Direct Design and Indirect Design of Concrete Pipe
Part 1

Josh Beakley
March 2011
Why are We Here?

- AASHTO LRFD Bridge Design Specifications
  - Metal Pipe – Section 12.7
  - Concrete Pipe – Section 12.10
  - Plastic Pipe – Section 12.12
AASHTO Section 12.10.1

General

“The structural design of the types of pipes indicated above may proceed by either of two methods:

– The direct design method at the strength limit state as specified in Article 12.10.4.2, or

– The indirect design method at the service limit state as specified in Article 12.10.4.3.”
# Indirect Design Method

The following Fill Height Tables have been developed by the American Concrete Pipe Association (ACPA) using the Indirect design method in accordance with Section 12.10.4.3 of the AASHTO-HIP Design Specification, 4th Edition, 2007 with 2009 interim. Live load was distributed through the pipe in accordance with Chapter 4 of the ACPA Concrete Pipe Design Manual.

Fill Height Tables are based on:
1. 90 ft 100 sf
2. AASHTO HL-93 live load
3. Passive Proprietary Embankment Condition - this gives conservative results in comparison to trench conditions.
4. A Type 1 installation requires greater soil stiffness than the surrounding soils than the Type 2, 3, and 4 installations, and is thus harder to achieve.

Thereby, field verification of soil properties and compaction levels should be performed.

### D-Load (lb/ft²) for Type 1 Bedding

<table>
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<tr>
<th>PIPE I.D. (Inches)</th>
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</table>
For Special Cases, use the Direct Design Method

Reinforcement for flexural resistance provided in a length, \( b \), usually taken as 12.0 in. shall satisfy:

\[
A_y \geq \frac{g \phi d - N_u - \sqrt{g \left[ g(\phi d)^2 - N_u (2\phi d - h) - 2M_u \right]}}{f_y}
\]

(12.10.4.2.4a-1)
### Fill Heights – Type 2 Installation

<table>
<thead>
<tr>
<th>Pipe Size (in)</th>
<th>Class III</th>
<th>Class IV</th>
<th>Class V</th>
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<td>D</td>
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<td>24</td>
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</tbody>
</table>

I = Indirect Design in Accordance with AASHTO Section 12  
D = Direct Design in Accordance with AASHTO Section 12
Benefits of the Indirect Design Method

- Its validity has been proven over time
- It is a simple method to use
- It is a proof-of-performance method
Benefits of the Current Direct Design Method

- It is simple (relatively speaking)
- It is safe
- It is proven
Intention for Direct Design

- Used for higher strength pipe that can not be found in ASTM C 76 Tables
- Used for larger diameter pipes
- Used for specific loads and load cases
- Used when stirrup reinforcement is required
Some Factors for Difference

- Reinforcement Proportions
- Size Factor
- Steel Reinforcement Properties
- Double Reinforcement
Conservative Designs

- The simplification of the direct design process for concrete pipe results in conservative designs.
- The designs are most conservative for smaller diameter pipe.
Reinforcement Proportions
Reinforcement Proportions

$A_{so} = 0.6 A_{si}$

Previously - $0.75 A_{si}$
Correction for Larger Springline Reinforcing

\[ c = \text{correction factor} = 0.57 \left(1 + \frac{f_s' A_s d_2}{f_{s1} A_{s1} d_1}\right) \]

“A Theory for Structural Behavior of Reinforced Concrete Pipe”, Frank Heger, 1962
\[0.60 \text{ Asi} \leq 36'' \leq 36'' \leq 66'' > 66''\]
Size Factor
Mc = 0.318  Mc = 0.249  Mc = 0.324  Mc = 0.254
Three-Edge Bearing Test

\[ M = 0.318 \, P \, r \]
Experimental vs FEM Results

Pipe# 18SP

(72.73 psi)

(75 psi)
Location of First and Second Crack

1st Crack

2nd Crack

Small Diameter

Large Diameter
Mc = 0.318  
Mc = 0.249  
Mc = 0.324  
Mc = 0.254
First Crack vs. Second Crack

D-load Required for Second Crack Versus Pipe Diameter

Pipe Inside Diameter (inches)
Three Edge Bearing Test Load

(c) First Stage Cracking

\[ I_{1e} = 0.3 I_0 \]

\[ I_0 \times 0.5 \times 0.5 \]

\[ I_1 = I_3 = I_{1e} \]

Loading Diagrams

(d) Second Stage Cracking

\[ I_4 = 0.31 I_2 \times 0.31 I_2 \]

\[ I_1 = I_2 = I_3 \times I_4 \]

\[ \frac{P_1}{2} \times \frac{P_2}{2} \]

(e) Ultimate Condition - Flexural Failure

\[ P_0 \]

\[ \frac{P_1}{2} \]

\[ \frac{P_2}{2} \]

Moment (Coefficient x Pr)

\[ M = P_0 (249 - 50 \sin \theta) \]

\[ M = P_0 (324 - 50 \sin \theta) \]

\[ M = P_0 (272 - 50 \sin \theta - 0.18 \cos \theta) \]
17.3 The wall thickness of nonreinforced concrete pipe shall not be less than \( h \), where

\[
h = \frac{-N_s + \sqrt{N_s^2 + 288f_{mr}M_u}}{24f_{mr}} \quad (17-1)
\]

\[
f_{mr} = k_{mr} f_{mr} \sqrt{f'_c} \quad (17-2)
\]

where \( k_{mr} \) = see Table 17.3-1

**TABLE 17.3-1.** \( k_{mr} \) Values for Nonreinforced Pipe

<table>
<thead>
<tr>
<th>Pipe Diameter [in. (mm)]</th>
<th>( k_{mr} )</th>
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<tbody>
<tr>
<td>12 (300)</td>
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<td>18 (450)</td>
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<td>24 (600)</td>
<td>10.0</td>
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<td>30 (750)</td>
<td>9.0</td>
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<tr>
<td>36 (900)</td>
<td>7.5</td>
</tr>
</tbody>
</table>

\( k_{mr} \) = SIZE FACTOR
“However, the author has also made experimental calculations based on the theory of plasticity and found reasonably good correspondence with the test results although it seems unlikely that an unreinforced concrete pipe might be calculated in accordance with a theory of this kind.”

“After the occurrence of the first crack the pipe will not collapse, however, because a hinge is created at the crown.”

“Calculation of Unreinforced Concrete Pipes Based on a New Theory for the Rupture”, John B. Ingwersen, Danish Concrete Industry, Association
We calculate the moment at a specific location without any consideration of moment distribution as a result of the pipe size or reinforcement proportions.

\[ M_i = C_{mi} \frac{W_i D_m}{2} \]  
\[ N_i = C_{ni} W_i \]  
\[ V_i = C_{vi} W_i \]  

The non-dimensional coefficients, \( C_{mi} \), \( C_{ni} \), and \( C_{vi} \) for determining the stress resultants \( M_i \), \( N_i \), and \( V_i \), respectively, at governing locations at the crown, invert, springline and at the critical locations for shear in the invert and crown regions are given in Table 8.4 for the above types of applied load. Two coefficients are given for live load. The coefficients with an \( L_1 \) sub-

<table>
<thead>
<tr>
<th>Location</th>
<th>Load Type</th>
<th>Coefficients</th>
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<td>( W_{L2} )</td>
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<td>Critical Shear</td>
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<td>Invert ( \theta = 12^\circ )</td>
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Section 8.3 and Chapter 5. The term, \( W_i \), in each of these equations represents the following loads on the pipe: Pipe Weight, \( W_p \); Earth Load, \( W_e \); Fluid Load, \( W_f \); and Live Load, \( W_{L1} \).
0.5 * P * r_m = M_{sum}

0.5*(DL_u * D_i/12) * D_m/2 = M_{sum}

DL_u = [48/(D_i * D_m)] * M_{sum}

“Experimental Evaluation of SIDD Design Procedures for Shear, Radial Tension and Crack Width Control with Emphasis on Small Diameter Concrete Pipe”, SG&H, 1993
Steel Properties
Stress-Strain Curves

![Stress-Strain Curves Diagram](image)

- Welded wire fabric
- Grade 75
- Grade 60
- Grade 40
Actual Stress-Strain Curves

**Stress vs. Strain**

- **Actual Stress**
- **Strain Curves**

**Stress (psi)**
- $\sigma_1$
- $\sigma_2$
- $\sigma_3$

**Strain**
- $\varepsilon_1$
- $\varepsilon_2$
- $\varepsilon_3$

- $9.643 \times 10^4$
- $8.75 \times 10^4$
- $7.5 \times 10^4$
- $7.5000$
- $6.25 \times 10^4$
- $5 \times 10^4$
- $3.75 \times 10^4$
- $2.5 \times 10^4$
- $1.25 \times 10^4$
- $0$

- $0.01$
- $0.008$
- $0.006$
- $0.004$
- $0.002$

- $0$

- $0$
Power Formula (Smooth Wire)

\[ f_s = E_s \varepsilon_s \left[ Q + \frac{1 - Q}{1 + ((E_s / Kf_{py})^R)^R} \right] \leq f_{pu} \]

<table>
<thead>
<tr>
<th>fpu</th>
<th>fpy</th>
<th>Q</th>
<th>Es</th>
<th>R</th>
<th>K</th>
</tr>
</thead>
<tbody>
<tr>
<td>80</td>
<td>65</td>
<td>0</td>
<td>28310.54</td>
<td>2.2680</td>
<td>1.23847</td>
</tr>
</tbody>
</table>
Compression/Tension in Bending

Stresses

\[ 0.85f_c b \alpha \]

\[ 0.85f_c b \alpha \]

N.A.

\[ A_{fs} \]

\[ A_{fs} \]

Strains

\[ \varepsilon_c = 0.003 \]

\[ \varepsilon_s = \varepsilon_y \rightarrow f_s = f_y \]

N.A.
Neutral Axis Iteration

- $C = 0.5''$
  - $\varepsilon_c = 0.003$
  - $\varepsilon_s = 0.007$
- $C = 0.6''$
  - $\varepsilon_c = 0.003$
  - $\varepsilon_s = 0.006$
- $C = 0.7''$
  - $\varepsilon_c = 0.003$
  - $\varepsilon_s = 0.005$

Mild Steel
- $T = A_s f_y$
- $T = A_s f_y$
- $T = A_s f_y$

Wire
- $T = A_s f_s$
- $T = A_s f_s$
- $T = A_s f_s$
Rebar versus Wire

Stress-Strain Curves

- \(\sigma_{l_1}\)
- \(\varepsilon_{l_1}\)
- \(\varepsilon_{l_0}\)

\(-10^5, -10^4, -10^3\)
We Force Ourselves to be
Beyond the Yield Point

g' := b \cdot fc \cdot [0.85 - 0.05 \cdot (fc - 4.0)]

\[
A_{\text{semax}} := \frac{55 \cdot g' \cdot \phi \cdot d}{f_y \cdot (87 + f_y)} \cdot \frac{0.75 \cdot Nu}{f_y}
\]

equation 12.10.4.2.4c-2
“When tested, the yield strength shall be determined at an extension under load of 0.0035 mm/mm (0.0035 in./in.)”

“The material shall not exhibit a definite yield point as evidenced by a distinct drop of the beam or halt in the gage of the testing machine prior to reaching ultimate tensile load.”

ASTM A82. “Standard Specification For Steel Wire, Plain, for Concrete Reinforcement”
Crack Control

Work with Stresses Below Yield

Stress-Strain Curves

\[ \sigma_i = 9.376 \times 10^4 \times 6 \times 10^4 \]

\[ \varepsilon_i = 0.002, 0.004, 0.006, 0.008 \]

\[ \sigma, \varepsilon \]

\[ 9.6 \times 10^{-3} \]

\[ \sigma_i, \varepsilon_i \]
When Does Crack Control Kick In?
Power Formula (Smooth Wire)

\[ f_s = E_s \epsilon_s \left[ Q + \frac{1-Q}{1 + ((E_s \epsilon_s / K f_{py})^k)^R} \right] \leq f_{pu} \]

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Double Reinforcement
LRFD 5.7.3.2 Flexural Resistance

\[ M_n = A_{ps} f_{ps} \left( d_p - \frac{a}{2} \right) + A_s f_s \left( d_s - \frac{a}{2} \right) - \]

\[ A' f' \left( d_s' - \frac{a}{2} \right) + 0.85 f_c' (b - b_w) h_f \left( \frac{a}{2} - \frac{h_f}{2} \right) \]

(5.7.3.2.2-1)
Reinforcement for flexural resistance provided in a length, $b$, usually taken as 12.0 in. shall satisfy:

$$A_s \geq \frac{g\phi d - N_u - \sqrt{g\left[g(\phi d)^2 - N_u(2\phi d - h) - 2M_u\right]}}{f_y}$$

(12.10.4.2.4a-1)
$0.85 f_c^* b^* a$

$a = \beta_1 * C$

$A_s f_y$

$N_u$

$M_u$
A Weakness of Equation 12.10.4.2.4a-1 – One Layer of Steel

Long-Term Monitoring Results for Strain in Reinforcement of Segment C2
(average fill height 18.75 ft)

Max strain in steel is 13% of yield

UNO – Behavior and Design of Buried Concrete Pipe – 48 inch pipe
Strain Distribution

Figure 33 Strain distribution at the crown at 0.01 in crack load
Two Cages

Small Diameter Pipe

Large Diameter Pipe

$\varepsilon_{cu} = 0.003$

$\varepsilon_{Oi}$

$\varepsilon_{oi}$
36 inch Class III Pipe

Second Cage

Neutral Axis $C = 0.56$ in

Flexure

Stress-Strain Curves

$\sigma_{yi} = 9.6 \times 10^3$
Second Cage

Neutral Axis C = 1.15 in

Crack Control
To Be Continued.....