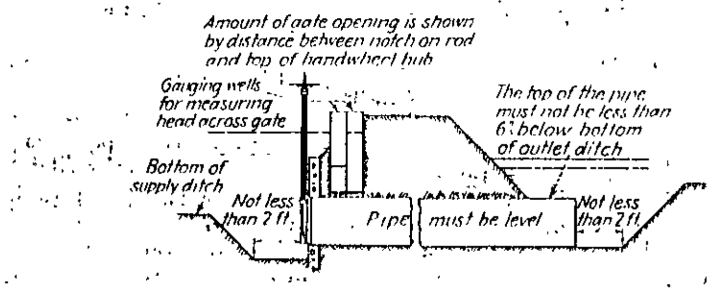


FIG. 14. Typical meter gate installation.

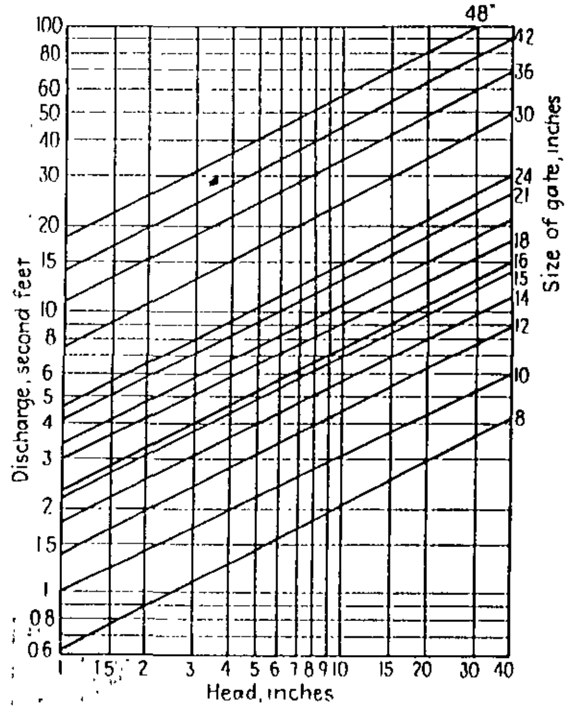


FIG. 15. Discharge capacities of fully opened meter gates.

When the turnout is in use, the water stands in one stilling well at the level of the water in the upstream canal and in the other well at the static level of the water in the turnout pipe at a point 12 in. downstream from the face of the gate seat. The difference in these two levels represents the static pressure on the gate opening and is denoted in these considerations as the "head on the gate."

Meter gates can be fastened to corrugated or smooth pipe of any length, as the only friction involved is in the first foot of pipe which is furnished as part of the meter gate. The discharge capacity at different heads of fully opened meter gates as manufactured by the California Corrugated Culvert Co. are given in Fig. 15. Charts are available in their "Handbook of Water Control" for reduced flows with lesser gate openings.

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