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# GENERAL FEATURES OF DESIGN (H 200)

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# H 200 GENERAL FEATURES OF DESIGN

## H 210 UNDERGROUND CONDUITS

Underground conduits include circular, box, casings, and tunnel structures (incl. arch shaped). The design loads for these structures are discussed in Section H 300, Design Loads and Distribution of Loads. Materials, design and construction specifications are covered in Section 207, 303, and 306, Standard Specifications and are supplemented here and in Section H 400.

## H 211 CIRCULAR CONDUITS

For structural design, circular conduits are separated into two major classes-rigid and flexible. Those considered as rigid conduits include, cast iron, nonreinforced concrete, reinforced concrete, and vitrified clay pipes. Flexible conduits include corrugated steel pipe, steel pipe, and plastic pipe. Pipe is usually installed in open trench but may be constructed by tunneling or jacking if necessary (subsections 212 and 213).

### H 211.1 TYPES AND USES

### H 211.11 DELETED

### H 211.12 CAST IRON PIPE (CIP) AND DUCTILE IRON PIPE

These pipes are primarily used where watertight joints are required and where the pipe must be capable of supporting relatively high internal pressures or external loads.
They are often used in pumping plants, force mains, and as a substitute for clay pipe sewers located close to underground structures or subjected to heavy earth or traffic loads.

Pit cast pipe and centrifugally cast pipe are the two major types of cast iron pipe. Pit cast pipe has lower strength: 11/31 iron strength; Modulus of Rupture = 31,000 psi (213.7 MPa); Tensile Strength = 11,000 psi (75.8 MPa). It has almost gone out of use in the western U.S. and is not readily available. Centrifugally cast pipe has higher strength (18/40 iron strength), is cast in metal or sand-lined molds, and is in general use. When greater strength, toughness, or ductility is needed, ductile iron pipe should be selected in lieu of cast iron pipe.

Iron pipe is specified by internal diameter, type, and pressure class or thickness class.

## H 211.13 NONREINFORCED CONCRETE PIPE (CP)

This type of conduit is used for small diameter perforated pipe subdrains and for cast-in-place nonreinforced concrete storm drain pipe. Precast pipe is available in diameters from 6-inch (152 mm) to 21-inch (533 mm) and is manufactured in two classes: standard strength and extra strength. Extra strength should be specified. Cast-in-place nonreinforced pipe is available in sizes from 24-inch (610 mm) to over 96-inch (2438mm) diameter.

Construction requirements are listed in Standard Specifications, Section 306-4.

## H 211.14 REINFORCED CONCRETE PIPE (RCP)

This type of pipe is used mainly for storm drains. Most sewers larger than 42 inches (1,067 mm) are also constructed of RCP with a plastic lining to protect it from corrosion by sewer gas. RCP with internal diameter if 108 inches (2,743 mm) or less is specified by internal diameter, D-Load, and case of bedding installation. For pipes larger than 108 inches (2,743 mm), structural details should be provided as described in Subsection H 211.2.

## H 211.15 VITRIFIED CLAY PIPE (VCP)

Due to its high resistance to corrosive atmospheres, sewers 42 inches (1,067 mm) or less in internal diameter are usually constructed of extra strength VCP. VCP is specified by internal diameter. For open-trench construction, the type of bedding and concrete blanketing, when required, should be indicated on the plans.

## H 211.16 CORRUGATED STEEL PIPE (CSP)

Materials and fabrication of CSP are covered in Subsection 207-11 of the Standard Specifications. Pipes are specified by minimum internal diameter, type, and gage (metal thickness). This pipe is generally available with standard corrugations or with helical corrugations.

To improve flow characteristics and prevent excessive abrasion, the invert is usually paved with bituminous material. Both external and internal bituminous coating may be necessary for corrosion protection, depending on soil conditions (consult with Geology and Soils Engineering Section).

CSP is used for storm drains, as an alternative to RCP, where it is necessary to lay pipe on a steep grade. It is also used as a temporary bypass during construction of a permanent storm drain or sewer facility.

## H 211.17 STEEL PIPE

Steel pipe is generally not used for sewer or storm drain gravity lines because it is susceptible to corrosion. It is used, however, for pressure piping systems in pumping plants and sewage treatment plants. It is also used as jacked casing for installing other pipes.

## H 211.18 PLASTIC PIPE

In the United States, plastic pipe has been growing in use since the 1970's. The market continues to grow as new compounds are formulated, manufacturing is improved and applications are found in larger diameter pipe. Plastic pipe offers several advantages such as inherent corrosion resistance and ease of construction due to its lighter weight.

Plastics are composed of thermoplastic and thermosetting resins such as acrylonitrile butadiene styrene (ABS), fiber-reinforced (CCFRPM or FRP), polyethylene (PE), poly vinyl chloride (PVC) or saturated-fibers such as cured-in-place pipe (CIPP). FRP, PE and PVC formulations are generally identified by "cell classification" per ASTM D 3262, D 3350 and D 1784, respectively. The cell classification is a series of numbers and letters that correspond to the ranges of properties in a plastic compound. This range does, however, allow different compounds to have the identical cell classification. Plastic pipe shall be specified on the project drawings by manufacturer, pipe strength, bedding and diameter (see Special Order 001-0296). The pipe strength is expressed as the stiffness factor [lb-in$^2$ per lineal inch or MPa-m$^2$ per lineal m], the product of the initial flexural modulus and pipe wall cross section moment of inertia, or other equivalent term. Compliance is determined by a comparison of the tensile strength, elongation, flexural modulus, specific gravity, impact strength and apparent cell classification between the supplied material and material used for the qualification tests. The list of currently approved plastic sewer products with their compounds and resins is provided in the latest edition of “Brown Book” (City of Los Angeles Department of Public Works Additions and Amendments to the Standard Specifications for Public Works Construction).

Plastic pipe is designed to function either as a no-load-bearing (non-structural) liner or as a load carrying (structural) pipe. For either purpose, the Bureau has established a minimum design life of 50 years before first maintenance. All plastic pipes are designed with the same flexible conduit theory. Plastic pipe with properly designed bedding is approved by the Bureau as an alternate construction material for rigid conduits.

Liners are used to stabilize pipe corrosion, remedy exfiltration, remedy moderate amounts of groundwater infiltration, lower hydraulic friction coefficients or restore scoured areas. All rehabilitation liners 18" (450 mm) or greater should be reviewed by the Structural Engineering Division to verify the host pipe condition and ovality.

Many plastic pipes can be used for direct burial, directional drilling and microtunneled installations or for sliplined structurally deficient host pipes. If unusual circumstances are present, site specific calculations are necessary. All pipes 18" (450 mm) or greater and load carrying pipes of any size must be designed by the Structural Engineering Division (SED). SED will prepare site specific calculations, make bedding selection, and generate a list of all pipe products suitable for the project with the necessary pipe information as noted above.

This Standard is for City projects only. Other agencies (i.e., CalTrans) will have varying design requirements.

## H 211.2 DESIGN OF REINFORCED CONCRETE PIPE (RCP)

The design of RCP is limited only by the capacity of the testing facility, i.e., the maximum size on which the D-Load bearing strength test can be performed. Usually, pipe sizes to 108 inches (2,743 mm) can be so tested. For these sizes, the designer determines the required D-Load as discussed under D-Load Design, Subsection H 211.21.

For pipe sizes greater than 108 inches (2,743 mm), the required pipe should be designed in accordance with Subsection H 211.23. The designed pipe should be detailed on the plans similar to Figure H 211.23, with both alternates No. 1 and No. 2, being shown.

Except for RCP in railroad R/W, the recommended minimum earth cover is one foot (0.31 m). When a cover less than one foot (0.31 m) is necessary, a bedding installation providing a load factor of 3.2 is recommended for all pipe sizes (see Standard Plan S-251). The recommended minimum cover for RCP in railroad R/W is discussed in AREA, Chapter 1, Part 5. Concrete encasement requirements for pipe sizes 36 inches (914 mm) and smaller under shallow cover are shown in Figure G 613, Storm Drain Manual.

### H 211.21 RCP D-LOAD DESIGN, ID EQUAL TO OR LESS THAN 108 INCHES (2,743 MM)

A chart showing the required pipe D-Load and bedding based on trench condition is shown in the Storm Drain Design Manual, Figure G 613. Earth cover over the pipe is limited to 40 feet (12.192 m) or less. Unit weight of soil is 100 pcf, and the live loading is AASHTO HS20-44 (Highway Loading). When these parameters do not apply, the required D-Load strength of RCP can be determined by the following formula:

$$D - Load = \frac{W \times S.F}{Di \times L.F.}$$

Where

W = Total vertical load = We + Wl

We = Earth Load, plf (N/m)

Wl = Live Load, plf (N/m)

S.F. = Safety Factor:
For storm drains, use 1.25
For sewers, use 1.5

Di = Internal diameter, feet (m)

L.F. = Load factor which depends upon bedding installation

Earth loads on underground conduits are discussed in Subsection H 371. For earth covers 8 feet (2.438 m) or less, pipe should be designed for positive projection condition, since formulas for trench condition give excessive, erroneous results for shallow pipe.

The distribution of live load is explained in Subsection H 374. Neglect the effects of truck live load for depths of earth cover greater than 8 feet (2.438 m). The horizontal force due to truck live load is neglected for D-load design.

The effects of internal water pressure are discussed in Subsection H 211.23. In a majority of designs, the increase in allowable stress will more than offset this loading. Surcharge loads, such as buildings, should be considered in the design.

Generally, the **load factor** to be used for trench, negative projection, or positive projection conditions depend on the bedding and are as defined in Standard Plan S-251. However, for positive projection, an increased load factor may be used if there is little or no possibility of removing the lateral support along the pipe by a future excavation.

Spangler's formula (see Reference 1) for computing this increased load factor uses the assumption that active soil pressure against the sides of a pipe placed in a positive projection condition is a significant factor in increasing its supporting strength. The formula is:

$$Load\ Factor = \frac{1.431}{N - XQ}$$

Where

N = A parameter which is a function of the distribution of vertical load to the subgrade. Use.840 for earth bedding and .505 where a concrete cradle is used.

X = A parameter which is a function of the area of vertical projection of pipe on which the active lateral pressure of the fill material acts.

| **Projection Ratio (p)** | **Value of X** |
|---|---|
| 0 | 0 |
| .3 | .217 |
| .5 | .423 |
| .7 | .594 |
| .9 | .655 |
| 1.0 | .638 |

Q = **(MK/C)(H/B)+M/2**

Where

Q = Ratio of total lateral pressure to total vertical load.

K = Rankine's lateral pressure ratio = 0.33.

M = Fractional part of B over which lateral pressure acts; can be assumed to be the same as p, projection ratio. P = H/B

C = Load coefficient for positive projection condition. (See Subsection 371.31)

H = Height of fill above top of conduit, feet (m).

B = Outside width of the conduit, feet (m).

The minimum pipe strengths specified should be 800D, except in the following instances:

a. 1,350-D for streets designated as State highways.

b. 800-D for railroad R/W as specified in AREA, Chapter 8, Part 10. Yerify this with the affected railroad company, as their minimum requirements may differ.

The maximum recommended D-Load values are shown in Figure H 211.21. When the calculated D-Load exceeds these limits, consider bedding installations with higher load factors because considerable care in manufacturing and special design are necessary to obtain higher D-Loads.

The calculated D-Load should be rounded off as follows:

| | |
|---|---|
| 36-inch (914 mm) diameter and less | To next highest 250 of calculated value. |
| 39- to 60-inch (991 to 1,524 mm) diameter | To next highest 100 of calculated value. |
| 63- to 108-inch (1,600- to 2,743 mm) diameter | To next highest 50 of calculated value. |

### 211.22 RCP JACKED, PLACED IN CASING OR TUNNEL

Required D-Load design of RCP jacked or placed in jacked casing or in tunnel should be as follows:

a. **Jacked RCP** - Design the pipe as described above, except for the earth load which is calculated per Subsection H 372.2. A load factor of 2.7 is recommended.

   The economy of jacking is discussed in Subsection H 212. Requirements for jacking RCP, are specified in Section 306-2.1 of the Standard Specifications.

   For requirements of shoring jacking pits, refer to Subsection H 212.5, Box Conduits to be Jacked.

b. **RCP in Casing or Tunnel** - Design the pipe as described above except that a load factor of 4.5 is recommended.

### H 211.23 RCP, ID GREATER THAN 108 INCHES (2,743 MM)

Two design methods are currently used; the "Elastic Analysis Method" and the "Elastic Center Method."

In the **Elastic Analysis Method coefficients** are used to calculate the critical moments and thrust forces. The coefficients were published in the **Engineering News-Record**, Volume 87,1921, page 768. These values and applicable loading conditions are shown in Appendix No. A-3.

For the **Elastic Center Method**, see "Analysis of Arches, Rigid Frames, and Sewer Sections," Portland Cement Association Publication No. ST53. This method is used in the computer program, "Reinforced Concrete Arch" (refer to Section H 190).

The loadings used are the same as for RCP D-Load design, and should also include the effect of internal water pressure. Generally the pipe is designed for the conduit flowing just full in combination with other loading conditions using the standard design stresses. When the hydraulic gradient exceeds 5 feet (1.52 m) above the top of pipe, especially in cases of shallow earth cover, design analysis should include the following case:

> Pressure due to hydraulic head from the soffit of conduit to hydraulic gradient, the internal water pressure assuming the conduit flowing just full, dead weight of the structure, and the vertical and horizontal earth loadings. For this loading condition, the allowable stresses may be increased by one-third.

Soil reaction to vertical loads, is assumed to be uniformly distributed under a portion of the bottom of pipe depending on the pipe bedding. Allowable stresses are recommended as follows:

| | | | | | |
|---|---|---|---|---|---|
| f 'c | = | 4,500 psi (31.03 MPa) | fc | = | 1,800 psi (12.41 MPa) |
| n | = | 8 | fs | = | 24,000 psi (165.48 MPa) |

For other concrete design criteria, see Section H 410.

The recommended minimum pipe wall thicknesses and steel areas are listed below. When a greater wall thickness is required for extreme loads, the pipe manufacturer should be contacted for recommendations. The minimum steel area specified is for a circular cage (Cage D, Figure H 211.23 ) and provides for pipe handling and other loads during construction.

| ID [inch] | ID [mm] | Minimum Thickness [inch] | Minimum Thickness [mm] | Minimum Steel Area [inch$^2$] | Minimum Steel Area [mm$^2$] |
|---|---|---|---|---|---|
| 114 | 2,896 | 10.5 | 267 | 0.25 | 161 |
| 120 | 3,048 | 11.0 | 279 | 0.27 | 174 |
| 126 | 3,200 | 11.0 | 279 | 0.30 | 197 |
| 132 | 3,353 | 11.5 | 929 | 0.32 | 206 |
| 138 | 3,505 | 11.5 | 929 | 0.35 | 226 |
| 144 | 3,658 | 12.0 | 305 | 0.38 | 245 |

## H 211.3 DESIGN OF CAST IRON PIPE (CIP) AND DUCTILE IRON PIPE

Refer to Section 207-9, Standard Specifications and to the following specifications of the American Water Works Association for detailed design requirements:

a. Thickness Design of Cast Iron Pipe - ANSI-AWWA C101

b. Thickness Design of Ductile Iron Pipe - ANSI-AWWA C105

It is suggested that earth loads be calculated as discussed on Subsection 371.

## H 211.4 DESIGN OF FLEXIBLE PIPES

The design of flexible pipes made of corrugated metal, thin-walled steel, or plastic is based on horizontal deflection under load. The pipe itself has relatively little inherent strength, and a large part of its capability to support vertical loads is derived from passive pressures induced as the sides move outward against the soil.

For plastic pipe design, see Subsection H 211.41. For materials which do not creep due to constant loading (such as corrugated metal or thin-walled steel), the allowable long-term deflection recommended for design is 5 percent of the nominal pipe diameter (2-1/2 percent if the pipe is to be lined and coated).

The deflection calculation is based on Spangler's deflection formula for flexible conduit (see reference 2) and is as follows:

$$d = \frac{D.K.W.(FS).R}{E.I + 0.061E'.R}$$

Where:

d = Deflection of the pipe, inches (long term).

D = Deflection lag factor, dimensionless. recommended value is 1.5

K = Bedding constant, dependent upon bedding angle, dimensionless. recommended value is 0.1

W = $W_e + W_l$ = Total vertical load per unit length of pipe, lbs./lin. inch.

$W_e$ = Earth load (weight of prism of soil above pipe is recommended). See optional method for computing earth load below.

$W_l$ = Live load, for earth covers 8 feet or less, Subsection H 374

R = Third power of r.

r = Mean radius of pipe, inches.

E = Modulus of elasticity of pipe material, psi.

FS = Factor of safety. Recommended value is 1.5.

I = Moment of inertia per unit of cross section of pipe wall, inches/inch.

or E.I = Stiffness factor for pipe, inch$^2$ lb/inch. = 0.149 R (PS), where PS is pipe stiffness = load per inch of pipe per inch deflection (published or test value similar to three-edge bearing), psi

E' = Modulus of soil reaction, psi; 100 for Case I, 700 for Case II (using Type "A" material) and 1,000 for Case II (using Type "B" material) bedding installation per Standard Plan S-251.

The designer should note that the first term of the denominator (E.I) represents the inherent strength of the pipe which should not be less than 15 percent of the term (0.061E'.R) for direct burial installations.

For small pipes, the earth load is the weight of the prism of earth above the pipe (outside diameter x depth x weight of earth). For larger pipe, (24-inch or greater diameter) the designer should consider the earth load based on Marston's Theory for flexible conduit as follows:

$$W_e = \frac{w \cdot C_d \cdot B_c \cdot B_d}{12}$$

Where:

w = Unit weight of earth (110 pcf minimum)

$C_d$ = $[1 - e^{(-2Ku.H/B_d)}] / [2K_u]$

$B_c$ = Pipe outer diameter, inches.

$B_d$ = Trench width = $B_c$ + 20 inches.

$K_u$ = Soil friction coefficient (recommended value is 0.15).

H = Depth of earth fill over pipe, feet.

The coefficient E' represents the supporting strength of the surrounding soil which has considerable influence on the deflection of flexible pipes. For this reason, the loss of support due to future excavations adjacent to a proposed flexible conduit should be considered. Where adequate clearance to future excavations cannot be provided, a rigid conduit should be specified.

Figure G 613 A of the Storm Drain Design manual shows the required strength gage requirement for CSP.

## H 211.41 DESIGN OF PLASTIC PIPE

The design of flexible pipe (plastic) is heavily dependent on soil-pipe interaction. Pipe bedding and other support conditions are critical to the ability of a plastic pipe system to withstand loads.

Plastic pipe is subject to relaxation from creep and plastic flow. For that reason, the short-term (initial) engineering values are often between two and six times larger than the long-term (50-year, steady state design) values. The factor used to reduce the short-term values is unique for each specific compound. For CIPP liners the long-term values are estimated at 40% of the short-term engineering values. If supported by laboratory testing, higher retention values may be used up to 50% maximum retention of the initial design values. The preferred data is 10,000 hours duration flexural creep tests such as AASHTO Section 18 cantilever beam tests, ASTM D 2990 (ASTM D 790 tests modified for a constant load), or ASTM D 2412 tests (modified for a constant load). For most homogeneous plastics, substitute data may be derived from 10,000 hours duration tensile creep tests such as ASTM D 2990 (ASTM D 638), ASTM D 2837, or ASTM D 2992. Compressive creep data are generally not useful.

Lower grade compounds often result in a zero long-term value, suggesting that the plastic is not suitable for load carrying use. Plastic pipes with unproven long-term strength have questionable load carrying capacity and are particularly susceptible to excessive pipe deflection or loss of integrity at joints and gaskets. Plastics differ from other types of flexible conduits such as corrugated metal pipe or thin-walled solid metal pipe that do not exhibit significant creep phenomena.

For direct burial pipe, the recommended initial pipe stiffness is 46 psi (200 psi for ABS truss wall pipe). This is intended to provide reasonable assurance that the pipe bedding can be installed without distorting the flexible conduit. Other installation methods, such as microtunneling may require a greater pipe stiffness. Internal trussing or bracing of a flexible pipe to facilitate installation introduces unanticipated concentrated loads and is not permitted. The suggested minimum bedding installation per Standard S-251 is as follows:

| Type | Cover | Bedding |
|---|---|---|
| Solid Wall PVC, ABS | Less than 4' (1.22 m) | 5** |
| or HDPE | 4' to 18' (1.22 m to 5.49 m) | 2 |
| | 18' to 30' (5.49 m to 9.14 m) | 4 |
| Truss Wall ABS or | Less than 4' (1.22 m) | 5** |

| | | |
|---|---|---|
| CCFRPM | 4' to 10' (1.22 m to 3.05 m) | 2* |
| | 10' to 22' (3.05 m to 6.71 m) | 2 |
| | 22' to 30' (6.71 m to 9.14 m) | 4 |

* May be Type “A” material.

** The use of Case 5 Bedding installation (concrete encasement) for pipes with earth covers less than 4 feet (1.22 m) is to avoid pipe failure due to low resistance of the plastic pipe system to fatigue stresses from repetitive traffic loads. Case 5 bedding also protects pipes within the 4 feet (1.22 m) utility zone from errant construction and disruption of the pipe bedding envelope.

Plastic pipes are designed by using a series of equations to calculate the minimum required stiffness factor. The stiffness factor that governs the design is the largest resulting from the deflection, minimum strength (for direct burial and microtunneling installations), buckling, handling buckling and thrust equations. Most of the equations listed in the next sections yield long-term values. These values must then be converted to short-term values using the ratio of the individual product’s short-to-long term engineering values. The final short-term values should then be compared to the required pipe stiffness and any product specific minimum standard dimension ratio (SDR), when applicable, to determine the governing value. Except when the "handling buckling" criterion controls, the designer should consider increasing the required wall thickness to account for scouring over the design life. Scouring is a concern in conduits with either a high gravel load or high flow velocity. A method of measuring abrasion resistance is modified ASTM C 501.

It has been suggested by some researchers that the calculated stiffness factor may be reduced by using an “enhancement factor”, “bedding factor” or other factor. The basis of those recommendations are laboratory research that used small pipe diameters in circular pipe conditions unattainable in the field. The effect of out-of-roundness, pipe corrosion, offset joints, debris and other expected irregularities significantly affects the pipe behavior and "enhancement". Due to a lack of substantiation, non-uniform application by the industry and arbitrary selection of the factors, those factors are not recognized at this time.

H 211.41.1 DESIGN LOAD

The design load shall include soil, internal, groundwater and traffic loads. Load that is present for more than one month is a dead load. Traffic load is instantaneous and considered a live load. Loads of intermediate duration require judgment by the designer or an additional geotechnical investigation. An example of intermediate duration is the added weight due to precipitation absorbed by an overbearing soil which does not freely drain.

**Soil**: Except for microtunneled, jacked, or pipes installed in jacked casings, the prism load shall be used whose applied pressure is (Depth of Cover) x (Soil Density). The soil density is assumed to be 120 pcf (1920 kg/$m^3$) and may be modified by a soil investigation, but shall remain a minimum of 110 pcf (1760 kg/$m^3$). For rehabilitation installations of plastic pipes with deep soil covers, a project specific soils report may allow use of effective soils height. For earth loading on jacked pipe refer to section H 372.2. For micro-tunneled pipe and pipe installed in jacked casing, effective soils height shall be used as provided by soil investigation.

**Internal**: Negative (vacuum) or positive (under head) pressure that may occur near siphons or pumping plants.

**Groundwater**: The minimum design height shall be the larger of, 1) the reported static (present for more than one month) groundwater height above the plastic pipe, 2) the host pipe inside diameter, or, 3) the plastic pipe outside diameter. The designer should also consider the potential for higher groundwater fluctuations during wet years and the effect of pipe buoyancy in saturated soils. For load carrying pipes, where no groundwater is present, designers can consider these values solely as design minimums which may be omitted when the sum of other applicable loads is greater. Also, consideration of all three conditions listed above may be limited to the Buckling and Thrust equations of Sections H211.41.4, H211.41.6. For other calculations (deflection, strength) use only conditions 2 & 3 (the greater of 1 diameter of the host pipe ID or the plastic pipe OD).

**Traffic**: Uniform loads are available from Bureau of Engineering Manual Part H. At 8 feet (2.5 m) of soil cover, vehicular live loads are assumed to have dissipated for small circular conduits. For plastic pipes installed under railways, the railroad dead and live loads shall be calculated based on the latest edition ACPA (American Concrete Pipe Association) Design Data 3.

The sum of the design loads is W, representing a line load applied along the centerline of the conduit:

W [pounds per lineal inch or MPa-mm] =
(Vertical pressure at the outside face of the plastic pipe crown) x (Host pipe outside diameter)
For direct burial pipe, W [pounds per lineal inch or MPa-mm] =
(Vertical pressure at full cover depth) x (Pipe outside diameter)

H 211.41.2 DEFLECTION

Strict deflection limits are necessary so that, 1) equipment used for maintenance operations such as cleaning or clearing blockages will not abrade the plastic pipe, 2) street maintenance is avoided for surface settlements due to soil reconsolidating with the deflecting pipe, and 3) the use of “circular” pipe equations is validated. Maintenance concerns govern at smaller diameters while street maintenance concerns govern with larger diameters. The integrity of the plastic pipe itself is generally not in question since nearly all plastic pipe can sustain much larger deflections without adverse effect.

The table below establishes the allowable deflection for use with the equations herein.

| **TABLE 211-41.3(A) ALLOWABLE CALCULATED DEFLECTION** (also see Standard Specifications Section 306-1.2.12) | |
|---|---|
| **PLASTIC PIPE OUTSIDE DIAMETER** | **ALLOWABLE DEFLECTION** (Percent of mean plastic pipe inside diameter) |
| < 12 inches (300 mm) | 3.75% |
| < 30 inches (750 mm) | 3.00% |
| < 60 inches (1500 mm) | 2.25% |
| < 90 inches (2250 mm) | 1.88% |
| < 120 inches (3050 mm) | 1.50% |
| Over 120 inches (3050 mm) | 1.13% |
| "Tight-fit" liners | |
| A) With ring [hoop] strength present in the host pipe & no longitudinal cracks | 5.00% |
| B) Otherwise | 3.75% |
| Annular-grouted sliplined pipes (With host pipe in good condition) | 3.75% |
| Non-traffic areas (Surface settlement is tolerable) | 3.75% |

"Tight-fit" requires the intentional removal of organic debris, sediment or other material tending to separate the liner from host pipe. Rehabilitation liners installed "tight-fit" only deflect due to compressive ring shortening because geometric restraint is provided by the host pipe after progressive structural deflections of 1.5 to 2%. These liners are credited with higher allowables when host pipes offers hoop resistance (RCP with reinforcing intact or VCP without longitudinal cracks). The allowable deflection is also liberalized for the redundant pipe system in annular grouted sliplined pipes and for non-traffic installations to account for the minimal impact with surface settlements.

Deflection is calculated using Spangler's Iowa Formula:

$$\text{Deflection} = \frac{K.W.r^3}{(E.I) + (0.061E'.r^3)} \quad \text{[in or mm]}$$

or solving for the stiffness factor, the minimum long-term stiffness factor is

$$\text{Minimum } (E.I)_1 = r^3\left(\frac{K.W}{d} - 0.061E'\right) \quad \text{[lb-in}^2\text{/lin. in. or MPa-m}^2\text{/lineal m]} \qquad \text{(Eq. 1)}$$

E = Long-term flexural modulus of elasticity [psi or MPa]. For homogeneous plastics, the long-term tensile modulus [psi or MPa] may be substituted.

E' = Soil modulus [psi or MPa]

I = Effective transverse pipe-wall moment of inertia after an allowance for scouring [$in^4$/in or $mm^4$/mm]

K = Dimensionless bedding constant

d Allowable calculated deflection [in or mm]

r = Mean pipe radius [in or mm]

Notes:

1. The long-term modulus of elasticity, E, is derived from extended laboratory tests extrapolated to 50 years. It is often expressed as a fraction of the initial (short-term) modulus of elasticity. For CIPP liners, E shall be taken as 40% of the design initial modulus of elasticity of 250,000 psi, which is equal to 100,000 psi. See H 211.41 for exceptions.
2. The soil modulus, E', varies by degree of compaction and soil type. Values are generally determined from a soil investigation with a functional range from zero to 3,000 psi (20.7 MPa). The soil modulus may be assumed to be a maximum of 1,250 psi (8.6 MPa) for liner installation projects where the host pipe ring strength is present and uniform bearing between the plastic pipe and host pipe can be achieved. The soil modulus is recommended to be 100 psi (0.7 MPa) for Case I; 700 for Case II (using Type "A" material) and 1,000 for Case II (using Type "B" material) bedding installation per Standard Plan S-251. The "Minimum Strength" requirement limits the effective soil modulus to the stiffness factor ÷ 0.15 for direct burial and microtunneling application.
3. The factor of safety is omitted. The uncertainty that exists with assumptions regarding the progression of structural deflection and the long-term stability of bedding is considered mitigated by using the allowable deflection limits, long-term plastic values and prism loading.
4. The moment of inertia for ring equations is derived from a unit length, transverse section of pipe wall.
5. The bedding constant, K, is tentatively assumed as 0.10 to represent average bedding conditions for sliplined and direct burial installations. Based on two destructive tests conducted by the Bureau in 1994, this factor is under study as unconservatively low for annular grouted, sliplined projects.
6. The mean radius is (OD+ID)÷4.
7. Since prism loads are used, the deflection lag factor was taken as unity and omitted from the equation.
8. Deflection can be expressed as a sum of the dead load and live load deflections. Dead load deflection is calculated using the long-term modulus of elasticity and live load deflection may be calculated using the short-term modulus of elasticity.

## H 211.41.3 MINIMUM STRENGTH

For direct burial and microtunneling installations, the inherent strength of the pipe stiffness factor, $E.I$ , shall be 15% of the soil strength. This requirement is consistent with the development of the Iowa Formula, fits slight bedding variations and provides reasonable constructability. With terms from the denominator of the Iowa Formula:

$$\frac{E.I}{15\%} = 6.67\left(E.I\right) \geq 0.061E'\,r^3 \quad \text{(For direct burial)}$$

In weak bedding, this criterion is often immaterial. However, with high soil modulus and a pipe with a relatively weak stiffness factor, the minimum pipe strength will govern in the Iowa Formula. Substituting the minimum strength in terms of soil strength and K = 0.1 into the Iowa Formula yields:

For direct burial and microtunneling, the minimum long-term stiffness factor is as follows:

Minimum $(E.I)_2 = 0.0130\left(W\,\frac{r^3}{d}\right)$ [lb-in$^2$/lineal in. or MPa-m$^2$/lineal m] (Eq. 2a)

with a corresponding effective soil modulus for subsequent calculations,

E'(effective) $= 1.426\,\frac{W}{d}$ [psi or MPa] (Eq. 2b)

H 211.41.4 BUCKLING

The minimum long-term stiffness factor for this condition is as follows:

Minimum $(E.I)_3 = 0.250\left(\frac{W}{C}\right)^2\left(\frac{r}{B'E'}\right)$ [lb-in$^2$/lin. in. or MPa-m$^2$/lineal m] (Eq. 3)

B' = Empirical elastic support coefficient, $\frac{1}{1+4\,e^{-0.065H}}$ OR $\frac{1}{1+4\,e^{-0.213H}}$ [SI]

C = Correction factor for ovality, see the table below.

FS = The factor of safety is established as 2.0 and incorporated into the equations

H = Depth of fill above pipe [ft. or m]

q = (Maximum ID - Average ID) ÷ Average ID, ovality.

Table 211.41.4(A) - **Correction Factors for Ovality**, C = $[(1 - q) \div (1 + q)^2]^3$

| | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|
| | | VCP HOSTS | NOT POSSIBLE | GOOD CONDITION | LONGITUD. CRACKING | VIS. PERCPT FLATTENING | | | CONSIDER PIPE REPLACEMENT |
| | | RCP HOSTS | GOOD CONDITION | UNIF. CONC CORROSION | VAR. CONC CORROSION | VAR. CONC CORROSION | REBAR CORROSION | | |
| **q** | 0 | 0.01 | 0.02 | 0.03 | 0.04 | 0.05 | 0.06 | 0.07 | ≧0.10 |
| **C** | 1.00 | 0.91 | 0.84 | 0.76 | 0.70 | 0.64 | 0.59 | 0.54 | Equations invalid, use finite elem. analysis. |

Notes:

1. For liner installation projects where the host pipe ring strength is present and uniform bearing between the plastic pipe and host pipe can be achieved, use B' = 0.50.
2. The condition of the host pipe and its effect on the installed plastic pipe shape must be considered. Ovality is the sum of the installed out-of-roundness and the allowable deflection due to loading after installation. For liners, the tendency of the liner to reflect the eroded host pipe shape should also be considered.

This correction factor is used to fit elliptical shapes (with its added stress points) with the circular pipe equations above. For q greater than 0.05, a gross design problem exists which require detailed further investigation. When q approaches 0.10, the equations are no longer correctable and finite element analysis is recommended.

3. The above equation is based on an AWWA C-950 equation developed for FRP pipe.

## H 211.41.5 HANDLING AND INSTALLATION:

The minimum long-term stiffness value for this condition is

Minimum $(E.I)_4 = 0.75\, r^3 \left( \dfrac{E}{E_S} \right)$ [lb-in²/lineal in. or MPa-m²/lineal m] (Eq. 4)

$E_S$ = Short term modulus of elasticity of pipe [psi or MPa]

Note: When the pipe is exposed to temperatures that significantly differ from 77 degrees Fahrenheit (an ambient sewer), consult with the pipe manufacturer to modify the modulus of elasticity for temperature effects.

## H 211.41.6 THRUST

The minimum long-term stiffness value for this condition is

Minimum $(E.I)_5 = 0.083E \left( \dfrac{W}{F_c} \right)^3$ [lb-in²/lineal in. or MPa-m²/lineal m] (Eq. 5)

FS = The factor of safety is established as 2.0 and incorporated into the equations.
$F_c$ = Allowable long-term compressive strength [psi or MPa]
$= c_T \,.\, F_t$
$F_t$ = Allowable long-term tensile strength [psi or MPa]
$C_T$ = 0.9 for homogeneous plastics, and 0.8 for composite plastics such as CIPP and CCFRPM

## H 211.41.7 DESIGN LIMITATIONS

These equations are derived from gravity flow, thin-walled, circular pipe research. When conduits are severely corroded, oval or other non-circular shape, finite element analysis is recommended unless the plastic pipe can be kept circular. These procedures also assume that the pipe is prismatic and that the failure is along the radial direction. For non-prismatic pipe, i.e., ribbed sections, the general equations apply but local deflection and local buckling require investigation. The designer shall also consider whether a longitudinal analysis is warranted. In such case, each plastic pipe length is analyzed as a beam on an elastic foundation. The pipe is then checked for long-term longitudinal flexural and buckling stresses.

## H 211.41.8 SPECIAL DESIGN CONSIDERATIONS

### A. Optimal Design

For cases where buckling (Equation 3) controls, the design should be optimized such that the deflection and buckling equations will yield the same required wall thickness. This is accomplished by modifying the effective soil modulus and balancing the two equations. The resultant is a pipe that theoretically has a simultaneous failure in deflection and buckling.

### B. Shallow Cover

When plastic pipe is installed in streets with less than 4 feet (1.2 m) of cover, provide Standard Plan S-251 Case V bedding (concrete encasement) to protect the pipe from localized, traffic impact loads. This avoids pipe failure due to low resistance of plastic pipes to fatigue stresses from repetitive traffic loads and careless excavation operations.

### C. Installation in Utility Zones or Near Adjacent Structures

Where the bedding is likely to be disturbed, the plastic pipe should either be designed with reduced soil modulus or upgraded to Standard Plan S-251 Case V bedding. This generally occurs near conduits and other buried structures that require future excavation (for maintenance or replacement) and high utility zones at shallow covers. Rigid pipe alternatives should be considered.

### D. High Temperature

Elevated temperatures can drastically affect the design properties of thermoplastic pipe. If effluent is tributary to powerplants, laundromats, or other “hot” sources, use a non-plastic pipe alternate or determine and consider the effect of temperatures.

### E. Detailed Study

For larger projects, the magnitude warrants more detailed, project-specific study. The designer should consider the following:

1. A geotechnical investigation to detect the frequency and amount of groundwater surcharge over the design life of the pipe.
2. A geotechnical investigation to determine the modulus of soil reaction, soil density and whether a Marston load with deflection lag factor is appropriate.
3. A hydraulic study to determine whether scouring is occurring. For lines carrying high velocities, sacrificial wall thickness should be added.
4. A series of chemical tests to determine the design strength of the anticipated sanitary sewage. This information must be tempered with predictions of future use of the line, zoning changes, etcetera. When extrapolated over the design life of the pipe, less expensive plastic resins and compounds may be acceptable. At extremes, sewers may range from pH=1 to pH=12 or more.
5. A schedule of required pipe wall thicknesses that vary by reach.

6. Segregate "structural rehabilitation required" areas from "liner required" areas on rehabilitation projects. Data from coring, person-entry investigations, closed-circuit televising and/or other investigation would be required.

F. Storm Drains

The use of plastic pipe for storm drains present special considerations for the designer.

1. Plastic pipe is combustible and should be protected where fire is likely. Access to the pipe should be restricted with grates or bars to prevent vandalism and arson. Other devices include inlet/outlet structures of reinforced concrete which also extend several diameters into the pipe to protect the plastic pipe during brush fires.
2. Scouring must be considered when runoff has either a high velocity or an abrasive load.

G. Information Derived from Tests

1. Green Book 210-2.3.3 (Pickle Jar Test)
   The 112 days test that simulates a 50-year sewer chemical exposure. It measures the stability of pipe materials in chemical solutions expected to be found in Los Angeles sewers and the durability of a pipe independent of ambient conditions. This test is stricter than ASTM D 543, a test often referenced by manufacturers.
2. Green Book 210-2.3.4
   Pull test for locking extensions.
3. ASTM C 501 (Modified Abrasion)
   Provides a measurement of volume loss due to abrasion. This factor is used to increase the calculated wall thickness.
4. ASTM D 638 (Tensile Properties)
   Used to find tensile strength. When lacking other tests, the tensile strength is by rule of thumb within 10 to 20% of the compressive strength for homogeneous plastics.
5. ASTM D 790 (Flexural Properties)
   Used to find flexural strength and flexural modulus.
6. ASTM D 1784, D 3262 or D 3350 (Material Identification)
   Defines the "cell classification", a standardized method to identify compounds.
7. ASTM D 2412 (Parallel Plate Test)
   The measured pipe resistance at 5% deflection between two parallel plates loaded at ½" (12.7 mm) per minute.
8. ASTM D 2837 or D 2992 (Hydrostatic Design Basis)
   The 10,000 hours (13.9 months) test intended to estimate plastic tensile creep for thermoplastic pipe. This test provides information to determine the long term modulus of elasticity.
9. ASTM D2990 (Creep)

A test to estimate creep by modifying ASTM D 638 and D 790 for long term.

H 211.41.9 DESIGN SPECIFICATION

The drawings shall identify the type of plastic pipe, minimum stiffness factor or minimum wall thickness (when appropriate), diameter, bedding, initial and long-term design stresses, depth to groundwater, and equivalent soil height used for design.

H 211.5 DELETED

H 211.6 DESIGN OF VITRIFIED CLAY PIPE (VCP)

Since VCP is manufactured with a specific assigned strength (see Section 207-8, Standard Specifications), the required supporting strength is provided by varying the bedding installation. The following formula is used to calculate the required load factor of the bedding:

$$\text{Load Factor} = \frac{(W)\ (FS)}{\text{Strength of Pipe}}$$

Where:

W = Total vertical Load (see Subsection H211.21)

FS = 1.5

Strength of Pipe = 3-Edge Bearing Test Strength

For pipes installed in trench condition, use Figure F 242.3, Sewer Design Manual, for determining the required case of bedding installation.

## H 212 REINFORCED CONCRETE BOX CONDUITS

Any closed rectangular frame having the configurations shown in Figure H 212 may be considered as a reinforced concrete box.

Box structures are commonly used or storm drains, sewers, waterline, pedestrian or equestrian subways and tunnels.

H 212.1 ECONOMY

Generally, the most economical ratio of side wall thickness to top slab thickness will be about 0.9. For small boxes, economy is often achieved by using 6-inch (152 mm) side walls with one curtain of steel.

Where the clear span of box conduits exceeds 12 feet (3.66 m), a cost study should be initiated to determine the advisability of using additional cells with shorter spans.

Analyze each project to determine the optimum number of structural design sections. It is generally economically desirable to change concrete thicknesses and reinforcing for a particular cross-section at about 2-foot (0.61 m) increments of cover where the depth of cover is varying gradually. The maximum variation in cover should not, in general, exceed 4 feet (1.22 m) for a given section. Because of the extra construction costs involved in changing structural sections, the minimum length should be about 50 feet (15.24 m). Installation by jacking or tunneling the box conduit may be more economical and practical than open trench construction. This is generally not true unless:

a. Excavation for the trench would be grater than 30 feet (9.14 m) deep.

b. Open trenching would unacceptably interfere with roadway or railway traffic.

c. Existing improvements such as large conduits, building foundations, or bridge piers may be endangered by open trench construction.

Tunneling or jacking operations are usually more expensive if done by Change Orders and should be a bid item whenever possible.

For additional requirements and information pertaining to jack ing of box conduits, see Subsection H 212.5.

## H 212.2 DESIGN LOADS

Box conduits should be designed for the dead weight of structure and vertical and horizontal earth loadss together with the combinations of vertical live load, horizontal live load, internal water pressure, and uplift pressure which give the greatest stresses in the various members of the structure. They should be checked for anticipated construction loads; in particular, the loads resulting from flooded backfill to the top of conduit combined with the dead weight of the structure should be considered. Allowable stresses for temporary construction loads may be increased 50 percent.

For design loads refer to the following Subsections:

| | |
|---|---|
| Live Load - | H 330, H 374 |
| Dead Load - | H 320, H 370 |
| Horizontal Load: | |
| Earth - | H 373 |
| Live Load - | H 374.2 |
| Water Pressure - | H 381 |

Allowable stresses are discussed in Section H 410, Concrete.

**H 212.3 DESIGN METHODS**

Box conduits less than 5 feet by 5 feet (1.52 m by 1.52 m) should be designed assuming the slabs and walls are simply supported. Box conduits greater than 5 feet by 5 feet (1.52 m by 1.52 m) should be designed as rigid frames. Computer programs are available for analysis or design of single or double cell box conduits (Section H 190).

Analysis should be based on spans from centerline to centerline of supports. Where the members are assumed to be of constant cross-section, the stiffness of the invert slab should be calculated using the thickness at the center of the span. Design moment should be that at the face of support. Correction of moments from centerline to face of support is based on the assumption that variation in shear between the face of support and centerline is linear. The design shear should be at a distanced from the face of the support (d = effective depth of the member - should not include depth of fillets). Axial thrust is considered in design of walls but not in design of top and bottom slabs. Negative reinforcement in bottom slabs should be based on the additional thickness due to invert drop. Where nominal fillets 4 to 6-inches (102 to 152 mm) are used, they should be neglected in design calculations.

For box culverts with earth cover of 3 feet (0.91 m) or less, only one-half of the negative moment due to lateral earth pressure should be used to reduce positive moment in top and bottom slabs. For Cover greater than 3 feet (0.91 m), the entire negative moment should be used.

Temperature variation is not considered to be a major factor in the design of conduits.

Where the structure is subject to substantial unbalanced lateral loads, a sidesway analysis is recommended. This does not include the normal installation where unbalanced loads result from live load application.

## H 212.4 DESIGN CRITERIA

Some criteria which affect the design and detailing of box structures are as follows:

### H 212.41 THICKNESSES OF MEMBERS

Minimum thickness of walls should be 8 inches (203 mm) where two curtains of steel are used or 6-inches (152 mm) for one curtain. The minimum thickness of top slab should be 7-1/2 inches (191 mm). Thickness of invert slab should be measured at the center of span, minimum 8 inches (203 mm).

### H 212.42 STEEL CLEARANCES

Steel clearances should be detailed on the project drawings as shown in Section H 413.

### H 212.43 LONGITUDINAL REINFORCEMENT

Except where distribution steel is required, minimum longitudinal reinforcement should be #4 bars at 18 inches (457 mm) at each layer of transverse reinforcing. Longitudinal steel should be continuous through construction joints, but not through expansion joints.

### H 212.44 TRANSVERSE REINFORCEMENT

Unbent transverse reinforcing steel should terminate 1-1/2 inches (38 mm) from concrete surfaces. In curved sections, bars are placed radially with spacing on centerline of construction for single-barrel boxes, or at centerline of outside barrel for multi-barrel boxes.

Splices in transverse reinforcement should not be permitted other than shown on the drawings. No more than 2 splices should be permitted in any longitudinal bar between transverse joints. Splices should be staggered.

Vertical reinforcing in interior walls and interior faces of exterior walls may be spliced at the construction joint at the base of the wall. Splices should be per Section H 415, splicing of reinforcement.

## H 212.45 DISTRIBUTION STEEL

When design cover is three feet (0.91 m) or less, distribution steel is placed in the top slab, transverse to the main reinforcement, to provide for lateral distribution of concentrated live loads. The amount of distribution steel per foot (m) of slab width including normal longitudinal reinforcement is equal to the following percentage of the transverse reinforcing required for positive moment:

For main reinforcement parallel to traffic:

$$\text{Percentage} = \frac{100}{s} \text{ Maximum 50 percent}$$

For main reinforcement perpendicular to traffic:

$$\text{Percentage} = \frac{220}{s} \text{ Maximum 67 percent}$$

Where S equals the square root of slab span in feet (m).

## H 212.46 FILLETS AND INVERT SLOPE

Fillets should be placed at the junction of walls and top slab, either 4 inches by 4 inches (102 mm by 102 mm) or 6 inches by 6 inches (152 mm by 152 mm), at the Contractor's option. Larger fillets may be used if structural requirements dictate.

Invert slabs should slope one inch (25 mm) from the base of walls to the center of the invert for inside widths of 20 feet (6.10 m) or less, and two inches (51 mm) for greater widths.

## H 212.47 CONSTRUCTION JOINTS

Construction joint details should be shown on the project drawings. The bottom wall joint is located 4 inches (102 mm) to 12 inches (305 mm), at the Contractor's option, above the top of the invert slab. Transverse construction joints should be spaced not more than 50 feet (15.24 m) or less than 10 feet (3.05 m) apart and joints in slabs and walls should be in the same vertical plane, placed radial or normal to the centerline.

At the beginning and ending of all pours, a complete curtain of transverse reinforcement should be placed three inches (76mm) from the transverse construction joint.

Transverse construction joints should not be placed within 30 inches (762 mm) of manhole or junction structure openings.

## H 212.48 OTHER REQUIREMENTS

In multi-barrel boxes, windows should be placed in interior walls as required to equalize flows. The interval should not exceed 500 feet (152 m). Windows should be 5 feet (1.52 m) wide and as deep as possible.

Concrete dimensions should be detailed horizontally or vertically on the profile, and parallel to or at right angles or radial to the centerline of conduit.

## H 212.5 BOX CONDUITS TO BE JACKED

Jacking of box conduits should not be specified where the cover is less than 6 feet (1.83 m), or where width of structure exceeds depth of cover.

Prior to specifying boxes to be jacked, the possibility of open cut construction should be thoroughly investigated. In many cases it is possible to close a roadway or tracks for a weekend, precast the box conduit and slide or lift it into place. This procedure, or some other may be considerably less expensive than jacking.

Where conduit is to be jacked under existing railroad tracks, the minimum jacking distance is usually 15 feet (4.57 m) on each side of the centerline of the tracks with the exception of Union Pacific tracks where the minimum distance is 10 feet (3.05 m). The method of construction should be discussed with the railroad company in advance.

Where box conduit is to be jacked in place a reinforced concrete pipe alternate should be specified if possible. Pipe is usually more economical to jack than box conduit due to the high cost of deadmen and sliding slab requirements for boxes.

Where a double box is to be jacked, an alternate of jacking two single boxes, side by side, should be specified. Single boxes should be designed with no-load on one side. Provision should be made for back-packing with grout after the boxes have been jacked in place (Section 306-2, Standard Specifications).

Shorings for jacking pits should be designed using the loadings specified in Subsection H 373.2. Allowable stresses are usually the same as used in falsework design, Section 30 3-1.6, Standard Specifications.

The entire reach of box conduit should be jacked as a unit. Therefore, a long enough reach should be available for construction of the jacking pit.

### H 212.51 REINFORCEMENT

The leading and trailing S feet (1.52 m) of box sections to be jacked should have additional transverse and longitudinal reinforcement; the area of longitudinal steel in each face of all members (except interior walls of multiple boxes) should not be less than 0.002 times the gross concrete area. In addition, the area of transverse steel in each face of slabs and exterior walls should not be less than 0.002 times the gross concrete area for the leading and trailing five feet (1.52 m).

### H 212.52 STRUCTURAL NOTES

All drawings indicating box conduits to be jacked should include the following notes:

a. The Contractor shall use jacking heads er load spreading beams of such design and size as to spread the jacking force uniformly over the entire invert section.

b. If the load spreading device or jacking head selected does not permit extension of the longitudinal steel for splicing, continuity may be maintained by doweling from the adjacent section.

c. The leading edge of the conduit shall be equipped with a jacking head securely anchored thereto. The length and details of the jacking head shall be subject to the approval of the Engineer.

d. The method of jacking, including guide rails, slabs, cradles, and other hardware will be subject to written approval by the Engineer.

## H 213 TUNNELS

Two alternative methods of construction which can be considered when jacking of a conduit is not practical are: (1) Placing precast pipe in a tunnel, and (2) Using cast-in-place RC tunnel. Tunneling consists of open face mining of the soil and lining the hole with timber, concrete, or steel supports and lagging. For precast pipe in tunnel, pipe is laid to grade and the void between the pipe and tunnel lining is backfilled, usually with pumped concrete. Tunnels crossing faults or cut-to-fill transitions in areas of anticipated settlement may require joints of special design to minimize and localize damage caused by movement.

The design loads for tunnel structures are discussed in Subsection H 372.

### H 213.1 TUNNEL SAFETY

Under the Tunnel and Safety Act of 1972, the State Division of Industrial Safety requires submittals for purposes of classifying proposed tunnels or major tunnel repair projects as to potential for encountering flammable gas. The submittals should be in compliance with the Tunnel Safety Orders, Section 8422(d).

### H 213.2 CAST-IN-PLACE RC TUNNEL

Generally, the configuration of cast-in-place RC tunnel, is arch-shaped.

The most economical shape may depend not only on hydraulic and structural requirements but also on the Contractor's familiarity with construction methods for that shape. The circular (Figure H 213.2A) and horseshoe shapes (Figure H 213.2B) are two of the more economical arch sections. Project drawings should include at least these two alternative cast-in-place sections. Where lateral resisting soil pressures are inadequate in reducing moments, a parabolic shape, creating a more pronounced peak, is an effective alternative. This shape more nearly conforms to the pressure line of the resultant loads, and a greater part of the load is carried as direct thrust without producing large bending moments.

A suggested method of design is the elastic center method as published in "Analysis of Arches, Rigid Frames and Sewer Section," Publication No. ST53 of Portland Cement Association. Based on this method, a computer program for reinforced concrete arches is available (Section H 190).

### H 213.3 TUNNEL SUPPORTS AND SHAFT SHORING

Supports and shoring need not be detailed on the plans, since a standard plan (S-254) is available which contains the pertinent tunnel notes, structural requirements, clearances, details, and design criteria for contractor design.

## H 220 OPEN CHANNEL STRUCTURES

Open channel structures are usually rectangular or trapezoidal. The rectangular "U" channel is designed as a rigid frame and the "L" channel designed as a cantilever retaining wall with a nominal nonstructural connecting or "floater" slab. The trapezoidal channel can be designed as a rigid frame, cantilever wall with invert slab, or as a wall slab supported by the earth slope. The choice depends upon the soil properties and geometry of the structure. Trapezoidal channels are not effective against uplift forces, and a careful investigation of groundwater conditions and the need for subdrains should be made.

In the type selection, consideration should be given to hydraulics, cost of right-of-way, economy of construction, properties of soil, case of excavation and backfill, groundwater level, subdrainage requirements, slope of adjacent surface, live loading, etc.

A computer program is available to analyze or design a reinforced concrete rectangular open channel with a uniform invert section. The computations are made assuming the vertical walls are cantilevered from a beam on an elastic foundation (see Section H 190).

## H 221 DESIGN CRITERIA

The structure should be designed for effects of dead load (earth and structure weight), lateral forces (earth and live load surcharges), and uplift pressures, in combination with the channel empty and flowing full. The sections should be checked for stability and soil reaction. The invert slab should be analyzed for the effect of buckling forces.

**Allowable Stresses** should be as listed in Section H 400, Concrete Design, modified for flooded backfill and flotation forces (see Subsections H 221.1 and H 221.41).

It is recommended that a soil report be obtained prior to beginning the design of the structure.

### H 221.1 LATERAL LOADING (CHANNEL EMPTY)

**Earth Pressure** - Refer to Subsection H 373. The Contractor may be permitted to use flooded backfill if recommended in the soil report. In this case, the channel section should be designed to withstand the increased load recommended in the soil report for flooded backfill. If a subdrainage system is provided or if the soil is granular and will drain well, the allowable design stresses may be increased 50 percent because of the short time duration of the load.

**Live Load Pressure** - All channels (regardless of location) should be designed for at least H10-44 truck live load surcharge. This provides nominal lateral resistance against earthquakes and sudden draw down. For channels adjacent to or within a public street easement, private or maintenance road, refer to Subsection H 374.22, for design loading.

## H 221.2 LATERAL LOADING (CHANNEL FLOWING FULL)

For the condition of the channel flowing full, assume that the soil outside of the wall offers an active pressure equal to 3/4 of the minimum design earth pressure. For example, where the walls of the channel are vertical and the earth backfill is level with the top of the channel, the channel section should be designed for 35 pcf (561 Kg/m3) EFP applied to the water side with the water surface at top of the wall.

## H 221.3 STABILITY AND SLIDING

Open channels should be analyzed for stability, soil reaction, and sliding due to the effect of lateral forces. The center invert slab should be checked for buckling forces transmitted by the adjoining retaining walls. The thrust transmitted to the center invert slab is the total horizontal force minus the effective vertical force times the sliding friction coefficient. Sliding friction coefficient = 0.40 or as noted in soil report.

Minimum safety factor against sliding = 1.5

Minimum safety factor against overturning = 1.75

## H 221.4 VERTICAL LOADS

Dead load including earth load is discussed in Subsection H 320. Reaction soil pressure on "U" channels is ordinarily based on the invert slab as a beam on an elastic foundation. However, for narrow channels, the soil pressure may be assumed to be uniform.

## H 221.41 HYDROSTATIC UPLIFT EFFECT

A thorough investigation including a soil report should be made to establish the maximum level of ground water to be expected. In addition to drilling and monitoring observation wells, groundwater records are available for this purpose at LACFCD and at the office of State Department of Water Resources.

Storm drains are frequently located in natural watercourses where the adjacent soil has a high groundwater level. When this is the case a subdrainage system should be provided. The soil report should include a recommendation on the need for subdrains.

Channels should be designed to resist flotation forces with a minimum factor of safety against flotation equal to 1.5. When designed for flotation forces, the allowable stresses may be increased 50 percent if a subdrainage system is provided or if the soil is granular and will drain well.

**Rigid Frame "U" Section** - Rigid frame sections not provided with subdrainage should at least have adequate weight and strength to withstand hydrostatic forces consistent with the maximum level of external groundwater. The structure should be designed for the full flotation force. Sufficient weight is obtained by an extension of the wall heels.

With a subdrainage system, uplift pressures may be reduced to the extent warranted by the soil permeability and effectiveness of the subdrainage system. For design purposes, the minimum groundwater level should be assumed to be at least 2 feet (0.61 m) above the invert for channels without subdrainage, or at the top of the outlets for channels with a subdrainge system.

**"L" Wall Sections** - For "L" sections, the invert slab has little weight and must be well protected from uplift pressures. Generally, continuous or isolated "heel drains" and "floor slab drains," with outlets at intervals into the channel are used. Heel and floor slab drains may be interconnected, or sand and gravel blankets may be desirable, depending upon severity of uplift. Vertical cutoff walls can be spaced along the channel reach to stiffen and localize damage to the invert slab.

**Trapezoidal Sections** - The invert slab and bank linings of trapezoidal sections are incapable of resisting much hydrostatic head. Consequently, the subdrainage system must be chosen to assure that the upward pressure will be less than the weight of lining.

## H 221.5 LOW FLOW CHANNEL

Low flow channels are small rigid frame 'U" or "V" sections constructed in the invert subgrade at the centerline of the channel. They concentrate low flows, promote adequate velocities for the movement of sand, and provide an effective location for outlet of subdrainage systems. They are also useful during construction to eliminate nuisance water (see Figure G 635, **Storm Drain Design Manual**).

**H 221.6 SUBDRAINAGE SYSTEMS**

Effective subdrainage increases the soil bearing capacity, relieves lateral and upward hydrostatic pressures, and allows the use of more economical structures.

Based on a study of groundwater levels to be expected and soil permeability, longitudinal heel drains, and longitudinal and transverse floor slab drains should be placed, and interconnected to reduce hydrostatic heads to acceptable levels. The use of filter blankets of sand and gravel is a desirable method of equalizing pressure and usually results in minimizing the number of pipe drains required.

Subdrainage material used in subdrainage systems is as follows:

Filter Material:

(1) Sand - Sand for Portland Cement Concrete

(2) Sand and Gravel Mixture - Minimum of 40 percent sand mixed with crushed rock.

Drain Material - as specified in Figure G 635, Storm Drain Design Manual.

Gravel Blanket - Crushed Rock.

The use of plastic filter cloths in subdrainage system should be a definite consideration. Refer to FHWA Manual currently being prepared "Use of Engineering Fabrics in Transportation Related Applications".

Subdrainage material may be specified on the plans or in the Special Provisions.

**H 221.61 HEEL DRAINS**

Heel drains consist of longitudinal perforated pipe, laid with perforations down and cased in drain material or in a sand filter. They are placed on the wall heels of rectangular sections, or near the outer invert base in the case of trapezoidal sections (see Figure H 211.6B and Figure G 635, **Storm Drain Design Manual**). They should be located at the lowest level consistent with outlet requirements, since there purpose is to protect invert slabs and other linings designed to resist little uplift. Acceptable types and sizes of conduits are listed in Section G 635**, Storm Drain Design Manual**.

Heel drains discharge through spigot ells directly into the channel at specified intervals. When the channel width is greater than 40 feet (12.19 m), heel drains may be connected to lateral drains in the invert drain. Heel drains should be continuous except for a gap of about 3 feet (0.91 m) at 200-foot (61 m) intervals to isolate and limit the extent of blockage or other problems. Pipes are laid with the bell ends upstream. The bell at the upstream end of each drainage unit should be entirely closed by a mortared-in precast concrete cap (see Figure H 221.6A).

### H 221.62 WEEP HOLES

Weep holes are used in low water table areas where uplift is minimal, but where a clay or silt (capillary) type soil is encountered (see Figure G 635, **Storm Drain Design Manual**).

### H 221.63 FLOOR SLAB DRAINS

Floor slab drains consist of longitudinal pipe, lateral pipe, or a combination of the two laid in gravel-filled trenches or t previous blankets in thie channel floor subgrade (see Figure G 635, **Storm Drain Design Manual**). They are used where substantial groundwater flow is expected.

### H 221.64 SAND AND GRAVEL BLANKET

A sand and gravel blanket is often used in a soft subgrade of silt or clay which is moist from capillary action. The blanket can be lightly compacted by rolling, but, disturbance of the subgrade should be avoided. A soil report evaluating the need for this blanket should be obtained from the Geology and Soils Engineering Section, Construction Division.

## H 230 BRIDGES - GENERAL

General and specific features of structural design of superstructure portions of vehicular, pedestrian, bikeway, and railroad bridges are discussed in this section. Criteria for wingwalls, piers, abutments, and foundations are discussed in Section H 500, Foundations and Retaining Structures. As a design aid, refer to Caltran's manual "Bridge Design Details”.

For other applicable design related requirements, see the following:

| | |
|---|---|
| Type Selection | Subsection H 152.4 |
| Design Standards, Codes, and Specifications | Section H 120 |
| Design Loads and Load Distribution | Section H 300 |
| Materials and Design Specifications | Section H 400 |

## H 231 CLEARANCES AT STRUCTURES

Minimum standard vertical and horizontal clearances from obstructions to traveled roadway, railway, falsework, channels, and equestrian crossings are as follows.

### H 231.1 TRAVELED ROADWAY

### H 231.11 VERTICAL CLEARANCE

The minimum design vertical clearance between finished roadway surface and overhead structure soffit or other obstruction is 15.25 feet (4.65 m) including 3 inches (76 mm) allowance for future resurfacing of roadway. This provides about 14.41 feet (4.4m) clearance above raised medians and sidewalks.

Vertical clearance to minor overhead structures, such as pedestrian and bikeway bridges should be increased 2 feet (0.61 m) or more. However, if a minor structure is flanked on both approaches by other structures having lesser clearance, then the highest of these approach clearances should govern.

### H 231.12 HORIZONTAL CLEARANCE

Provide as much horizontal clearance to fixed objects as economically feasible in order to obtain greater safety as well as esthetic quality.

The minimum horizontal clearance from curb (or edge of traveled way if no curb exists) to bridge piers, abutments, retaining walls or other obstructions is as follows:

a. Two-way traffic - 6 feet (1.83 m) minimum on each side.

b. Divided roadways - 4.5 feet (1.37 m) minimum on the left and 6 feet (1.83 m) on the right in the direction of traffic.

Additional horizontal clearance should be provided where necessary to meet sight distance requirements.

### H 231.2 RAILROAD

The minimum clearances to structures and obstructions over, adjacent to, or under railroad tracks are established by the Public Utilities Commission (PUC) of the State of California and are set forth in their General Order No. 26-D, in effect since February 1, 1948, and in subsequent orders. The horizontal clearances illustrated in Figures H 231.2A and H 231.2B are to structures and parallel tracks usually encountered in bridge projects and are those adopted by the Division of Structures.

California Department of Transportation. These clearances should be used on all projects unless increased by the individual railroad company (see Section H 270). Unusual cases of obstructions that require less clearance, if not covered in PUC's General Order NO. 26-D, will require approval of the Commission.

The minimum vertical clearance over mainline railroad tracks is 23 feet (7.01 m). In some cases additional vertical clearance may be required. For example, Union Pacific Railroad Company requires 26 feet (7.93 m) clearance for future electrification of its mainline track between Ogden, Utah and Los Angeles.

## H 231.3 FALSEWORK CLEARANCE

Where construction falsework is to be used over a traveled way, allowance should be made for falsework depth as shown below:

**FALSEWORK ALLOWANCE**

| **Clear Span of Falsework** | **Minimum Falsework Depth** |
|---|---|
| 20' (6.10 m) (One lane + two 4' (1.22 m) shoulders) | 1'-7" (0.48 m) |
| 32' (9.75 m) (Two lanes + two 4' (1.22 m) shoulders) | 1' -9" (0.53 m) |

When possible, a minimum vertical clearance of 14.5 feet (4.42 m) and a minimum horizontal clearance of 4 feet (1,22 m) from edge of traveled way to falsework posts should be provided. Also, a minimum width of falsework opening at least 20 feet (6.10 m) wide for a single lane or 32 feet (9.75 m) wide for a double lane should be provided. A substantial barrier to protect falsework supports from traffic may be needed. The requirements for the temporary railing should be shown on the plans or included in the provisions. "Impaired vertical clearance" signs should be posted on all approaches to falsework with less than 15 feet (4.57m) overhead clearance. Temporary vertical clearances of less than 14.5 feet (4.42 m) may be used for falsework in those cases where necessary for economy of construction. In addition to "Impaired Vertical Clearance" signs, all approaches should be fitted with light weight chains or other "tell-tales" strung across the roadway at the height of falsework so as to noisily warn drivers of overheight vehicles when struck.

Less than 13.5 feet (4.12 m) minimum falsework clearance is not recommended in any case.

# H 233 SEISMIC DESIGN CRITERIA

The following seismic design criteria apply to all bridges and other similar structures. Additional criteria may be found in BPDM, Vol. I. Refer also to Section H 360.

## H 233.1 RESTRAINING FEATURES AND CONNECTIONS

Provide restraints which limit total movement in all directions at hinges, rollers, rockers, and other movable bearings so as to prevent members from sliding off supports. (Examples of restraints are keys, vertical bolts in slotted holes, horizontal tension bolts, and wire strand.) Such restraints should permit calculated movements (such as temperature and shrinkage) but prevent relative movements between member and support from exceeding design magnitude. Longitudinal restraints are not required at end abutments where girder seat widths are at least 10 percent of abutment height (1'-6" (457 mm) minimum). Refer to details on Figures H 233.1A, B and C.

## H 233.2 COLUMNS

Spirally reinforced columns should be used in lieu of tied columns wherever possible. Minimum spirals or ties are No. 4 at 3-1/2-inch (89 mm) spacing, anchored with 10-inch (254 mm) tails.

No lap splices should be used in main column reinforcement where column height is 30 feet (9.14 m), or less. For column heights greater than 30 feet (9.14 m), 60 diameter laps may be used for No. 11 or smaller bars but these should be located at least 10 feet (3.05 m), from ends of columns. Reinforcement larger than No. 11 should be spliced by welding or approved mechanical splices.

## H 233.3 FOOTINGS

Minimum recommended top reinforcing in spread footings and pile caps is 20 percent of required bottom reinforcing (No. 5 at 12 (305 mm) minimum). Refer to BPDM, Vol. I for additional requirements.

## H 233.4 EARTH BACKFILL

Earth backfill behind abutments and retaining walls need not be assumed to contribute to Earthquake loads and, where practical, should not be assumed to resist earthquake loads. Where it is impractical to carry earthquake loads into the foundation (as in the case of a high cantilever abutment) and some passive earth pressure is assumed, provision should be made for wall movement necessary to develop passive resistance. Connections or joints between superstructure and abutment should be designed with consideration given to large translation and rotation.

In addition to cantilever loadings, abutments should be designed to act as simple spans between footing and superstructure under active soil pressure.

### H 233.5 PILE FOUNDATIONS

Stresses in piles should be checked under combined dead loads and earthquake loads. Capacity of piles need not be governed by deflection. Passive soil pressure applied to piles, as well as depth of "fixity," should be determined by the soils engineer. Lateral loads in excess of pile bending capacity may be resisted by pile batter. Horizontal component of pile batter may be added to bending capacity.

### H 233.6 OTHER FACETS

a. Provide for maximum relative movements between independent structures by use of isolation joints. Clearances should be at least the sum of maximum deflections of both structures.

b. Bearing friction due to gravity loads should not be assumed to resist lateral earthquake loads.

c. Design all structures against collapse when subjected to "large" ground motions even though major structural damage might occur. Joints and connections should be detailed to yield gradually and resist total separation.

## H 234 BRIDGE RAILINGS

Following are general types of bridge railings commonly encountered.

a. Vehicular bridge railings, Subsection H 243.

b. Combination vehicular and pedestrian bridge railings, Subsection H 243.

c. Pedestrian bridge railings, Subsection H 254.

d. Bikeway bridge railings, Subsection H 262.

e. Railroad bridge railings, Subsection H 278.

The type and finish of railing to be used on each bridge should be determined by the engineer and the architect during type selection (Subsection H 152.4). In selecting the type of railing, consideration is given to anticipated volume of pedestrian traffic and proximity of schools and playgrounds. Where pedestrians are expected, the minimum dimension of any opening should not exceed 9 inches clear.

Careful attention should be given to the treatment of railings at approaches. Exposed rail ends and abrupt changes in geometry should be avoided, and a smooth transition provided to divert impacting vehicles.

On bridge widening projects, if the exisiting railing satisfies design requirements and is in good condition, it could be reused on the new structure. If the existing railing would require alteration or refinishing, the designer should compare the cost and appearance with that of providing a new and possibly more modern railiny.

Railing expansion joints are provided to accomodate temperature change. In the railing spanning deck joints, the railing joint is increased to accomodate deck movement.

## H 240 VEHICULAR BRIDGES

Design considerations included in this section are: Roadway width, sidewalks, curbs, deck surfacing, girder bearings, railings, expansion joints, drainage, and utilities. (See also Section H 230, Bridges - General).

## H 241 BRIDGE ROADWAY WIDTH, SIDEWALKS, CURBS, AND DECK SURFACING

### H 241.1 ROADWAY WIDTH

Design roadway widths, like other geometric features of alignment, should be determined in close cooperation between the District Engineering Office and Structural Design Office. The width of roadway is the clear width measured normal to centerline of roadway between curbs or between interior faces of railings on bridges without curbs.

The width of the bridge roadway should conform to the width of approach roadway (or planned future width). Standard street dimensions for City streets should be used in most cases. Lesser widths can be used where left turn pockets are not required.

If approach curbs are not provided, the bridge roadway width will generally equal the full shoulder width of the approach roadway. The interior face of railings should be at least 6 feet (1.22 m) outside the approach traveled way.

For bridges with spans greater than 100 feet (30.48 m) where parking will not be allowed on the bridge, the reduction of parking lanes to shoulder widths should be considered. Where left turn pockets do not encroach onto the bridge, the reduction of median widths should be considered. Design roadway widths less than standard should be approved in advance by Caltrans if federal funding is to be used. For these projects, lane widths are at least 11 feet, shoulder widths are at least 8 feet, and median widths at least 2 feet. In rare cases where existing buildings or other costly right-of-way interfere, lesser widths may be approved for federal funding.

## H 241.2 SIDEWALKS

When bridge sidewalks are required, the minimum width should be 7 feet (2.13 m) measured from curb face to face of railing. To match approach sidewalks, the use of wider sidewalks up to 10 feet (3.05 m) may be advisable on short span bridges.

Crushed aggregate base is placed under the sidewalk to provide space for utility placement. See Figure H 241.

## H 241.3 CURBS

The recommended minimum height of curbs on edges of bridges is 10 inches (254 mm ). Curbs up to 12 inches (305 mm) may be used to provide for utilities or to match existing curbs. Median curbs should conform to those of the adjacent street, usually 6 or 8 inches.

When bridge sidewalks are not required for pedestrian traffic, a minimum sidewalk or safety curb 2 feet (0.61 m)- wide should be used.

## H 241.4 DECK SURFACING

Generally, the concrete deck serves as the wearing surface on new bridges. On bridge widening projects, asphalt with a minimum thickness of 1-1/2 inches (37.7 mm) may be used to match the surface material of the existing structure. The designer should also consider the use of asphalt surfacing on short-span bridges (less than 50 feet (15.24 m) clear span) to simplify the finishing operations required for the bridge deck. Modification of deck joint details may be required to accommodate asphalt.

# H 242 GIRDER BEARINGS

Rotational and/or translational bearings are provided at girder supports to allow for movements due to temperature, creep, shrinkage, girder deflection or rotation. Girder bearings should be protected from dirt and water and the design should provide for inspection and cleaning. At least one point of girder support should be fixed against longitudinal translation. Where more than one point is fixed, the bearings (and structural supports) must be designed to resist the resulting forces.

All bearings should be provided with side bars, anchor bolts, shear keys, or similar devices to resist earthquake, wind and other forces acting laterally on the structure. Expansion joint filler or neoprene should be specified at the sides of concrete shear keys to minimize the danger of binding and progressive failure.

## H 242.1 STEEL GIRDERS

Steel girder supports can be fitted with rollers, rockers, sliding bearings of teflon or other low friction material, or elastomeric bearing pads. They are often restrained against translation with anchor bolts through the bottom flanges (in slotted holes at the sliding end).

## H 242.2 CONCRETE GIRDERS

As for steel girders, concrete girders may require an expansion device at the supports. These are normally placed under the end diaphragms. Where the diaphragm is cast monolithic with and extended below the soffit of the superstructure to rest on piles or on a footing or pile cap (end diaphragm abutment), bearings may be deleted. In that case, thermal movements are absorbed by the earth behind the end diaphragms. For spans greater than 150 feet, elastic shortening due to post-tensioning and rotation of the girder at the bearing must be provided for by installing a greased slide surface and neoprene pad bearing. Figure H 242 suggests guidelines for use in controlling abutment movement.

# H 243 RAILINGS

In selecting railings for vehicular bridges, the length of the bridge and the volume and type of anticipated pedestrian traffic should be considered. On short bridges less than 20 feet long or culverts (see Subsection H 112. Definitions), pedestrian railing is adequate. On longer structures a traffic barrier railing or combination railing is necessary in order to meet the requirements for federal funding. Railing design requirements are listed in the AASHTO specifications. A railing base to accommodate the railing selected should be designed and detailed on the plans. To provide a maximum of uncluttered sidewalk space the possibilty of supporting electroliers on the railing base should be considered and discussed with the Bureau of Street Lighting. Design loads for railings or railing bases are referenced in Subsection H 333.1.

Typical vehicular railings are illustrated in Figures H 243A, B, and C. Except for types B and E, standard detail drawings are available from the Structural Engineering Division.

**Type A** - This steel pipe railing is not intended to meet the strength requirements for traffic barrier railing. Its use should be limited to situations where a reduced strength is acceptable such as for spans less than 20 feet. It may be used in rural areas where little pedestrian traffic is expected, and should not be used to prevent pedestrian access to areas outside of the roadway. This railing has the advantage of being "self-cleaning" (open at bottom).

**Type B** - This aluminum barrier railing may be used in urban areas where no pedestrian traffic is expected, but should not be used to prevent access to areas outside of the roadway. Due to the higher cost of this railing, its use should be limited to situations where special architectural treatment is required.

It is identical to Type C railing with the vertical pickets omitted. It is installed on a railing base (traffic barrier) minimum 1'-2" (0.36 m) wide by 1'-6" (0.46 m) high (above sidewalk grade).

**Type C** - This aluminum barrier railing is Type B railing with the addition of vertical pickets. It should be used in urban areas where pedestrian traffic is anticipated and special architectural treatment required. The railing base is similar to Type B.

**Type D** - This steel barrier railing is installed on a railing base (traffic barrier) with minimum height of 1'-5-1/2" (0.46 m). This is the preferred railing where sidewalks are included on bridges.

**Type E** - This steel barrier railing with chain link fencing is installed on a railing base (traffic barrier) with minimum height of 1'-5-1/2" (0.45 m) and is used where objects being thrown to the roadway below may be a problem. Details are similar to State Standard Plan B11-42, Barrier Railing Type l2, and B11-9, Chain Link Railing Type 4 (January 1975).

**Picket Railing** - This is not a barrier railing. However, it can be used in conjunction with a railing base designed as a traffic barrier with a minimum height of 2'-3" (0.69 m).

On the standard drawing, which is available from Structural Engineering Division, the designer should complete the table of "Railing Data" as follows (refer to Figure H 243.1 for Tables I and II):

1. H - Railing height (feet)(m).

2. Material - Specify galvanized steel (A53).

3. Picket and Post - Indicate "standard" or "extra strong" pipe (See Table I).

4. Post Spacing - Maximum post spacing depends on the railing height (See Tables I and II).

5. Dimension "a" - Use 6" (152 mm) for H less than 5'-6" (1.68 m) or 9" (229 mm) for H equal to or greater than 5'-6" (1.68 m).

6. Panels - Specify the shape of panelsas "straight" or "curved" (see Table II). For radii of 30 to 50 feet (0.14 to 15.24 m), both straight and curved panel options should be shown on the plan.

**Temporary Railings** - Where a temporary traffic barrier is needed during construction (bridge widenings, protection of falsework, etc.), the use of State of California Standard Plan B11-30, Concrete Barrier Type Railing, should be considered. Precast segments are available and effective for this purpose, but are heavy and should be used on structures only after investigation.

## H 244 EXPANSION JOINTS

It is necessary to provide for movements and stresses induced in bridye structures by changes in temperature or concrete creep and shrinkage. The effects of temperature on a concrete abutment in contact with earth are less than effects on the superstructure.

Structure geometry and orientation of rigid piers and columns affect the magnitude and direction of movements of bridge superstructures. Superstructure movements of a skewed bridge with wide piers will tend to be normal to the piers rather than parallel to centerline of roadway. Expansion joints, shear keys and restrainers should be designed to accommodate this tendency.

The calculated anticipated total joint movement from the narrowest to the widest joint opening in inches (mm) due to the effect of thermal, creep, and shrinkage forces is called movement rating (MR). The MR calculations can be checked by using the table shown in Figure H 244. The calculated MR figures should be rounded upward to the next l/2-inch.

## H 244.1 DECK JOINT TYPE

The following types of joints are commonly used on bridge decks: Open joint, open joint with expansion armor, unsealed jojnt with joint filler, and sealed joint with joint filler.

**Open Joint** - This type should be used only where erosion of slopes, staining of concrete or dropping of debris below the bridge is not a problem. The faces of open joints should be battered 1:12, with the bottom of the joint larger than the top to allow debris to fall through without binding and damage to concrete.

**Open Joint with Expansion Armor** - This type passes water and debris like an open joint and use should be limited to bridge widening or repair projects where it is desired to match an existing joint, or new construction where expansion cannot be accommodated by one of the other joint types. Open joints in the deck should be 1/2 inch (12.7 mm) greater than the anticipated movement.

**Unsealed Joint with Joint Filler** - This type accomodates only very small movement and should be used only where it is unlikely that foreign material will be forced in to the joint (such as a longitudinal joint located at the crown of the roadway section).

**Sealed Joint with Joint Filler** - This type is preferred for all transverse deck joints unless special conditions or large movements warrant the use of a different type. Sealed joints are schematically shown in Figure H 244.1, and detailed on a standard sheet, "Joint Seal Detail," available from Structural Engineering Division for inclusion in the contract plans. When this standard sheet is to be used, calculate the movement rating (MR) and contributing length of bridge deck and tabulate these data in the "Joint Data" table of the standard sheet for each joint location. The MR is needed by the contractor to select the size and kind of seal to be used. The contributing length is needed in order to calculate the width of saw cut at the time of installation for Type B seals.

Type A seals are limited to a maximum of 1/2" (12.7 mm) MR. Type B seals are limited to a maximum of 2" (51 mm) MR. Sealed joints having a MR greater than 2" (51 mm) are specially designed and detailed on the plans. The Standard Sheet is not applicable in this case. Dimension "a" for specially detailed joints shall be MR less anticipated shortening, where S = 2, 1.5 or 1.0 for summer, fall/spring or winter, respectively. Commercial sealed joints for large movements are available, but should be investigated before use.

### H 244.2 JOINT FILLER

Premolded joint filler or expanded polystyrene are acceptable compressible materials for use as a filler in expansion joints. Expanded polystyrene should be used in joints where a large thermal movement is expected. The thickness of premolded joint filler should be twice the calculated movement from average to the fully compressed position. The thickness of expanded polystyrene should be 10 percent greater than the calculated movement from open to fully compressed position. The filler type should be shown on the plans or specified.

### H 244.3 WATERSTOPS

Waterstops should be installed with all expansion joints over a horizontal girder seat or above a roadway, pedestrian way, or other improved or unimproved area where water or debris dropping down could create an erosion problem, other maintenance problem, or nuisance to persons or property.

Waterstop is manufactured either from neoprene or from polyvinyl chloride (PVC). When used in conjunction with the standard sheet "Joint Seal Detail," tabulate "yes" in the "Joint Data" table under "Waterstop Required."

## H 245 BRIDGE DRAINAGE

### H 245.1 SURFACE DRAINAGE

Cross-drainage of bridges is controlled by a crown or tent roadway surface normally centered on the bridge except for one-way traffic or superelevated bridges where a cross-slope is used. The crown or cross-slope should match that of the approach pavement except on long bridges, where a minimum 1-1/2 percent slope should be used and need not match that of the approach pavement.

Longitudinal drainage is provided by camber or gradient. Storm water should be directed to inlets located off the bridge structure where feasible. Where deck drains are required, they should be inconspicuous and should discharge at locations that will not damage property. For bridges over channel, the use of drop drains should be considered.

### H 245.2 GUTTERS

In general, concrete gutters need not be constructed on bridges where an asphalt wearing surface is provided. Some exceptions are:

a. Concrete gutters should be constructed on bridges when there will be frequent or continual presence of water in the gutter which cannot be feasibly intercepted upstream of the bridge structure. In such cases, gutters should be used to prevent gradual erosion and deterioration of the asphalt.

b. Concrete gutters may be constructd on short-span bridges less than 50 feet ( 15.24 m), where continuous asphalt paving across the structure would facilitate maintenance resurfacing.

## H 246 UTILITIES ON STRUCTURES

Bridges should be designed to accommodate existing and future utilities to the extent feasible. The designer should contact all utility companies which have existing facilities in the bridge area or might reasonably be expected to install a utility at a later date (see Subsection H 154.2 and Figure H 154.2, Utility Location Form Letter). Openings in diaphragms and abutments, higher curb faces, wider pile spacing or other similar design adjustments should be made to accommodate utilities on the structure unless it is determined that such placement is not in the public interest.

### H 246.1 UTILITY PLACEMENT

In order to preserve bridge architecture and neighborhood aesthetics, it is good practice to avoid the attachment of utility conduits to the outside of bridges where they may be visible to the public. Other locations, such as interiors of box girders, are more satisfactory from the standpoint of structural safety, appearance, and the avoidance of interference with future construction.

Small conduits (approximately 8 inches (203 mm) and less in diameter) may be placed within the crushed aggregate base beneath the sidewalk. Conduits may also be placed within the cells of box girders or hung between the girders of T-beam or steel girder bridges. The location of all existing, new and relocated utilities should be shown on the contract drawings.

## H 246.2 UTILITY ENCASEMENT

All conduits on bridges carrying volatile gas or liquid should be encased throughout the length of the bridge structure. Conduits bedded in earth (such as earth backfill above box culverts) need not be encased if the conduit is at least the normal depth, 2.5 feet (0.76 m) cover.

When encasement is required, a sleeve approximately 3 inches (76 mm) larger than the outside diameter of the pipeline should be used. The casing is vented at each end of the bridge through a standpipe-type vent or opening into the approach fill so that no pressure buildup is possible. The exposed portions of the vent should be located outside the sidewalk in areas which will be inconspicuous. Encasement and vent requirements are noted on the contract drawings similar to the following:

> "4 inches (102 mm) SCG in 8 inches (203 mm) casing: Casing shall be vented at each end of the bridge structure with a standpipe-type vent or opening into approach fill. Exposed portion of vent shall be located outside the sidewalk in inconspicuous areas."

The length of encasement, location of vents, and other details need not be shown on the contract drawings, but should be detailed on the construction permit drawings prepared by the utility company.

## H 246.3 UTILITY ACCESS

Where conduits are carried within the cells of box girders, access to such cells should be provided when requested by the utility company. Access may be provided by a manhole in the deck (preferably in a sidewalk area) and manhole openings through the diaphragms. The number of manholes in the bridge deck or sidewalk should be kept to a minimum. In structures carrying a large number of utilities, abutments with galleries to provide access to the girder cells may be desirable. A standard sheet of "Miscellaneous Bridge Details" is available from Structural Engineering Division for inclusion in the contract plans which includes details for utility access openings.

## H 246.4 FUTURE UTILITIES

Requests are frequently received to place utilities on existing bridges where no provisions were made for carrying additional utilities. Often, the utilities are too large to place in the sidewalk area, or other existing utilities occupy all available space. In anticipation of this problem, provision should be made for unanticipated future utilities on new bridges where this can be done at a reasonable cost. Usually, openings for future utilities should be located in the exterior bay on each side of the structure. This will allow excavations to be made in the approach sidewalk areas rather than within the traveled way. Openings in caps and diaphragms should be a minimurn of l.5 feet (0.46 m) high by 2 feet (0.61 m) wide, if possible, to provide crawl space. Removable panels should be used so that conduits can be placed through abutment backwalls or end diaphragms with a minimum of concrete removal and expense. Suggested locations for future utilties are shown in Figure H 246.4.

## H 247 OTHER DESIGN SITUATIONS

The designer is often confronted with the following design problems:

a. To design a temporary bridge structure using limited financing, to be replaced with a future permanent structure.

b. To design a bridge less than ultimate width using limited financing, to be widened at a future time.

c. To design the widening of an existing bridge to meet new street width requirements, or implement an originally planned ultimate width.

### H 247.1 TEMPORARY BRIDGES

Unless the scope of the temporary structures is much less than the permanent one, it is economically advantageous to construct the permanent structure now rather than a temporary structure to be reconstructed at a later date (see Subsection H 152.4, Project Type Selection). Temporary structures have a way of becoming "permanent" and, except in emergencies, design standards should not be compromised for temporary construction.

### H 247.2 BRIDGE CONSTRUCTED NOT TO ULTIMATE WIDTH

Where less than the ultimate superstructure width is to be constructed, consideration should be given to placing piles, footings, or strengthened channel walls for the ultimate width. The additional cost of doing that work now is usually much less than a future remodeling. Removable railings and curbs as well as a convenient join line should be provided.

### H 247.3 WIDENING OF EXISTING BRIDGE

In the design of a bridge widening, the features of the existing bridge may limit the alternatives available to the designer.

Aesthetic compatibility may be a prime factor. Other control features may be the location of existing expansion joints, piers, and abutments. The long-term dead and live load deflections and settlements of the new structure must be compatible with those of the existing structure.

## H 247.31 DEFLECTION

Deflection and camber of bridge deck members is discussed in Subsection H 232. For bridge widening, additional provisions should be made to accommodate the differential deflection between the new and old structures. Failure to do so may result in a ridge or vertical offset in the roadway, cracking of the connecting deck slab, spalling of the deck, or transfer of new dead and live load to the existing structure, with possible damage. If applicable, the top of the new deck should be constructed higher than the existing so as to match after the new span deflects. Closure pours, where used, are generally placed not sooner than 14 days after the falsework has been released in order to minimize the effects of long term deflection and settlement.

The effects of deflection can often be minimized by using prestressed concrete in the widening, resulting in little or no dead load deflection.

## H 247.32 PIER AND ABUTMENT SETTLEMENT

An existing structure will ordinarily not be subject to settlement of its footings due to widening, but it is likely that footings under the widened portion will settle. A positive attafhment of the existing footing to the new footing may cause ove in the old footing. However, the consequences of such overstress may not be as critical as those of differential settlement and its effects on the bridge superstructure. Therefore, it is generally preferable when designing a widening to provide a positive attachment by chipping keys or roughening the concrete surface on the existing footing and abutment as well as providng dowels.

There are some types of footings and abutments which should not have positive attachment. Cantilever abutments which would tend to have a significant lateral deflection should not be attached to the existing abutment. Piers and abutments with spread footings where a large settlement is expected should not be attached to the existing structure. Suitable provisions for the settlement should be provided in the design and details. Flexible joints at join lines and provision for future pavement leveling courses may be necessary.

### H 247.33 ADDITIONAL CRITERIA

Detours are not normally available for use during construction and it is generally necessary to construct the widening in the presence of existing traffic. If feasible, the existing curb should remain in place. Access for pedestrian traffic should be provided. Temporary roadway widths should be shown on the plans.

When widening or remodeling prestressed girder bridges, holes should not be drilled into the prestressed concrete, nor should the prestressed concrete be broken or cut except at carefully selected locations. Th ese should be detailed on the plans to avoid damage to tendons or overstressing of concrete.

Longitudinal deck joints which allow a vertical displacement between old and new structures should be avoided if possible. If such a joint is necessary, it should be located at the lane line. Ridges which develop at the joint due to deflection or settlement should not exceed 3/8 inch (9.5 mm). The width of a longitudinal deck joint with sealant should not exceed 1/2 inch (12.7 mm).

Monolithic attachment of the widened and original decks by lapping or welding reinforcing steel provides a better riding surface, is better appearing, and reduces maintenance problems. Separate closure pours should generally be made after falsework removal. Figures H 247.33A, B, and C show typical bridge deck widening details that should be incorporated where the design permits.

## H 250 PEDESTRIAN BRIDGES

The design features of pedestrian bridge superstructure included in this section are: Width of bridge deck and approaches, railings, and vibration control. See also Section H 230, Bridges - General.

## H 251 WALKWAY WIDTHS

Walkway width should be a minimum of 7 feet (2.13 m) clear between railings posts. The walkway should be wide enough to accommodate the expected number of pedestrains and should be as wide as the approach sidewalks where feasible.

For combined pedestrian-bicycle pathways (Type III, Subsection H 261), the required width need not ordinarily be increased over that required for a bicycle bridge with two-way traffic. However, the required width should be verified by the Central Engineering District or appropriate district office.

## H 252 BRIDGE DECK SURFACING AND CURB

No allowance for a wearing surface need be made. A brush or broom finish is desirable for adequate traction on all concrete decks.

It is desirable that there be no curbs or railing bases. This will facilitate self-cleaning of the bridge.

## H 253 RAMPS AND STAIRWAYS

The use of ramps instead of stairs is desirable. Ramps are convenient to pedestrians with handicaps or with strollers, bicycles, wheelchairs, or other wheeled devices. The slope of ramps should not exceed 12 percent. With a slope exceeding 8 percent, an abrasive aggregate finish and a handrail should be used. On ramps with a slope of less than 8 percent, a broom finish should be used.

Stairways should be used only if available space is inadequate for ramps. Handrail should be provided on both sides of the stairway, and a center handrail should be considered if the stairway width is greater than 7 feet (2.13 m). The stairway should have nonslip treads, and all landings should have a broom finish.

## H 254 RAILINGS

For a general discussion of bridge railings, see Subsection H 234. Railings with a 5'-6" (1.68 m) minimum height should be provided along the outside edges of pedestrian bridges, unless a lower height is approved during project type selection. The minimum dimension of any railing opening should not exceed 9" (229 mm).

Wherever possible, a railing which allows self-cleaning is preferred; i.e., rail ings without curbs or bases to collect debris.

The lateral deflection of railing should be considered in the design and railing made stiff enough to discourage vandalism.

The two basic types of pedestrian railings in general use are chain link and picket railings. Chain link should be considered where prevention of objects being thrown to the roadway below is a requirement. Refer to State of California, Department of Transportation's Standard Plans (197 5), B11-8, B11-28, and B11-52 for chain link railing details. Picket railings should be used for most pedestrian bridges.

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## H 255 CLEARANCES AND EXPANSION JOINTS

Clearances of pedestrian bridges over traveled roadways, railroads, and channels are discussed in Subsection H 231.1.

Expansion joint requirements are the same as those for vehicular bridges, Subsection H 244. The use of joints with offsets, openings or depressions which might permit tripping or entry of high-heeled shoes should be avoided as should joints of soft or penetrable material.

## H 256 DECK DRAINAGE

Wherever possible on walkway deck, a 1 percent cross slope or crown should be provided for drainage.

Longitudinal drainage may be necessary where frequent washing is anticipated, especially with a roadway below. This may be provided by camber or gradient used in conjunction with an inverted crown section, curb or railing base. Where feasible, the accumulated storm water should be directed to inlets located off the bridge structure.

## H 257 VIBRATION DESIGN CRITERION

The stiffness and natural frequency of pedestrian bridges should be selected such that vibrations caused by human impact are not objectionable or annoying to the pedestrian.

All pedestrian bridge design should be checked for these qualities by utilizing a Reiher-Meister Graph for "Human Response to Vibrations" as modified by K. H. Lenzer and as shown on Figure H 257A.

The ranges of response which should be used in the design are as follows (see graph):

a. **Not Perceptible and Slightly Perceptible -** These two ranges are acceptable for design.

b. **Clearly Perceptible** - This range is marginal, and acceptability depends on where the response falls within this range and the structure damping properties. Otherwise, a revision of the section properties may be necessary, especially for steel or prestressed concrete structures where damping effects are slight.

The two variables of dynamic response plotted on the graph in Figure H 257A are "Natural Frequency" and "Dynamic Deflection."

**Natural Frequency** - This value is the first mode natural frequency of the structure. Non-prismatic sections may require computer analysis or approximation by a numerical method. Prismatic sections may be analyzed by one of the elastic beam vibration formulas. The equation for a simple beam is shown in Figure H 257B.

**Dynamic Deflection** - The theoretical maximum dynamic deflection of a pedestrian structure (resonance condition) can be assumed to be approximately 74 percent (empirical value) greater than the maximum static deflection. The assumed static load (P) should be 300 pounds (1334N), which includes this effect of impact.

Static deflections of commonly used beams (prismatic sections) are shown in Figure H 257.1.

## H 260 BIKEWAY BRIDGES

Design considerations included in this section are: Classification as to type, railings, bridge widths, and others. The discussion of pedestrian bridges, Section H 250, also applies to bikeway bridges, in regards to walkway widths, approaches, railings, deck drainage, and vibration design criteria.

The general features, such as clearances to bridges are discussed in Section H 230, Bridges - General.

## H 261 GENERAL CLASSIFICATIONS

Bikeway bridges can be classified into combinations of the following four general categories, as shown in Figure H 260.

**Type I** - Bikeway bridge which exclusively provides a bicycle pathway.

**Type II** - Bikeway bridge which includes a separate walkway as an integral part of the bridge.

**Type III** - Bikeway bridge which provides a combined walkway and bicycle pathway.

**Type IV** - Bikeway bridge as an integral part of a vehicular bridge.

In general, for safety reasons, a combined pedestrian-bicycle pathway is not recommended. However, when dictated by economics, right-of-way, or other considerations, it may be selected. It is better to provide a physical separation, such as a railing between bicycle and pedestrian traffic. When separation is not possible, a 'Yield to Pedestrian' sign should be placed at each approach to the bridge.

## H 262 RAILINGS

A railing 5'-6" (1.68 m) minimum height, should be used along the outside edge of bikeway bridges and may be required as a physical separation between vehicular and bicycle or pedestrian and bicycle traffic. All railings should be designed in accordance with the loading criteria for pedestrian railing, except where a barrier railing to control vehicular traffic is required. The minimum horizontal clearance between railing and the street curb line should be 18" (457 mm).

The railing selected should be smooth without any sharp or protruding members.

## H 263 WIDTH OF BICYCLE PATHWAY

A bicycle pathway on a bridge should be as wide as the approaching bikeway. The recommended minimum clear distance between the railing is 6'-0" (1.83 m) for one-way bicycle traffic and 12'-0" (3.66 m) for two-way bicycle traffic. Separate sidewalk width should be in accordance with Subsection H 251.

When a combined pedestrian-bicycle pathway is contemplated, the required minimum width need not ordinarily be increased over that required for two-way bicycle traffic. However, all required , widths should be approved by the appropriate District Office.

## H 264 RECOMMENDED GRADE

Although in special situations steeper grades may be used, the recommended maximum grade is 5 percent for bikeway bridges.

## H 270 RAILROAD BRIDGES

The design features discussed in this section are: Bridge width, bridge deck, utility placement, girder bearings, railings, and expansion joints. For general features such as clearances to traveled way, see Section H 230, Bridges - General.

Design and construction should comply with the special requirements of~the following railroad companies, which, in most cases, will maintain the superstructure (the City will maintain the substructure, piers and abutments up to the girder seats).

**Atchison, Topeka and Santa Fe Railway Company:**

a. All steel in the superstructure of bridges carrying tracks (except steel in railings and minor hardware) should be "copper bearing" steel and be noted on the plans as such. "Copper bearing" steel contains not less than 0.2 percent of copper.

b. The protective covering to be provided over membrane waterproofing on the superstructures bridges carrying tracks should be asphalt plank.

c. The earth faces of abutments should be damp-proofed with either asphalt or coal-tar emulsions. The method of application and general requirements for these bituminous emulsions are specified in the AREA Specifications. A strip of of waterproofing should be placed at the junction of the stem and the footing.

d. Longitudinal decks or ballast troughs should be constructed on a slope of not less than 0.04" per foot (3.3 mm/m).

e. Walkway plates should be not less than 5/16" (7.9 mm) thick.

f. Alemite fittings should be used for all large bearings.

g. Access ladders should be provided for inspection of abutment bearings.

h. High strength bolts may be substituted for rivets in field connections.

i. Concrete and steel structures are designed for Cooper's E-8U loading with diesel impact.

j. All structures over tracks having less than 15 feet horizontal (4.57 m) clearance to centerline of tracks should have columns with a minimum area equivalent to a 4-foot (1.22 m) circular concrete column.

**Southern Pacific Transportation Company:**

a. Ballast plates on bridges carrying tracks should be low-alloy corrosion-resistant structural steel conforming to ASTM A-242.

b. Pins for bearings should be 3" (76 mm) in diameter or more.

c. The toe of slope of earth fill approaches to bridge structures over railroads is located so that the railroad can maintain its standard roadway ditch section, the outer limit of which is 15 feet (4.57 m) from the centerline of tracks (see Figure H 231.2B for FHWA maximum clearances).

d. Bridges of the through girder or through truss design should not be constructed except in unusual circumstances where deck type structures cannot be used.

e. Working area for coupling cars should be provided on all reconstructed and new bridges regardless of location. A ballast trough with a minimum clearance of 8'-0" (2.44 m) from centerline on tangent track and 9'-0" (2.74 m) on curved track will provide this working area and facilitate the removal of ties.

f. Clearance for through girder bridges from centerline of track to inside edge of widest flange plate should be at least 9'-0" (2.74 m) for tangent track, and 10'--0" (3.05 m) for curved track (Figure H 271).

g. Knee braces for through plate girder bridges should not be less than 2'-6" (0 .76 m) wide at the top of the floor beam.

h. The minimum thickness of concrete deck slabs should be 5-1/2" (139.7 mm) over tops of floor beams spaced approximately 1'-0" (0.61 m) center to center.

i. High strength bolts may be substituted for rivets in field connections.

j. All structures should be designed for Cooper's E-72 loading. Lighter design loading should be considered only in rare instances, such as special purpose spur tracks.

k. For prestressed concrete bridges, special layout of prestressing tendons is required. The layout which includes a minimum number of straight strands, can be obtained from SPTC.

**Union Pacific Railroad Company:**

a. All bridges to carry main line traffic should be designed for Cooper's E-80 Loading.

Requirements of Most Railroads:

a. The length of cover plates should be determined either by rigorous analysis or in accordance with the following procedure (see "Structural Design," by Sutherland and Bowman, page 136):

$$L' = (0.1 + 0.9\ a/A)L$$

Notations:

L = Length of span

a = Area of cover plate in question +cover plates outside

A = Total flange area

= cover plates + angles + web/8) at span centerline

L' = Length of cover plate

b. Ballast troughs of all railroad bridges should be sloped for drainage. If the tracks are level, the depth of ballast can be varied. This will increase the vertical dimension from top of rail to bottom of superstructure and should be considered in determining vertical clearances.

**Preferences of Most Railroads:**

a. Abutment stems should be 0.2H in thickness at the base.

b. Columns should be 0.2H in thickness at the base.

c. Floor beams should be 21" (533 mm) or more in depth.

Typical sections commonly used for railroad bridges are shown in Figure H 270.

## H 271 BRIDGE WIDTH

The width of a railroad bridge shall be determined by the number of tracks needed by the railroad company plus the required clearances. For clearances, see Figures H 231.2A, B and H 271.

## H 272 WALKWAYS

Walkways should be provided if requested by the railroad company. Walkways are constructed for the use of railroad company maintenance personnel, not the general public, and should be of a width specified by the company.

## H 273 CURBS

Curbs should be provided on bridge decks as requested by the railroad company. Curbs are used to retain ballast and should be of sufficient height to prevent spillage of ballast.

## H 274 BRIDGE DECK

**Drainage** - The bridgetdeck should be designed to drain properly. Longitudinal ballast troughs should be sloped a minimum of 1 percent for drainage. A greater slope, if practical, is desirable. Where deck drains or collectors are installed, suitable ballast guards an flashings are provided.

**Waterproofinq and Membrane Protection** - Generally, all railroad bridge decks are waterproofed according to the requirements of the **AREA Manual**, Chapter 29, Waterproofing. The waterproofing membrane is protected from the ballast by means of a covering. Specific requirements may vary with the railroad company involved. For example, the Southern Pacific Transportation Company requires waterproofing with a butyl-rubber membrane protected with asphalt cover panels.

A flexible joint detail should be provided when the waterpoofing membrane and the membrane protection covering span over any expansion joint in the deck.

## H 275 BALLAST, TIES, AND RAILS

Information about ballast, ties, and rails should be obtained from the railroad company involved, since these items vary with each railroad company. The information to be obtained should include the following:

a. Dimension of ties and rails.

b. Depth of ballast.

c. Extent of track work to be done by contractor.

d. Extent of track work to be done by railroad company

The information should encompass all track work including main, lateral, or shoofly tracks.

## H 276 UTILITY PLACEMENT

Generally, only those utilities necessary for the operation of the railroad are permitted on the bridge by the railroad company. Where utility placement is allowed by agreement between the railroad and utility companies, the designer should provide openings for the placement. Refer also to Subsection H 246, Utilities on Structures.

## H 277 GIRDER BEARINGS AND EXPANSION

Bearings are discussed in Subsection H 242, Girder Bearings. Specifications for expansion bearings are published in the AASHTO specifications. Abutment bearings should be accessible for inspection purposes.

## H 278 BRIDGE RAILING

Railjngs should be durable and of sufficient height to protect the trainman (a minimum of 3 feet (0.91 m) above the top of track elevation is recommended). The type of railing used depends on railroad company requirements and architectural considerations. For additional discussion of bridge railing, refer to Subsection H 234.

## H 279 EXPANSION JOINTS

A general discussion relating to expansion joints and joint fillers is in Subsection H 244. The sealed joint need not be used where membrane waterproofing is required.

## H 280 MISCELLANEOUS STRUCTURES

Miscellaneous structures discussed here include pumping plants, force mains, thrust blocks, special manholes, special catch basins, and sign structures.

## H 281 PUMPING PLANT STRUCTURES

A pumping plant structure houses pumps for raising the hydraulic head of a sewer or storm drain system. Pumping plants are usually concrete subterranean structures which vary in size from a single pump in a four-foot square concrete vault to multiple pumps housed in structures with dimensions of 30' x 60' (9.14 m x 18.29 m) or larger. Small plants may be prefabricated structures which house the pump, motors, valves, and electrical equipment. Larger plants usually perform more elaborate func tions in addition to pumping, such as storage or detention of sewage, screening of solids, silt removal, and controlling and monitoring flow.

In addition to the usual design dead load, live load, and lateral forces, the structure should be designed for vibration any buoyancy.

**Vibrations** - Design of foundations and other structures which support rotational machinery (such as centrifugal pumps and fans) should provide sufficient stiffness such that the natural frequency of the support is either 1.5 times higher or lower than the operating frequency of the machine. The August 1971 edition of the ACI Journal, pp. 568 and 569, provides additional information and references on this subject.

**Buoyancy** - Buoyancy should be analyzed as recommended by the soil report. The safety factor against buoyancy should be at least 1.25. Soil friction on walls should not be used to resist buoyancy.

## H 282 FORCE MAINS

A force main is a discharge pipe line which carries flow under pressure usually to a point of higher elevation and is used in both sewerage and drainage systems. Force main pipes may be constructed of cast iron or ductile iron, cement mortar lined and coated steel pipe (used in fittings for concrete cylinder pipe), or reinforced concrete.

Force mains are designed to resist earth load, live load, and surcharge load, in combination with internal pressure (refer to Subsection H 211, Circular Conduits).

## H 283 THRUST BLOCKS

Thrust blocks are required at pipe bends to resist the static and dynamic loads produced by moving liquid, and to transmit these loads into the trench walls or backfill. The entire load must be taken by thrust blocks because force main pipes normally have compression type joints which cannot resist axial tension. In gravity flow sewers and storm drains, the dynamic loads are generally much smaller than the static loads and can-usually be neglected.

The allowable design soil passive pressures should be as recommended by a soils report. Piles may be required when pipe is laid in soft soil.

# H 284 SPECIAL CATCH BASINS AND MANHOLES

Catch basins (CB) and manholes (MH) which do not conform to the conditions shown on the Standard Plans require special design and detailing. The following design approaches are suggested:

**Method 1, Minor Modifications** - Examine the Standard Plan to determine if the standard structure can be slightly modified to meet the special conditions. For example, when only minor modifications are needed, the Standard Plan may be referred to and only the modifications detailed or noted on the plans. Refer to the detail as "Modified CB..." or "Modified MH...." on the drawings.

**Method 2, Extensive Modifications** - Where the modifications required are extensive, and where Method 1 cannot be applied effectively, design and detail the entire structure using appropriate loadings and code requirements.

## H 284.1 SPECIAL CATCH BASINS

Modifications of standard catch basins generally consist of an increase in width of opening, a change in top slab or wall thicknesses, or an increased loading on the catch basin. A different direction of flow, or an outlet arrangement contrary to the standard may also require modifications.

**Openings** - An increase in the width of opening at the curb face requires a special examination of the reinforcement around the opening and in the top slab to insure structural adequacy. An increase in the number of support bolts and in the length of the protection bar can usually be covered by an added note.

**Top Slab** - The top slab of standard CB has been designed for a minimum thickness of 4-1/2 inches (115 mm) with reinforcement for a minimum design live load per Subsection H 333.4. Any increase in loading should be reflected in an appropriate increase in the top slab reinforcment. If thickness must be increased, modifications of the steel plate and the support bolts should be considered.

## H 284.2 SPECIAL MANHOLES

Standard manholes and manhole shafts are designed for specific depths. For instance, the shaft thickness of brick manholes is controlled by maximum shaft design depths of 22' (6.77 m) and 35' (10.67 m). Increased depths require special~ design to insure structural adequacy. Increased shaft thickness is usually required for an increased design depth. Design may also be necessary when the conduit or structure served by the manhole is not standard. An increase in the invert slab thickness, shaft support thickness, or in the reinforcing may be neccessary. For analysis of a manhole shaft, it is recommended that the earth pressure be distributed according to Figure H 257.1. For the value of P, see Subsection H 373.2, Lateral Earth Pressures, Arching Effect.

The ring should be analyzed using Office Standard No. A-3, "Moment and Thrust Coefficients for Underground Pipes," in the Appendix.

The structure supporting the manhole shaft should be adequately reinforced around the opening to resist loads transmitted by earth backfill, the shaft, and all live loads.

## H 285 SIGN STRUCTURES

Overhead sign structures are designed in accordance with AASHTO Specification for the Design and Construction of Structural Supports for Highway Signs (latest edition). Loading is as specified in Section H 380, Other Loads and Forces.

The minimum vertical clearance over traveled roadway should be 18 feet (5.49 m).

## REFERENCES

l. M. G. Spangler, "The Supporting Strength of Rigid Pipe Culverts," Bulletin No. 112, Iowa Engineering Experiment Station, Ames, Iowa, 1933, p. 53.

2. M. G. Spangler, "The Structural Design of Flexible Pipe Culverts," Iowa Engineering Experiment Station, Bulletin No. 153, 1941.