Preface

This lab handout is intended to give an overview of a Multi storied Building and a Balanced Cantilever Bridge structural analysis and design. It concentrates on the gravity loading only. This handout provides a basic guideline for analysis, design and detailing works as well as reviewing a standard code of practice. To provide the undergraduate students a well-organized, user-friendly, and easy-to-follow resource, this handout is divided into two major parts. The first part mainly focuses on the structural analysis and design of Reinforced concrete (RC) Multistoried Building that includes design of Slab, Beam, Column, Stair, Water reservoir and Lateral load analysis. The other part deals with the Balanced Cantilever Bridge including an introduction to Bridge Engineering, details about Balanced Cantilever Bridge, design of Deck Slab, design of Railing, Post and Curb/Sidewalk, design of Interior Girder considering dead and live loads only, design of Exterior Girder considering dead and live loads only, design of Diaphragm or Cross Girder and Design of Articulation. Handouts of Dr. Khan Mahmud Amanat, and Mr. Ruhul Amin, of BUET were helpful as well as suggestions from some faculty members of the Department of Civil Engineering, AUST.

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### Part 1

**Structural Analysis and Design of the Multistoried RC Building**

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# Part 2

Preliminary Design of the Superstructure of a Balanced Cantilever Bridge for Gravity loading

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Part 1: Structural Analysis and Design of the Multistoried RC Building

1.1 Introduction

Generally, the design of any structure (building, bridge etc.) can be dividing in two segments,

- Foundation design (footing, basement, retaining wall, abutment, underground water reservoir etc.)
- Design of superstructure (beam, column, slab, girder, stair etc.)

Figure 1: Super structural elements
Figure 2: Foundation elements
Figure 3: Gravity load distribution
Steps of design

- Specify the type of structural system like RCC or Steel or Composite, beam supported or flat plate or braced etc.
- Specify the loads based on the type of services, like residential or commercial or institutional etc. from codes and judgments.
- Prepare a preliminary model of the structure based on preliminary calculations and judgment.
- Analyze the model for desired load combinations according to BNBC in the context of Bangladesh,
  
  I. DL+LL
  
  II. 1.4DL+1.7LL
  
  III. 0.75[1.4DL+1.7LL±1.7{1.1(EQ_x or EQ_y)}] ~ 1.05DL+1.275LL±1.4(EQ_x or EQ_y)
  
  IV. 0.75{1.4DL+1.7LL±1.7(W_x or W_y)} ~ 1.05DL+1.275LL±1.275(W_x or W_y)

- Design the structural elements separately by considering their integrity and construction feasibility of that design.
1.2 Notations

**U.S.D Method**

- $f'_c = \text{Cylindrical strength of concrete}$
- $f_y = \text{Yield strength of reinforcement}$
- $V_c = \text{Allowable shear force without web reinforcement} = 2 \lambda \sqrt{f'_c b_w d}$
- $V = \text{Allowable shear force with web reinforcement} = 8 \lambda \sqrt{f'_c b_w d}$
- $V = \text{Allowable peripherial shear force in slab and footing without web reinforcement} = 4 \lambda \sqrt{f'_c b_w d}$

**Strength reduction factors:**
- # Flexure, without axial load = 0.90
- # Axial compression and axial compression with flexure:
  - Members with spiral Reinforcement = 0.75
  - Other reinforcement = 0.65
- # Shear and torsion = 0.75
- # Bearing on concrete = 0.75

**W.S.D Method**

- $f'_c = \text{Cylindrical strength of concrete}$
- $f_c = 0.45 f'_c$
- $f_y = \text{Yield strength of reinforcement}$
- $E_c = 33 \times w^{1.5} \times \sqrt{f'_c}$
- $n \frac{E_s}{E_c} = \frac{29 \times 10^6}{145 \times 33 \sqrt{f'_c}}$
- $k = \frac{n}{(n+r)}$
- $j = 1 - k/3$
- $R = \frac{1}{2} f_c \times k \times j$
- $\nu_c = \text{Allowable shear stress without web reinforcement} = 1.1 \sqrt{f'_c}$
- $\nu = \text{Allowable shear stress without web reinforcement} = 5 \sqrt{f'_c}$
- $V_c = \text{Allowable peripherial shear stress in slab and footing without web reinforcement} = 2 \sqrt{f'_c}$

**Table 1:** Moment and shear values using ACI coefficients. (Ref: ACI Code, Design of Concrete)
Structure, 15th edition, Chap-11, P-363)

Figure 4: Moment coefficients for beam.

Numbers on beams refer to moment value coefficient, \( x \), in the equation:

\[
M_u = \frac{w_{u} L_{n}^2}{x}
\]

The number, \( n \), refers to the number of spaces into which the girder span is subdivided by beams.
### 1.3 Design of Stair

![Typical stair](image)

**Figure 5:** Typical stair

#### a) Assumptions and considerations

- $f_y = 60000$ psi
- $f'_{c} = 3000$ psi

Thickness of waist and landing slab = 6"

Live Load = 82 psf = 0.082 ksf (BNBC)

Floor Finish = 25 psf = 0.025 ksf

#### b) Load calculation

**Rises & Steps**

$$\text{Rises & Steps} = \left( \frac{1}{\frac{1}{2} + \frac{6}{12} + \frac{10}{12} + 3.5 + 9 + 150}{1000} \right) = 0.98 \text{ k}$$

**Waist slab**

$$\text{Waist slab} = \left( \frac{\sqrt{7.5^2 + 4.5^2 + \frac{6}{12} + 3.5 + 150}}{1000} \right) = \left( \frac{8.75 + \frac{6}{12} + 3.5 + 150}{1000} \right) = 2.3 \text{ k}$$
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Total Dead Load= Landing slab + (Rises & Steps + Waist)

\[ = \left( \frac{6+150}{1000} \right) + \left( \frac{0.98+2.3}{3.5+7.5} \right) / 2 = 0.1 \text{ ksf} \]

Total load, \( W = (0.082 \times 1.7) + [1.4 \times (0.1 + 0.025)] = 0.31 \text{ ksf} \)

c) Moment and reinforcement calculation

\[
M^+ = \frac{WL^2}{14} = \frac{0.31 \times (2 \times 3.5 + 7.5)^2}{14} = 4.7 \text{ k-ft/ft}
\]

\[
M^- = \frac{WL^2}{9} = \frac{0.31 \times 14.5^2}{9} = 7.24 \text{ k-ft/ft}
\]

\[d = (t-1) = (6-1) = 5''\]

\[
\rho_{0.005} = 0.85 \times \beta_1 \times \frac{f_{tc}}{f_y} \times \frac{0.003}{0.003 + \varepsilon_t} = 0.85 \times 0.85 \times \frac{3000}{60000} \times \frac{0.003}{0.003+0.005} = 0.0135
\]

\[M_u = \phi \times \rho_{0.005} \times f_y \times b \times d^2 \times \left( 1 - 0.59 \times \frac{\rho_{0.005} \times f_y}{f_{tc}} \right) \]

\[d^2 = \frac{7.24 \times 12}{0.9 \times 0.0135 + 60 + 12 \times \left( 1 - 0.59 \times \frac{0.0135 + 60}{3} \right)} = \frac{86.9}{8} = 11.28 \text{ in}^2\]

\[d = 3.36'' < \text{provided, 5'' (ok)}\]

**Table 2:** Minimum ratios of temperature and shrinkage reinforcement in slabs based on gross concrete area. (Ref: ACI Code, Design of Concrete Structure, 15th edition, Chap-12, P-385)

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

\[A_{s_{\min}} = 0.0018 \times b \times t = 0.0018 \times 12 \times 6 = 0.129 \text{in.}^2\]

\[+A_s = \frac{M+12}{\phi \times f_y \times (d-2)} = \frac{4.7+12}{0.9 \times 60 \times \left( 5 \times \frac{0.5}{2} \right)} = 0.23 \text{in.}^2 / \text{ft (controlled)}\]

\[a = \frac{A_s \times f_y}{0.85 \times f_{tc} \times b} = \frac{0.23 \times 60}{0.85 \times 3 \times 12} = 0.48 \text{ (ok)}\]

Now, \( \frac{0.11 \times 12}{0.23} = 5.74'' \); use \( \Omega 10 \text{mm} @ 5.5'' \) c/c alt ckd
Again,

\[-A_s = \frac{M_{12}}{\phi f_y (d-\frac{a}{2})} = \frac{7.24+12}{0.9+60\left(5-\frac{0.7}{2}\right)} = 0.34\text{in.}^2/\text{ft (controlled)}\]

\[a = \frac{A_s f_y}{0.85 f'_c + b} = \frac{0.34+60}{0.85+3+12} = 0.68\text{" (ok)}\]

The distance between two cranked rod is 11".

So, Required reinforcement = 0.34 - \frac{0.11+12}{11} = 0.22\text{in.}^2/\text{ft}

The extra negative reinforcement required, \frac{11}{\left(\frac{0.11+12}{0.22}\right)} = \frac{11}{6} = 1.83 \text{ So, use } 2-\phi 10\text{mm as extra top.}

For shrinkage, \(A_{s\min} = 0.0018 \times 12 \times 6 = 0.129\text{in.}^2\)

Now, \frac{0.11+12}{0.129} = 10.23\text{"}; use\phi 10\text{mm@10" c/c}

**d) Stair Beam**

Assume beam size, 10"x12"

\[d = (t-2.5) = (12-2.5) = 9.5\text{"}\]

So, self-weight = (0.83*1*150)/1000 = 0.12k/ft

Load on Stair beam = \frac{0.31 \times 14.5 \times 3.5}{7.5} + (0.42 \times 9 \times 0.12 + 0.12) \times 1.4 = 2.9 \text{ k/ft}

The stair beam will be designed as described in floor beam design segment.
Figure 6: Reinforcement details of stair
1.4 Design of OWR

**Figure 7:** Roof top water reservoir (Overhead water reservoir)

**a) Assumptions and considerations**

\[ f'c = 3000 \text{ psi} \]
\[ f_y = 60000 \text{ psi} \]

6th floor building of 2 units & 5 members in each unit.
Water consuming 210 per capita per day (BNBC 1993)

**b) Water reservoir size calculation**

Total members= 6*2*5= 60 persons.
Total water consuming= 60*210 = 12600 litters for a full day.
\[ \frac{12600}{1000} \text{ m}^3 = 12.6 \times 3.28^3 = 445 \text{ ft}^3 \]

Inner length & width of Reservoir are,
Length =14.5 ft and width = 7.5 ft (From plan)
so, Height $= \frac{445}{7.5 \times 14.5} = 4.09 \text{ ft} + 1 \text{ ft} = 5.09 \text{ ft} \sim 6 \text{ ft}$; [where, free Board= 1 ft]
Height $= 6 \text{ ft}$

![Diagram of pressure distribution on reservoir wall]

**Figure 8**: Pressure distribution on reservoir wall

c) **Vertical Reinforcement of wall**

Let wall thickness $= 5''$
so, Effective depth, $d = 5 - 1 = 4''$

$$
\rho_{0.005} = 0.85 \times \beta_1 \times \frac{f_{rc}}{f_y} \times \frac{0.003}{0.03 + v} = 0.85 \times 0.85 \times \frac{3000}{60000} \times \frac{0.003}{0.03 + 0.005} = 0.0135
$$

$$
M_u = \phi \times \rho_{0.005} \times f_y \times b \times d^2 \times \left(1 - 0.59 \times \frac{\rho_{0.005} \times f_y}{f'_{c}}\right)
$$

$$
d^2 = \frac{2.25 \times 12}{0.9 \times 0.0135 \times 60 \times 12 \times \left(1 - 0.59 \times \frac{0.003}{0.03 + 0.005} \times \frac{3000}{60000}ight)} = \frac{27}{7.59} = 3.56 \text{ in}^2
$$

$d = 1.92'' < \text{provided}, 4'' \text{ (ok)}$

$$
A_s_{min} = 0.0018 \times b \times t = 0.0018 \times 12 \times 5 = 0.12 \text{ in}^2/\text{ft}
$$

$$
A_s = \frac{M \times 12}{\phi \times f_y \times \left(d - \frac{a}{2}\right)} = \frac{2.25 \times 12}{0.9 \times 60 \times \left(4 - \frac{0.25}{2}\right)} = 0.13 \text{in}^2 / \text{ft (controlled)}
$$

$$
a = \frac{A_s \times f_y}{0.85 + f'_{c} \times b} = \frac{0.13 \times 60}{0.85 \times 3 + 12} = 0.26 \text{ (ok)}
$$

CE412: Structural Analysis & Design Sessional - II
Now, $\frac{0.11 \times 12}{0.13} = 10.15"$

use, φ10mm@10" c/c.

d) Horizontal reinforcement of wall

Force = $\gamma \times h \times \left( \frac{14.5}{2} + \frac{14.5}{2} \right) = 62.5 \times 6 \times \left( \frac{14.5}{2} + \frac{14.5}{2} \right) = 5438$ lb

Again,

$$\frac{\text{Force}}{\text{stress}} = \frac{5438}{f_y} = \frac{5438}{60000} = 0.09 \text{ in}^2/\text{ft}$$

As$_{\text{min}}$ controls.

Now, $\frac{0.11 \times 12}{0.12} = 11"$

Use φ10 @ 11" c/c

e) Design of bottom slab

Table 3: Minimum thickness of nonprestressed one-way slabs. (Ref: ACI Code, Design of Concrete Structure, 15th edition, Chap-12, P-384)

<table>
<thead>
<tr>
<th></th>
<th>$l$</th>
<th>20</th>
</tr>
</thead>
<tbody>
<tr>
<td>Simply supported</td>
<td>$l$</td>
<td>20</td>
</tr>
<tr>
<td>One end continuous</td>
<td>$l$</td>
<td>24</td>
</tr>
<tr>
<td>Both ends continuous</td>
<td>$l$</td>
<td>28</td>
</tr>
<tr>
<td>Cantilever</td>
<td>$l$</td>
<td>10</td>
</tr>
</tbody>
</table>

Thickness $= \frac{7.5}{20} \times 12 = 4.5$ in

Self-weight of slab $= (4.5/12) \times 150 = 56.25$ psf

$$\frac{5w_A l_A^4}{384 EI} = \frac{5w_B l_B^4}{384 EI}$$

$$w_A l_A^4 = w_B l_B^4$$

$$w_A = w_B \left( \frac{l_B}{l_A} \right)^4$$

$$w_A = 15.63 \times w_B$$

$$w_A + w_B = 56.25 \text{ psf}$$
$w_B = 3.38$ psf

$w_A = 52.87$ psf

Floor Finish = 25 psf = 0.025 ksf

As the slab is one-way slab, design only for short direction

Total load, $w = (0.0625*6*1.7) + [1.4*(0.05287 +0.025)] = 0.75$ ksf

**Moment for short direction**

$M^+ = \frac{wL^2}{14} = \frac{0.75*7.5^2}{14} = 3$ k-ft/ft

$M^- = \frac{wL^2}{24} = \frac{0.75*7.5^2}{24} = 1.75$ k-ft/ft

$M_u = \phi * \rho_{0.005} * f_y * b * d^2 * \left(1 - 0.59 * \frac{\rho_{0.005} * f_y}{f'_c}\right)$

$d^2 = \frac{3+12}{0.9+0.015+60+12\left(1-0.59\times\frac{0.015+60}{3}\right)} = 4.5$

$d = 2.12" < $ provided, 3.5" (ok)

$A_{s_{min}} = 0.0018*b*t = 0.0018*12\times4.5 = 0.1$ in$^2$/ft

$A_s = \frac{M\times12}{\phi*fy*(d-A)} = \frac{3+12}{0.9+60\left(3.5-\frac{0.4}{2}\right)} = 0.2$ in$^2$

$a = \frac{A_s*fy}{b*0.85+f'_c+b} = \frac{0.2+60}{0.85+3+12} = 0.39$ (ok)

Now, $\frac{0.11+12}{0.2} = 6.6"$;

Use $\phi_{10mm} @ 6.5"$ c/c alt. ckd and 1-$\phi_{10mm}$ as extra top.
f) Top slab

For top slab there is no water load and some live load which is negligible. As the bottom slab is controlled by 4.5" thickness, top slab will be governed by a thickness of 4.5” and $A_{S_{\text{min}}}$.

Figure 9: Reinforcement details of top slab overhead water reservoir
Figure 10: Reinforcement details of roof top water reservoir (elevation view)
g) **Load on beam**

Here, Load from Bottom Slab = 0.75 ksf

Beam Thickness, \( t = 12 \) in

Effective Depth, \( d = (12-2.5) = 9.5 \) in

Self-weight = \( 0.83*1*150 = 0.12 \) k/ft

![Figure 11: Load distribution of slab](image)

Trapezoidal portion,

\[
\frac{1}{2} \times (14.5 + 7) \times 3.75 \times (0.75 + 0.056 \times 1.4)
\]

\[
= \frac{14.5}{14.5} + 0.12 \times 1.4 + (0.42 \times 6 \times 0.15) \times 1.4 = 3 \text{ k/ft}
\]

The beam will be designed as discussed in floor beam design segment.
1.5 Lateral Loads Calculation of Residential Building

a) Earthquake Load Calculation:

From BNBC (2006)

Seismic Zone-coefficient, \( Z = 0.15 \) [Dhaka]

Structural Importance Coefficient, \( I = 1 \)

Response Modification Coefficient, \( R = 8 \)

Now,

Numerical Co-efficient, \( C = \frac{1.25*S}{T^2} = \frac{1.25*1.5}{(0.61)^2} = 2.6 < 2.75 \)

\( s_2 = 1.5 \)

Assume,

Height of structure from base = 56 ft

Dead load on each floor = 175 kip

\( T = C_t* (h_n)^{3/4} = 0.073*\left(\frac{56}{3.28}\right)^{3/4} = 0.61<0.7; \)

\( W = DL* \text{Area}* \text{Storyed} = 175*47.875*40.575*5*\left(\frac{1}{1000}\right) = 1699.7 \text{ k~1700 k} \)

\( V = \frac{Z*I*C*W}{R} = \frac{0.15*1*2.6*1700}{8} = 82.9 \text{ kip} \)

\( W_i = DL* \text{Area}*\left(\frac{1}{1000}\right) = 175*47.875*40.575*\left(\frac{1}{1000}\right) = 339.9 \text{ k ~ 340 k} \)

\( \Sigma W_i * h_i = 340*(6+16+26+36+46+56) = 63240 \)

Here, \( F_t = 0 \) \( \text{as}, T<0.7 \)

Load on each floor, \( F_x = \frac{(V-F_t)*W_i*h_i}{\Sigma W_i*h_i} \)

\( F_x = \frac{(82.9-0)*340*h_x}{53040} = 0.446 * h_x \)
Table 4: Equivalent earthquake forces at different levels.

<table>
<thead>
<tr>
<th>Floor</th>
<th>( h_x )</th>
<th>( \text{Force, } F_x = 0.53 \times h_x )</th>
</tr>
</thead>
<tbody>
<tr>
<td>Grade beam</td>
<td>6 ft</td>
<td>2.676</td>
</tr>
<tr>
<td>Ground floor</td>
<td>16 ft</td>
<td>7.136</td>
</tr>
<tr>
<td>1st</td>
<td>26 ft</td>
<td>11.596</td>
</tr>
<tr>
<td>2nd</td>
<td>36 ft</td>
<td>16.056</td>
</tr>
<tr>
<td>3rd</td>
<td>46 ft</td>
<td>20.516</td>
</tr>
<tr>
<td>4th</td>
<td>56 ft</td>
<td>24.976</td>
</tr>
<tr>
<td><strong>Total</strong></td>
<td></td>
<td><strong>82.956</strong></td>
</tr>
</tbody>
</table>

b) Wind Load Calculation

Here,
Gust Co-efficient, \( C_G = 1.43 \)

\[ C_c = 47.2 \times 10^{-6} \]

\( B = 47.875 \text{ ft} \)
\( L = 40.575 \text{ ft} \)
\( \text{Height}, h = 50 \text{ ft} \)

Now,
Important Co-efficient, \( C_I = 1.00 \)
Combined height & exposure Co-efficient, \( C_Z = \) Table 6.2.10 (BNBC 2006)
Wind Velocity = 210 \text{ km/hr} (Dhaka)

\[ q_z = C_C \times C_I \times C_z \times V_b^2 = 2.08 \times C_z \]

\[ C_z = 0.1879 \times z^{0.4435} \geq 0.368 \text{ (z = ht. above ground in meter)} \]

Table 5: Overall pressure coefficients, \( C_p \) for rectangular building with flat roof. (Ref: BNBC 2006)

<table>
<thead>
<tr>
<th>( h/B )</th>
<th>( L/B )</th>
</tr>
</thead>
<tbody>
<tr>
<td>( \leq 0.5 )</td>
<td>0.1</td>
</tr>
<tr>
<td>( 10.0 )</td>
<td>1.40</td>
</tr>
<tr>
<td>( 20.0 )</td>
<td>1.55</td>
</tr>
<tr>
<td>( \geq 40.0 )</td>
<td>1.80</td>
</tr>
</tbody>
</table>

Note: (1) These coefficients are to be used with Method-2 given in Sec 2.4.6.6a(ii). Use \( C_p = \pm 0.7 \) for roof in all cases.

(2) Linear interpolation may be made for intermediate values of \( h/B \) and \( L/B \).

Here,

\[ \frac{L}{B} = 0.85; \]
\[ \frac{h}{B} = 1.04; \]
\[ \therefore C_p = 1.49 \]

Table 6: Equivalent wind forces at different floor levels.

<table>
<thead>
<tr>
<th>Height (m)</th>
<th>( C_z )</th>
<th>( q_z )</th>
<th>( p_z = \frac{kN}{m^2} )</th>
<th>( F_z = P_z \times A ) (kN)</th>
<th>( F_z ) (kN)</th>
<th>( F ) (Kip)</th>
</tr>
</thead>
<tbody>
<tr>
<td>3.048</td>
<td>0.368</td>
<td>0.76544</td>
<td>1.63</td>
<td>1.63<em>3.0478</em>14.6</td>
<td>73.54</td>
<td>16.42</td>
</tr>
<tr>
<td>6.096</td>
<td>0.415</td>
<td>0.8632</td>
<td>1.84</td>
<td>1.84<em>3.0478</em>14.6</td>
<td>81.88</td>
<td>18.28</td>
</tr>
<tr>
<td>9.144</td>
<td>0.498</td>
<td>1.03584</td>
<td>2.21</td>
<td>2.21<em>3.0478</em>14.6</td>
<td>98.35</td>
<td>21.95</td>
</tr>
<tr>
<td>12.192</td>
<td>0.57</td>
<td>1.1856</td>
<td>2.53</td>
<td>2.53<em>3.0478</em>14.6</td>
<td>112.6</td>
<td>25.13</td>
</tr>
<tr>
<td>15.24</td>
<td>0.63</td>
<td>1.3104</td>
<td>2.79</td>
<td>2.79<em>3.0478/2</em>14.6</td>
<td>62.08</td>
<td>13.86</td>
</tr>
</tbody>
</table>
1.6 Design of Floor Slabs

Figure 14: Typical floor plan

a) Assumptions and considerations

f’c=3000 psi
fy= 60000 psi

Thickness, \( t = \frac{\text{long length}(0.8 + \frac{fy}{200000})}{36 + 9 \beta} \)
Considering the largest two panels of 22'-10"x13'-2" and 22'-10"x11'-6

So, $\beta = \frac{22.83}{13.17} = 1.73$

Thickness, $t = 5.8$ in. $\approx 5.5$ in.

**b) Load calculation**

Self-weight of slab = $\frac{15}{12} \times 150 = 69$ psf

Floor finish = 30 psf
Partition wall = 40 psf
Live Load = 40 psf (BNBC)

$W_{DL} = 69 + 30 + 40 = 139$ psf $\times 1.2 = 167$ psf
$W_{LL} = 40 = 40$ psf $\times 1.6 = 64$ psf

Total, $W = (167+64) = 231$ psf

$m = \frac{23.17}{22.83} = 0.58 ~ 0.6$ and case 4

$m = \frac{11.5}{22.83} = 0.5$ and case 9

**Table 7:** Moment coefficients for two-way slabs. (Ref: BNBC 2006)

<table>
<thead>
<tr>
<th>Conditions</th>
<th>Case 4</th>
<th>Case 9</th>
</tr>
</thead>
<tbody>
<tr>
<td>- $C_A$</td>
<td>0.089</td>
<td>0.088</td>
</tr>
<tr>
<td>- $C_B$</td>
<td>0.011</td>
<td>0.003</td>
</tr>
<tr>
<td>+$C_{A(DL)}$</td>
<td>0.053</td>
<td>0.038</td>
</tr>
<tr>
<td>+$C_{B(DL)}$</td>
<td>0.007</td>
<td>0.002</td>
</tr>
<tr>
<td>+$C_{A(LL)}$</td>
<td>0.067</td>
<td>0.067</td>
</tr>
<tr>
<td>+$C_{B(LL)}$</td>
<td>0.009</td>
<td>0.004</td>
</tr>
</tbody>
</table>

From judgment it can be said that the slab will be critical in short direction only.
c) Moment and reinforcement calculation

For, case 4
Short distance A, $+M = \{C_A (w_d) * W_{(od)} * A^2\} + \{C_A (l_d) * W_{(ld)} * A^2\} = 2.22 k - ft/ft$
short distance A, $-M = \{-C_A * W * A^2\} = 3.57 k-ft/ft$

For, case 9
Short distance A, $+M = \{C_A (w_d) * W_{(od)} * A^2\} + \{C_A (l_d) * W_{(ld)} * A^2\} = 1.41 k-ft/ft$
short distance A, $-M = \{-C_A * W * A^2\} = 2.69 k-ft/ft$

So, in short direction $-M = 3.5 k-ft/ft$ and $+M = 2.22 k-ft/ft$

$A_{s_{min}} = 0.002 * b * t = 0.002 * 12 * 5.5 = 0.132 \text{ in}^2/ft$

$$A_s = \frac{M_{12}}{f_y \phi \rho_{\text{cd}} \rho_{\text{od}} \rho_{\text{ld}} (d - \frac{a}{2})} = 0.11 \text{ in}^2/ft$$

$$\alpha = \frac{A_s f_y}{0.85 f_y c_w b} = \frac{0.11 * 60}{0.85 * 3 * 12} = 0.2 \text{ (ok)}$$

Now, $\frac{0.11 * 12}{0.13} = 10.15"$; use, $\phi 10\text{mm}@10"$ c/c alt. ckd.

Again,

$$A_s = \frac{M_{12}}{f_y \phi \rho_{\text{cd}} \rho_{\text{od}} \rho_{\text{ld}} (d - \frac{a}{2})} = 0.18 \text{ in}^2/ft \text{ (controlled)}$$

$$\alpha = \frac{A_s f_y}{0.85 f_y c_w b} = \frac{0.18 * 60}{0.85 * 3 * 12} = 0.3 \text{ (ok)}$$

The distance between two cranked rods is 20".

So, Required reinforcement = $0.18 - \frac{0.11 * 12}{20} = 0.114 \text{ in}^2/ft$

The extra negative reinforcement required, $20 / (\frac{0.11 * 12}{0.114}) = 20/11.58 = 1.73 \sim 2$ So, use 2-Ø10mm as extra top.

By observing the moment coefficients it can be said that, all the reinforcement in long direction will be controlled by $A_{s_{min}}$.

So, the reinforcement will be $\phi 10\text{mm}@10"$ c/c alt. ckd and 2-Ø10mm as extra top.
Figure 15: Reinforcement Details of Slab
1.7 Design of Floor Beams

**Figure 16:** Beam layout

**a) Assumptions and considerations**

Load on slab, \( W = 231 \text{ psf} \)

\( f' = 3000 \text{ psi} \)

\( f_y = 60000 \text{ psi} \)

**b) Load calculation**

Beam in-between A and B grid on grid 2

Trapezoidal panel:
\[ T_8 = \frac{1}{2} \times (22.92 + 11.42) \times 5.75 = 98.73 \, \text{ft}^2 \approx T_{10} \]

\[ T_{11} = \frac{3}{2} \times (22.92 + 9.75) \times 6.585 = 107.56 \, \text{ft}^2 \approx T_{13} \]

Assuming, a beam of width 12'' and height 18''

Self-weight = \( \frac{12 \times 18}{144} \times \frac{150}{1000} = 0.225 \, \text{kip/ft} \times 1.2 = 0.27 \, \text{kip/ft} \)

Load from Slab = \( \frac{0.231 \times 98.73}{22.92} + \frac{0.231 \times 107.56}{22.92} = 2.08 \, \text{kip/ft} \)

Partition wall on beam = 0.42 \times 9 \times 120 = 0.45 \times 1.2 = 0.54 \, \text{k/ft}

Total load = 0.27 + 2.08 + 0.54 = 2.89 \, \text{kip/ft}

c) Moment and reinforcement

At grid 3-A joint

- \[ M_u = \frac{w l^2}{16} = \frac{2.89 \times 22.92^2}{16} = 94.9 \, \text{kip-ft} = 1138.64 \, \text{kip-in} \]

At grid 3-C joint

- \[ M_u = \frac{w l^2}{9} = \frac{2.89 \times 22.92^2}{9} = 168.7 \, \text{kip-ft} = 2024.26 \, \text{kip-in} \]

At mid span

- \[ M_u = \frac{w l^2}{14} = \frac{2.89 \times 22.92^2}{14} = 108.44 \, \text{kip-ft} = 1301.3 \, \text{kip-in} \]

Here, \( d = 18 - 2.5 - 2 = 13.5'' \)

From table A.4 [Tension controlled], [Ref:Nilson pg:745]

\( \rho_{0.05} = 0.0135 \) and \( \varphi = 0.9 \)

\( A_s = \rho_{0.05} \times b \times d = 0.0135 \times 12 \times 13.5 = 2.2 \, \text{in}^2 \)

\( a = \frac{A_s f_y}{0.85 f_c' b} = \frac{2.2 + 60}{0.85 \times 3 \times 12} = 4.31'' \)

\( a < h_f, \) Rectangular beam analysis.

\[ c = \frac{a}{\beta_1} = \frac{4.31}{0.85} = 5.07'' \]
\[ M_n = A_s f_y \left( d - \frac{a}{2} \right) = 2.2 \times 60 \times \left( 13.5 - \frac{4.31}{2} \right) = 1497.54 \text{ kip - in} \]

\[ \varphi M_n = 0.9 \times 1497.54 = 1347.8 \text{ k - in} > M_u = 1301.3 \text{ kip - in} \]

The beam will be designed as singly reinforcement for midspan and grid 3-A joint.

\[ \varphi M_n = 0.9 \times 1497.54 = 1347.8 \text{ k - in} < M_u = 2024.26 \text{ kip - in} \]

The beam will be designed as doubly reinforcement for grid 3-C joint. Compression reinforcement is required as well as tension reinforcement.

**For grid 3-A joint,**

Assume, \( a = 5'' \)

\[ -A_s = \frac{M_u / \varphi}{f_y (d - a/2)} = \frac{1138.64 / 0.9}{60 (13.5 - 5/2)} = 1.92 \text{ in}^2 \]

\[ a = \frac{A_s f_y}{0.85 f_c' b} = \frac{1.92 \times 60}{0.85 \times 3 \times 12} = 3.76'' \]

\[ -A_s = \frac{1138.64 / 0.9}{60 (13.5 - 3.76/2)} = 1.81 \text{ in}^2 \]

\[ a = \frac{1.81 \times 60}{0.85 \times 3 \times 12} = 3.55'' \]

\[ -A_s = 1.79 \text{ in}^2 \]

**For midspan,**

Assume, \( a = 3'' \)

\[ +A_s = \frac{M_u / \varphi}{f_y (d - a/2)} = \frac{1301.3 / 0.9}{60 (13.5 - 3/2)} = 2.00 \text{ in}^2 \]

\[ a = \frac{A_s f_y}{0.85 f_c' b} = \frac{2 \times 60}{0.85 \times 3 \times 12} = 3.92'' \]

\[ +A_s = \frac{1301.3 / 0.9}{60 (13.5 - 3.92/2)} = 2.09 \text{ in}^2 \]
\[ a = \frac{2.09 \times 60}{0.85 \times 3 \times 12} = 4.1\text{''} \]

\[ +A_s = 2.1 \text{ in}^2 \]

**For grid 3-C joint**, Remaining moment, \[ M_1 = \frac{2024.26}{0.9} - 1497.54 = 751.64 \text{ kip} \text{ - in} \]

Using the strain distribution,

\[ \varepsilon' = \varepsilon_u \frac{c - d'}{c} = 0.003 \times \frac{5.07 - 2.5}{5.07} = 0.0015 \]

\[ f'_s = \varepsilon'_s E_s = 0.0015 \times 29000 = 43.5 \text{ ksi} \]

Compression reinforcement for grid 3-C joint,

\[ A'_s = \frac{751.64}{43.5 \times (13.5 - 2.5)} = 1.57 \text{ in}^2 \]

Total area of tensile reinforcement at 60 ksi, \[ A_s = 2.2 + 1.57 \times \frac{43.5}{60} = 3.34 \text{ in}^2 \]

The summaries of reinforcement are as follows,

At mid span, \[ +A_t = 2.1 \text{ in}^2 \]

Grid 3-A joint, \[ -A_s = 1.79 \text{ in}^2 \]

Grid 3-C joint, \[ -A_s = 3.34 \text{ in}^2 \text{ (tension) and compressive reinforcement, 1.57 in}^2 \]

Now, for structural integrity minimum 1/3 reinforcement need to be provided all through the beam and compressive reinforcement at Grid 3-C.

Provide \[ \frac{2.1}{3} = 0.7 \text{ in}^2 \] all through as positive reinforcement but it is less than compressive reinforcement 1.57 in².

Provide \[ \frac{3.34}{3} = 1.11 \text{ in}^2 \] all through as negative reinforcement. For a beam having width of 12", it is difficult to place more than three reinforcement in a row and more than five reinforcement in a face.

**d) Shear design**

\[ V_u = 0.5WL = 0.5 \times 2.89 \times 22.92 = 33.12 \text{ k} \]

\[ \varnothing \times V_u = 2 \times \varnothing \times \sqrt{f'_c} b \times d = 2 \times 0.75 \times \sqrt{3000} \times 12 \times 13.5 = 13.3 \text{ kip} \]

Use Ø10mm as shear reinforcement.

\[ s_{max} = \frac{A_{ofy}}{50b_w} = \frac{2 \times 0.121 \times 60000}{50 \times 12} = 24'' \]
\[ s_{\text{max}} = \frac{13.5}{2} = 6.5'' \text{(govern)} \]

\[ s_{\text{max}} = 24'' \]

\[
s = \frac{\varnothing \alpha f_y d}{V_u - \varnothing V_u} = \frac{0.75 \times 2 + 1.21 \times 6 + 13.5}{33.12 - 13.3} = 6.67''
\]

So, provide Ø10mm @ 6.5" c/c all through the beam.

Symmetric beam, so providing same reinforcement in B14 and B15. Design the beams for the load combinations as mentioned in BNBC using Approximate method for gravity load and Portal method for lateral load.

**Figure 17:** Reinforcement detail of beam
1.8 Design of Column

**Figure 18**: Interaction diagram for compression plus biaxial bending a) uniaxial bending about Y axis; b) uniaxial bending about X axis; c) biaxial bending about diagonal axis; d) interaction surface. (Ref: ACI Code, Design of Concrete Structure, 13th edition, Chap-8, P-274)
**Figure 19**: Interaction diagram for nominal column strength in combined bending and axial load. (Ref: ACI Code, Design of Concrete Structure, 13th edition, Chap-8, P-260)

**a) Assumptions and considerations**

\( f_y = 60000 \text{ psi} \)
\( f'_c = 4000 \text{ psi} \)

For a column,

\( P = 554 \text{ K} \)
\( M_x = 85 \text{ K-ft} \)
\( M_y = 120 \text{ K-ft} \)

For, tied column, due to accidental eccentricity strength reduction factor \( \alpha = 0.8 \) and

Based on importance strength reduction factor \( \phi = 0.65 \), (ACI Code, Design of Concrete Structure, 13th edition, Chap-8, P-252)
let, \( \rho_g = 2\% \)

Now, \( \phi P_n = \alpha \phi [0.85 f^c A_g + \rho_g A_g f_y] \)

\[
554 = 0.65 \times 0.8 \times [0.85 \times 4 \times A_g + 0.02 \times A_g \times f_y]
\]

\( A_g = 232 \text{ in}^2 \)

Let, 18"x15"

For My or dimension parallel to X axis,

\[
\gamma = d_x / D_x = (18 - 2.5 \times 2) / 18 = 0.72 \approx 0.7
\]

Eccentricity \( e_x = M_y / P = 120 / 554 = 0.21' = 2.6" \)

\( e_x / h = 2.6 / 18 = 0.14 \)

From graph, \( K_\eta = 0.79 \)

\[
\frac{P_y}{f^c A_g} = 0.79
\]

\( P_y = 853 \text{ k} \)

For Mx or dimension parallel to Y axis,

\[
\gamma = d_y / D_y = 0.67 \approx 0.6
\]

\( e_y = 85 / 554 = 0.15' = 1.8" \)

\( e_y / h = 1.8 / 15 = 0.12 \)

From graph, \( K_\eta = 0.85 \)

\[
\frac{P_x}{f^c A_g} = 0.85
\]

\( P_x = 918 \text{ k} \)

For \( P_o \), \( K_\eta = (1.1 + 1.12) / 2 = 1.11 \)

\[
\frac{P_o}{f^c A_g} = 1.11
\]

\( P_o = 1200 \text{ k} \)
\[ \frac{1}{P_n} = \frac{1}{P_0} + \frac{1}{P_y} - \frac{1}{P_0} \]

\[ = \frac{1}{510} + \frac{1}{650} - \frac{1}{1000} \]

\[ \phi \ P_n = 0.65 \times 700 \text{ k} = 455 \text{ k} < 554 \text{ k} \text{ (not ok)} \]

**Figure 20:** Minimum spacing between reinforcement bars

The distance between reinforcement bars must be such to allow the largest expected concrete size gravel to pass between them. In order to have properly anchored reinforcement, it is mandatory for rebars to be surrounded by concrete.

The minimum spacing between two reinforcement bars should be at least equal to the maximum coarse aggregate dimension plus a margin of 5 mm.
b) Tie bar

$\Phi 10\, mm$ bars are used.

**Longitudinal Spacing**

$16 \, d_b \, of \, main \, bar = 16\times20/25.4 = 12''$

$48 \, d_b \, of \, tie \, bar = 48\times10/25.4 = 18''$

Least dimension = 15"

So, spacing at top and bottom $12/2 = 6'' \, c/c$ and at middle span $12'' \, c/c$.

![Figure 21: Failure mechanism of a column](image)

A column with 10% fewer rebars has around 10% lower capacity strength. However, if we remove even a single intermediate stirrup, the capacity strength of that same column will be lowered even by 50%. This happens because the stirrup’s removal doubles the buckling length of the rebars previously enclosed by it.

**Cross sectional Spacing**

# the reinforcement at a distance greater than 6" from the outer most bar should be under a lateral tie and

# Alternate bar should be under lateral tie.
Figure 22: Tie arrangement of rectangular column ((Ref: ACI Code, Design of Concrete Structure, 13th edition, Chap-8, P-254)
Figure 2.3: Standard bar hook for tie and stirrup. (Ref: ACI Code, Design of Concrete Structure, 13th edition, Chap-5, P-177)

Figure 2.4: Typical column detail
References

- ACI code 318-14, American Concrete Institute, 2014.
- Concrete Technology by Neville.
- Design of RCC Members by WSD and USD Methods, Public Works Department (PWD), 1997.
- www.buildinghow.com
Part 2: Preliminary Design of the Superstructure of a Balanced Cantilever Bridge for Gravity loading

2.1 LECTURE PLAN

Lecture 1
- Introduction to Bridge Engineering
- About Balanced Cantilever Bridge

Lecture 2
- Design of Deck Slab, Railing, Post and Sidewalk
- Design of Interior Girder

(Dead load Calculation, Shear force diagram, Bending Moment Diagram for dead load)

Lecture 3 & 4
- Design of Interior Girder, Exterior Girder

(SFD & BMD for live load including truck load, tandem load and Lane load at different sections, Corresponding Impact shear & moment, Design of reinforcement for shear & moment)

Lecture 5
- Design of Cross Girder/ Diaphram and Articulation

2.2 SUBMISSION GUIDELINE OF BRIDGE DESIGN

The Design Report shall explain the details of the design process. It shall include the following items:

- Design Specification, Standards followed in Analysis & Design
- Loads and Load Combinations
- Design of Slab
- Design of Railing, Post and Sidewalk
- Design of Interior Girder
- Design of Exterior Girder
- Design of Diaphrams or Cross Girders
- Design of Articulation

[Note: Appropriate hand sketches showing the details of reinforcements must accompany all design calculations.]
2.3 INTRODUCTION TO BRIDGE ENGINEERING

a) What is a Bridge?

A Bridge is a structure providing passage over an obstacle without closing the way beneath.
The required passage may be for a road, a railway, pedestrians, a canal or a pipeline.

b) Requirements of an Ideal Bridge

Economical
Serves the intended functions with safety and convenience
Aesthetic elegant look

c) Selection of Bridge Site

A straight reach of the river
Steady river flow without serious whirls and cross currents
A narrow channel with firm banks
Suitable high banks above high flood level on each side
Rock or other hard strata close to the river bed level
Absence of sharp curves in the approaches
Avoidance of excessive underwater construction
Avoidance of expensive river training work
Proximity to a direct alignment of the connected road

d) Choice of a type of a Bridge

Channel Section
Sub-soil condition
Grades and Alignment
Hydraulic Data
Weather
Navigation requirements
Economic and Strategic considerations
Labour availability
Materials of Construction available
Period of Construction
Type of loading
Erection Facilities

e) Types of Bridge (based on action)

Slab Bridge
Deck-girder Bridge
Balanced- Cantilever Bridge

Suspension Bridge
Cable-stayed Bridge

Fig. 1: Deck-girder Bridge – *Niteroi Bridge, Rio De Janeiro, Brazil*
Fig. 2: Arch Bridge – *Sydney Harbour Bridge, Australia*

Fig. 3: Truss Bridge – *Ikitsuki Bridge, Nagasaki, Japan*
Fig. 4: Cable-stayed Bridge – *Rion Antirion Bridge, Greece*

Fig. 5: Suspension Bridge – *Akashi Kaikyo Bridge, Japan*
f) Types of Bridge (based on type of Support)

- Simply-Supported Bridge
- Continuous Bridge
- Fixed Bridge
- Cantilever Bridge

g) Types of Bridge (based on material)

- Concrete/ R.C.C Bridge
- Steel Bridge
- Stone Bridge
- Timber Bridge
- Composite Bridge
Table 1: Classification of Bridge (based on span length)

<table>
<thead>
<tr>
<th>Main Span Length</th>
<th>Type of Bridge</th>
</tr>
</thead>
<tbody>
<tr>
<td>0-10m</td>
<td>Beam/ Girder R.C.C Bridge</td>
</tr>
<tr>
<td>10-50m</td>
<td>Precast Concrete (PCC) I- Girder Bridge</td>
</tr>
<tr>
<td>50-100m</td>
<td>Prestressed (PSC) concrete Box-Girder Bridge</td>
</tr>
<tr>
<td>100-200m</td>
<td>Composite Bridge (Steel Girder &amp; Steel-Concrete Composite Slab)</td>
</tr>
<tr>
<td>&gt;200m</td>
<td>PSC Extrados Bridge</td>
</tr>
<tr>
<td>1000-1500m (1-1.5km)</td>
<td>Cable-Stayed Bridge</td>
</tr>
<tr>
<td>&gt;1500m (1.5km)</td>
<td>Suspension Bridge</td>
</tr>
</tbody>
</table>

**h) Different Parts of a Bridge**

- **Foundation**: The portion below the bed level of a river.
- **Substructure**: The parts below the bearings level and above the foundation.
- **Superstructure**: Components above the level of bearings.

Fig. 7: Different parts of a Bridge
i) Components of a Bridge

Deck Slab
Girder
Diaphragm or Cross Girder
Bearings for the decking
Abutment, Wingwall
Pier, Viaduct
Foundation (i.e. Pile)
Handrail, Curb/ Sidewalk

Approach to the Bridge *(to connect the bridge proper to the roads on either side)*
j) BRIDGE TERMINOLOGY

**Abutment**
- The end supports of the superstructure of a bridge.
- Supports the bridge deck at the ends.
- Retains the approach road embankment.

**Wing walls**
- The walls constructed on both sides of the abutments.
- Anchor the bridge to its approach road.
- Support the embankments of approach road.
- Protect the embankments from the wave action of running water.
Curb/Sidewalk

Raised portion of a roadway slab on both sides.
Provided to check the vehicle to fall out the bridge.
Width of 60cm & Height of 22.5 cm are adopted.
Roadside slope is kept as 1 in 8 upto 20cm & top portion is curved.

Footpath

The passage where only pedestrians are allowed to walk.
Width may be taken as 1.5 to 2.2 metre.

Handrail

Protective measures adopted to prevent the falling to river of the bridge users.

Pier

Intermediate supports of the superstructure of a bridge.
Transfer load from the superstructure to the sub-soil through the foundation.
Obstruct the flow of water on the upstream.
Facilitate a long bridge to be converted into segments.
Afflux

The rise in water level of the river near bridge due to obstruction created by obstruction of piers.

Afflux = Difference of levels of downstream and upstream water surface of bridge.

Freeboard

The difference between the high flood level and the level of the crown of the road at its lowest point.

Approaches/ Embankments

The structures that carry the road or railway track upto the bridge.

Approach Slab

The slab provided to join the approach road with the bridge.

One end rests on the backfill of the abutment and extends into the approach at least by 3.5m.

Backfill

Materials used to fill the space at the back of the bridge.

They are the broken stone, gravel, sand etc. and should be clean.
Total Span & Total Clear Span
The centre to centre distance between the end supports of a bridge is termed as total span.
Clear distance between the end supports is termed as total clear span.

Span & Clear Span
The centre to centre distance between any two adjacent supports is termed as span.
Clear distance between any two adjacent supports is termed as clear span.

Headroom
The distance between the highest point of the vehicle using that bridge and the lowest point of any protruding member of the bridge.

High Flood Level (HFL)
The highest water level ever recorded during a flood in a river or stream.

Low Flood Level (LFL)
The lowest water level in a river or stream during dry weather

Mean or Ordinary Flood Level (MFL)
The flood level that normally occurs every year.
k) Softwares for Bridge Design

- SAP 2000
- CSiBridge
- ADAPT ABI 2012
- Structural Bridge Design
- CRSI (Slab Bridge Designer)
- ANSYS Civil FEM Bridge
- MIDAS

2.4 ABOUT BALANCED CANTILEVER BRIDGE

a) Multiple simply supported span bridge

![Diagram of a bridge with simply supported span]

**Advantage**
- Determinate structure:
- No stress due to differential settlement.

**Disadvantage**
- Large magnitude of bending moment requiring bigger and heavier section: uneconomic

Fig. 13: A bridge having simply supported span
b) Continuous span bridge

Fig. 14: A bridge having continuous span

Advantage
Magnitude of maximum moment reduced:
Resulting in economic section

Disadvantage
Large bending moment due to uneven/differential settlement

c) What is a Balanced Cantilever Bridge?

- A cantilever bridge is a bridge built using cantilevers, structures that project horizontally into space, supported on only one end.

- The suspended span is designed as a simply supported span with supports at the articulations.

- A simple cantilever span is formed by two cantilever arms extending from opposite sides of an obstacle to be crossed.
d) Developing the idea of Cantilever form

Fig. 15: A bridge having intermediate hinges

e) Advantages of Balanced Cantilever Bridge

Being a Determinate Structure.
The problem of large stress due to differential support settlement is eliminated due to the internal hinges.
The design section becomes economic.
Less concrete, steel are required for cantilever design.

f) Disadvantages of Balanced Cantilever Bridge

Requires a little more skill on the part of the designer.
Requires more elaborate detailing of the reinforcements.
Articulations are very congested with steel and anchorages.
2.5 DETAILS OF SOME EXISTING BRIDGES

a) World’s largest Cantilever Bridge- Quebec Bridge, CANADA

Fig. 16: Quebec bridge, CANADA

- **Total length:** 987 m (3,239 ft)
- **Width:** 29 m (94 ft) wide
- **Longest span:** 549 m (1,800 ft)
- **Opened:** December 3, 1919
- **Carries:** 3 lanes of roadway
  - 1 rail line
  - 1 pedestrian walkway
- **Crosses:** St. Lawrence River
b) Bangladesh China Friendship Bridge

- **Bridge Type**: Pre-stressed concrete box girder
- **Length**: 151 m (over river *Dhaleswari* on *Dhaka-Munshigonj road*)
- **Width**: 10 m (carriage way - 7.5 m & sidewalk - 2x1.25 m)
- **No. of Lanes**: 2 Lanes
- **No. of Span**: 37 nos.
- **No. of Abutment**: 2 nos.
- **No. of Piers**: 38 nos.
- **Type of Foundation**: Pile foundation

![Bangladesh China Friendship Bridge](Fig.17)  
*Source: Googlemap*
Fig. 18: Spans of Bangladesh China Friendship Bridge
Fig. 19: Articulation/ Halving joint

Fig. 20: A back view showing diaphragm/cross girder and longitudinal girder
Fig. 21: Sebastian Intel Bridge, Florida, USA
**Support Details**

Fig. 22: Support details of Sebastian Intel Bridge, Florida, USA

Fig. 23: Diaphragm or cross girder of Sebastian Intel Bridge, Florida, USA
2.6 LOADS ON BRIDGE

- Dead load
- Live load (i.e. Vehicles and Pedestrians)
- Dynamic or Impact effect of live load
- Wind loading
- Seismic Forces
- Buoyancy
- Water current forces
- Thermal Forces
- Erection Forces
- Earth Pressure
- Centrifugal Forces (for curved deck)
- Longitudinal Forces (for stopping vehicle)
- Ice loading
Loads on Bridge (AASHTO 2012, Sec. 3.3.2)
The following permanent and transient loads and forces are considered to act on a bridge structure:

- **CR** = force effects due to creep
- **DD** = downdrag force
- **DC** = dead load of structural components and nonstructural attachments
- **DW** = dead load of wearing surfaces and utilities
- **EH** = horizontal earth pressure load
- **EL** = miscellaneous locked-in force effects resulting from the construction process, including jacking apart of cantilevers in segmental construction
- **ES** = earth surcharge load
- **EV** = vertical pressure from dead load of earth fill
- **PS** = secondary forces from post-tensioning
- **SH** = force effects due to shrinkage

- **BL** = blast loading
- **BR** = vehicular braking force
- **CE** = vehicular centrifugal force
- **CT** = vehicular collision force
- **CV** = vessel collision force
- **EQ** = earthquake load
- **FR** = friction load
- **IC** = ice load
- **IM** = vehicular dynamic load allowance
- **LL** = vehicular live load
- **LS** = live load surcharge
- **PL** = pedestrian live load
- **SE** = force effect due to settlement
- **TG** = force effect due to temperature gradient
- **TU** = force effect due to uniform temperature
- **WA** = water load and stream pressure
- **WL** = wind on live load
- **WS** = wind load on structure
c) Vehicular Live load

Fig. 25: Design Truck load (HS20-516)

DESIGN TANDEM LOAD

TANDEM load is subjected to dynamic allowance (impact)

Fig. 26: Design Tandem load
Fig. 27: Design Lane load

1. **Standard lane width**: 12 ft, Load occupies 10 ft width across lane.
2. **Fractional lanes not permitted**.
3. **For total bridge load**: lane loads may be reduced as follows:
   - 1 or 2 lane bridge: No reduction
   - 3 lanes: 90 percent
   - 4 or more lanes: 75 percent

LANE load is NOT subjected to dynamic allowance (impact)
DESIGN VEHICULAR LIVE LOAD

Vehicular live loading on the roadways of bridges or incidental structures, designated HL-93, shall consist of a combination of the:

- Design truck or design tandem, and
- Design lane load.

Each design lane under consideration shall be occupied by either the design truck or tandem, coincident with the lane load, where applicable. The loads shall be assumed to occupy 10.0 ft transversely within a design lane.

Dynamic Effect of Live Load (for Truck or Tandem)

IMPACT ALLOWANCE

- The term impact as ordinarily used in structural design refers to the dynamic effect of a suddenly applied load.

- In the building of a structure, the materials are added slowly; people entering a building are also considered a gradual loading. Dead loads are static loads; i.e., they have no effect other than weight.

- Live loads may be either static or they may have a dynamic effect. Any live load that can have a dynamic effect should be increased by an impact factor. While a dynamic analysis of a structure could be made, such a procedure is unnecessary in ordinary design. Thus, empirical formulas and impact factors are usually used.

- For highway bridge design, impact is always to be considered. AASHTO prescribes empirically that the static effect of live load be multiplied by a factor

\[(1 + IM/100)\]

To take into account the dynamic effect of live load.
LIMIT STATES:

**Strength I**—Basic load combination relating to the normal vehicular use of the bridge without wind.

**Strength II**—Load combination relating to the use of the bridge by Owner-specified special design vehicles, evaluation permit vehicles, or both without wind.

**Strength III**—Load combination relating to the bridge exposed to wind velocity exceeding 55 mph.

**Strength IV**—Load combination relating to very high dead load to live load force effect ratios.

**Strength V**—Load combination relating to normal vehicular use of the bridge with wind of 55 mph velocity.

**Extreme Event I**—Load combination including earthquake. The load factor for live load $\gamma_{EQ}$ shall be determined on a project-specific basis.

**Extreme Event II**—Load combination relating to ice load, collision by vessels and vehicles, check floods, and certain hydraulic events with a reduced live load other than that which is part of the vehicular collision load, $CT$. The cases of check floods shall not be combined with $BL$, $CV$, $CT$, or $IC$.

**Fatigue I**—Fatigue and fracture load combination related to infinite load-induced fatigue life.

**Fatigue II**—Fatigue and fracture load combination related to finite load-induced fatigue life.
For the present case

\[ DC = \text{Self weight of structural components} \]

\[ DW = \text{Weight of wearing course} \]

\[ LL = \text{Lane load with vehicle or tandem} \]

\[ IM = \text{Impact effect of vehicle or tandem load} \]

\[ PL = \text{Pedestrian load} \]

\[ \gamma_p(DC) + \gamma_p(DW) + 1.75(LL)(1+IM/100)_{\text{Truck/Tandem}} + 1.75(LL)_{\text{Lane}} + 1.75(PL) \]

\[ = 1.25(DC) + 1.5(DW) + 1.75(LL)(1+IM/100)_{\text{Truck/Tandem}} + 1.75(LL)_{\text{Lane}} + 1.75(PL) \]

2.7 DESIGN OF DIFFERENT COMPONENTS

(a)
Fig. 28: Longitudinal profile a three spanned balanced cantilever bridge

- Total span: \( L \)
- End span: \( L_E \)
- Middle span: \( L_M \)
- Suspended span: \( L_S \)
- Cantilever span: \( L_C \)

**Assumed Relations**

- \( L_M = 1.4 \, L_E \)
- \( L_C = 0.3 \, L_S \)
- \( H \geq 0.07L_S \) (Table 2.5.2.6.3-1, AASHTO 2012)
Assume

\[ b_w = \frac{H}{4} \sim \frac{H}{3} > 18'' \]

\[ t \geq \frac{S}{12} \geq 7'' \]

Typical section A-A

Fig. 29: Transverse section
Design Data for Students:

**COMMON DATA**
Wearing course, $w_{wc} = 30$ psf  
Width of side walk = 3′-6″

**Laneway width**
- **Sec-A**: 14′
- **Sec-B**: 13′
- **Sec-C**: 12′

Number of lanes = 2  
Concrete clear cover = Beam 1.5″, Slab: 1.0″

**Girder depth at pier**
$H_p = 2.0H$ for $L < 350′$,  
$= 1.5H$ for $L \geq 350′$

**PER STUDENT DATA**

<table>
<thead>
<tr>
<th>Student</th>
<th>Total Span, L ft</th>
<th>$f_c$ (ksi)</th>
<th>$f_y$ (ksi)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>250</td>
<td>4</td>
<td>72</td>
</tr>
<tr>
<td>2</td>
<td>253</td>
<td>4</td>
<td>72</td>
</tr>
<tr>
<td>3</td>
<td>256</td>
<td>4</td>
<td>60</td>
</tr>
<tr>
<td>4</td>
<td>259</td>
<td>4</td>
<td>60</td>
</tr>
<tr>
<td>5</td>
<td>262</td>
<td>4</td>
<td>60</td>
</tr>
<tr>
<td>6</td>
<td>265</td>
<td>4</td>
<td>60</td>
</tr>
<tr>
<td>7</td>
<td>268</td>
<td>4</td>
<td>60</td>
</tr>
<tr>
<td>8</td>
<td>271</td>
<td>4</td>
<td>60</td>
</tr>
<tr>
<td>9</td>
<td>274</td>
<td>4</td>
<td>60</td>
</tr>
<tr>
<td>10</td>
<td>277</td>
<td>4</td>
<td>60</td>
</tr>
<tr>
<td>11</td>
<td>280</td>
<td>4</td>
<td>60</td>
</tr>
<tr>
<td>12</td>
<td>283</td>
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<td>60</td>
</tr>
<tr>
<td>13</td>
<td>286</td>
<td>4</td>
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<td>4</td>
<td>60</td>
</tr>
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<td>16</td>
<td>295</td>
<td>4</td>
<td>60</td>
</tr>
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<td>298</td>
<td>4</td>
<td>60</td>
</tr>
<tr>
<td>18</td>
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<td>5</td>
<td>72</td>
</tr>
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<td>5</td>
<td>72</td>
</tr>
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<td>20</td>
<td>307</td>
<td>5</td>
<td>72</td>
</tr>
<tr>
<td>21</td>
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<td>5</td>
<td>72</td>
</tr>
<tr>
<td>22</td>
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<td>5</td>
<td>72</td>
</tr>
<tr>
<td>23</td>
<td>316</td>
<td>5</td>
<td>72</td>
</tr>
</tbody>
</table>
Instructions for Students

Follow the serial number of the students given in the previous table as starting from the smallest to upper student number for each section which will be provided in the class. Draw SFD, BMD of interior girder due to dead load and also verify those results using software.

Draw influence line diagram for shear and moment at the assigned sections and also verify them using software for at least three sections.

a) DESIGN OF DECK SLAB

Design for Dead Load

Fig. 30: Dead load on deck slab
Design for Vehicular Live load

Detailed analysis can be performed based on influence line to determine the maximum effect. Alternatively, Table A4-1 in Appendix A4 of AASHTO 2012 can be used.

Fig. 31: Vehicular live load on deck slab
Table A4-1 in Appendix A4 of AASHTO 2012, page 4-98

Important Assumptions...

- Multiple presence factors and the dynamic load allowance are included in the tabulated values.
- The moments are applicable for decks supported on at least three girders and having a width of not less than 14.0 ft between the centerlines of the exterior girders.
- For each combination of girder spacing and number of girders, the following two cases of overhang width were considered:
  - Minimum total overhang width of 21.0 in. measured from the center of the exterior girder, and
  - Maximum total overhang width equal to the smaller of 0.625 times the girder spacing and 6.0 ft. A railing system width of 21.0 in. was used to determine the clear overhang width. For other widths of railing systems, the difference in the moments in the interior regions of the deck is expected to be within the acceptable limits for practical design. The moments do not apply to the deck overhangs and the adjacent regions of the deck that need to be designed taking into account the provisions of Article A13.4.1.

### DECK SLAB DESIGN: VEHICLE LOAD

**AASHTO LRFD BRIDGE DESIGN SPECIFICATIONS**

Table A4-1—Maximum Live Load Moments per Unit Width, kip-ft/ft

<table>
<thead>
<tr>
<th>Negative Moment</th>
<th>Distance from CL of Girder to Design Section for Negative Moment</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>0.0 in.</td>
</tr>
<tr>
<td><strong>S</strong></td>
<td></td>
</tr>
<tr>
<td>7’ –0”</td>
<td>5.21</td>
</tr>
<tr>
<td>7’ –3”</td>
<td>5.32</td>
</tr>
<tr>
<td>7’ –6”</td>
<td>5.44</td>
</tr>
<tr>
<td>7’ –9”</td>
<td>5.56</td>
</tr>
<tr>
<td>8’ –0”</td>
<td>5.69</td>
</tr>
<tr>
<td>8’ –3”</td>
<td>5.83</td>
</tr>
<tr>
<td>8’ –6”</td>
<td>5.99</td>
</tr>
<tr>
<td>8’ –9”</td>
<td>6.14</td>
</tr>
<tr>
<td>9’ –0”</td>
<td>6.29</td>
</tr>
<tr>
<td>9’ –3”</td>
<td>6.44</td>
</tr>
<tr>
<td>9’ –6”</td>
<td>6.59</td>
</tr>
<tr>
<td>9’ –9”</td>
<td>6.74</td>
</tr>
<tr>
<td>10’ –0”</td>
<td>6.89</td>
</tr>
<tr>
<td>10’ –3”</td>
<td>7.03</td>
</tr>
<tr>
<td>10’ –6”</td>
<td>7.17</td>
</tr>
<tr>
<td>10’ –9”</td>
<td>7.32</td>
</tr>
</tbody>
</table>
Reinforcement Design of Deck

### Resistance factor $\phi$

- **Moment**: 0.90

### Deck Slab

- $A_s \geq \frac{M_u}{\phi f_y \left( d - \frac{a}{2} \right)} \approx \frac{M_u}{\phi f_y (jd)}$
- Assume $jd \approx 0.95d$
- Check $A_s \geq A_{s,\text{min}} = \frac{200}{f_y} bd$

### Determine $\beta_1$

- Depth of neutral axis, $c = a/\beta_1$
- Check $c < \frac{3}{8}d$ (tension controlled)
- Finally, $\phi M_n = \phi A_s f_y \left( d - \frac{a}{2} \right) \geq M_u$

---

**General Load Combination**

$1.25(DC) + 1.5(DW) + 1.75(LL)(1+IM/100)_{\text{Truck/Tandem}} + 1.75(LL)_{\text{Lane}} + 1.75(PL)$

**Design slab moment**,

$$M = 1.25 M_{DC} + 1.5 M_{DW} + 1.75 M_{LL} \quad \left[ \rightarrow M_{\text{STRENGTH}} \right]$$

Where $M_{LL}$ is the live load slab moment from Table A4-1 which includes the impact effect.

$$M_{\text{SERVICE}} = M_{DC} + M_{DW} + M_{LL} \quad [\text{required for crack control calculations}]$$
FIGURE 3.9
Variation of strength reduction factor with net tensile strain in the steel.

\[ \phi = 0.75 + (\epsilon_t - 0.002)50 \]
\[ \phi = 0.75 + 0.15\left[1/(c/d) - 5/3\right] \]

\[ \phi = 0.65 + (\epsilon_t - 0.002)(250/3) \]
\[ \phi = 0.65 + 0.25\left[1/(c/d) - 5/3\right] \]

FIGURE 3.10
Net tensile strain and \(c/d\), ratios.

\[ \epsilon_t = 0.005 \]
\[ c/d_t = \frac{0.003}{0.003 + 0.005} = 0.375 \]

(a) Tension-controlled member

\[ \epsilon_t = 0.004 \]
\[ c/d_t = \frac{0.003}{0.003 + 0.004} = 0.429 \]

(b) Minimum net tensile strain for flexural member

\[ \epsilon_t = 0.002 \]
\[ c/d_t = \frac{0.003}{0.003 + 0.002} = 0.600 \]

(c) Compression-controlled member
Control of Cracking by Distribution of Reinforcement
(Sec. 5.7.3.4 AASHTO 2012)

The spacing $s$ of mild steel reinforcement in the layer closest to the tension face shall satisfy the following:

$$s \leq \frac{700 \gamma_e}{\beta_s f_{ss}} - 2d_c$$

$\gamma_e =$ exposure factor

- 1.00 for Class 1 exposure condition
- 0.75 for Class 2 exposure condition

$d_c =$ thickness of concrete cover measured from extreme tension fiber to center of the flexural reinforcement located closest thereto (in.)

$\beta_s = 1 + \frac{d_c}{0.7(h - d_c)}$

$f_{ss} =$ tensile stress in steel reinforcement at the service limit state (ksi)

$h =$ overall thickness or depth of the component (in.)

Actual spacing of steel shall not be more than $s$ calculated above.

Assume $f_{ss} = f_y \times (M_{\text{SERVICE}}/M_{\text{STRENGTH}})$

Shrinkage & Temperature Reinforcement of Deck
(AASHTO 2012, Art 5.10.8)

Reinforcement for shrinkage and temperature stresses shall be provided near surfaces of concrete exposed to daily temperature changes and in structural mass concrete. Temperature and shrinkage reinforcement is provided to ensure that the total reinforcement on exposed surfaces is not less than that specified herein.

For bars or welded wire fabric, the area of reinforcement per foot, on each face and in each direction, shall satisfy:

$$A_s \geq \frac{1.30bh}{2(b+h)f_y}$$

(5.10.8-1)

$$0.11 \leq A_s \leq 0.60$$

(5.10.8-2)

$A_s =$ area of reinforcement in each direction and each face (in.$^2$/ft)

$b =$ least width of component section (in.)

$h =$ least thickness of component section (in.)

$f_y =$ specified yield strength of reinforcing bars

$\leq 75$ ksi
Where the least dimension varies along the length of wall, footing, or other component, multiple sections should be examined to represent the average condition at each section.

Spacing shall not exceed:
- 3.0 times the component thickness, or 18.0 in.
- 12.0 in. for walls and footings greater than 18.0 in. thick
- 12.0 in. for other components greater than 36.0 in. thick

For components 6.0 in. or less in thickness the minimum steel specified may be placed in a single layer. Shrinkage and temperature steel shall not be required for:
- End face of walls 18 in. or less in thickness.
- Side faces of buried footings 36 in. or less in thickness
- Faces of all other components, with smaller dimension less than or equal to 18.0 in.

Distribution Reinforcement of Deck
(AASHTO 2012, Art 5.10.8)

Reinforcement shall be placed in the secondary direction in the bottom of slabs as a percentage of the primary reinforcement for positive moment as follows.

For primary reinforcement parallel to traffic:
\[ \frac{100}{\sqrt{S_c}} \leq 50\% \]

For primary reinforcement perpendicular to traffic:
\[ \frac{220}{\sqrt{S_c}} \leq 67\% \]

where:
\[ S_c = \text{the effective span length of slab taken as equal to the effective length specified in Article 9.7.2.3 (ft)} = \text{clear distance between the girders.} \]
Fig. 32: Reinforcement detailing of Slab
b) DESIGN OF RAILING

Minimum height of rail post : 42 inch [Sec. 13.8.2]
Opening between rails shall be less than 6 inch for portion 27 inch vertically from walkway surface. Opening between rails shall be less than 8 inch for portion above 27 inch from walkway surface.

Each railing shall be designed for 50 lb/ft uniformly distributed load acting simultaneously in both vertical and horizontal direction.

Fig.33 : Side view and elevation view of railing and post

- Each railing shall be designed for 50 lb/ft uniformly distributed live load acting simultaneously in both vertical and horizontal direction.
- Opening between rails < 6 inch for portion 27 in. vertically from walkway surface.
- Opening between rails < 8 inch for portion above 27 in. from walkway surface.

Design Steps:

- Assume, 5in. x 5in. Railing
- Consider Live load on each railing = 50lb/ft
- Determine Dead load per unit length
- Determine total load \( w_T \) per unit length
- Determine Maximum Moment = \( \frac{1}{10} w_T l^2 \)
- Determine steel Area \( A_s \).
c) DESIGN OF CURB / SIDEWALK

Fig. 34: Loads on Curb or sidewalk

- Determine $P_1$, $P_2$, $P_3$, $P_4$.
- Determine bending moment $M$ at critical section
- Determine steel area, $A_{s1}$ due to $M$

General Load Combination

$1.25(DC) + 1.5(DW) + 1.75(LL)(1+IM/100)_{\text{Truck/Tandem}} + 1.75(LL)_{\text{Lane}} + 1.75(PL)$

$M = 1.25(M_{p1} + M_{p2}) + 1.75M_{LL}$, similarly for $F$ and $V$
Expansion gap Determination

Expansion gap is required to accommodate the thermal expansion-contraction. In Bangladesh seasonal temperature varies between 5 °C to 40 °C. For the purpose of design we take \( \Delta t = 40 \text{ °C} \).

Thermal expansion co-efficient of concrete \( \alpha_c = 0.00001 / \text{°C} \).

Therefore, maximum expansion/contraction shall be

\[ \Delta L = \alpha_c (\Delta t) L \]

where \( L \) is the length under consideration.

Total expansion may be divided at the two expansion gaps at the ends of the suspended span. Also we shall maintain a minimum of 1 inch gap in the event of extreme condition.

Thus, if \( L \) is the total span of the bridge and we confine the expansion/contraction only at the ends of suspended span then

\[ q = \alpha_c \Delta t \left( \frac{L}{2} \right) + 1 = 0.00001 \times 40 \times \frac{L}{2} \text{ (inch)} + 1 \text{ [rounded to higher } \frac{1}{2} \text{ inch]} \]
d) DESIGN OF INTERIOR GIRDER

![Diagram of a girder with labels for constant and variable depth, wearing surface, diaphragms, and flange dimensions.]

**Weight from constant depth part**
- \( w_{sw} \) = from concrete section
- \( w_{wc} \) = from wearing surface

**Diaphragms**
1. Width = 10” (intermediate), 18” (support)
2. Depth = 12” less than main girder
3. Spacing = 30ft max.

Fig. 35: Different dimensions of longitudinal girder

**Dead load Analysis of Interior Girder**

- Determine Dead load coming from self weight, wearing surface (DW).

- Determine self weight of cross girder/diaphram.
Fig. 36: Design sections of Interior Girder

**Table 2:** Determining concentrated load of cross girder/diaphragm on main girder

<table>
<thead>
<tr>
<th>Load of Diaphragm</th>
<th>Depth of Cross girder (in.)</th>
<th>Width of Girder, ( b_d ) (inch)</th>
<th>Load (lb)</th>
</tr>
</thead>
<tbody>
<tr>
<td>P1</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>P2</td>
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<td></td>
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<td>P3</td>
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<td>P4</td>
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<td>P5</td>
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<td></td>
</tr>
<tr>
<td>P6</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>
Fig. 37: SFD and BMD of interior girder due to DC dead load

Fig. 38: SFD and BMD of interior girder due to DW dead load (wearing course)

Live load analysis of Interior Girder
CE412: Structural Analysis & Design Sessional - II
Influence Line (IL)

- IL is a diagram showing the variation in shear, moment, reaction, stress in a structure due to a unit load moving across the structure.

- **Miller Breslay’s Principle**

  “The ordinates of IL for any stress element (such as axial force, shear force, bending moment or reaction) of any structure are proportional to those of the deflection curve which is obtained by removing the restrain corresponding to that element from structure & introducing in its place, a corresponding deformation into the primary structure which remains.”

Fig. 39: IL diagram for shear and moment at section 2
Fig. 40: IL diagram for shear and moment at section 4 and 7

LIVE LOAD MULTIPLIER

Truck wheel on one side may act directly on an interior girder. The other wheel shall be a distance apart from the girder. Thus full vehicle axle load may not act on one girder. This is considered using a distribution factor. (AASHTO Table 4.6.2.2.2b-1 and 4.6.2.2.3a-1)

INTERIOR GIRDER: Two or more lanes are loaded

Distribution factor for moment, \( \alpha_{i,m} = 0.075 + \left(\frac{S}{9.5}\right)^{0.6} \left(\frac{S}{L}\right)^{0.2} \left(\frac{Kg}{12.0 \cdot L \cdot t^3}\right)^{0.1} \)

Distribution factor for shear, \( \alpha_{i,v} = 0.2 + \frac{S}{12} - \left(\frac{S}{35}\right)^{2.0} \)
Fig. 41: Maximum positive moment at section 7 for forward and backward truck wheel load
Fig. 42: Maximum +/- moment, shear at section 7 for forward and backward truck wheel load
DESIGN TANDEM LOAD

APPLICATION OF DESIGN TANDEM LOAD

FOR MAXIMUM NEGATIVE

FOR MAXIMUM POSITIVE

Fig. 43: Maximum positive & negative moment, shear at section 7 for tandem load
EQUIVALENT LANE LOAD

Equivalent lane load must be used in addition to design wheel load to represent truck train.

FOR POSITIVE MAXIMUM

\[ M_7 \]

\[ 640 \text{ lb/ft} \]

FOR NEGATIVE MAXIMUM

\[ M_7 \]

\[ 640 \text{ lb/ft} \]

FOR MAXIMUM POSITIVE

\[ V_7 \]

\[ 640 \text{ lb/ft} \]

FOR MAXIMUM NEGATIVE

\[ V_7 \]

\[ 640 \text{ lb/ft} \]

\[ 640 \text{ lb/ft} \]

Fig. 44: Maximum positive, negative moment, shear at section 7 for equivalent lane load
Table 3:

### COMBINATION OF MOMENT: INTERIOR GIRDER

<table>
<thead>
<tr>
<th>Factor</th>
<th>Self weight Moment (DC)</th>
<th>Wearing Course Moment (DW)</th>
<th>Combined Positive Moment (1+IM100)</th>
<th>Combined Positive Moment (T[Truck])</th>
<th>Combined Positive Moment (T[Tandem])</th>
<th>Combined Positive Moment (T[Truck]+T[Tandem])</th>
<th>Combined Positive Moment (T[Truck]+T[Tandem]+1.25x+1.5b+1.75g+1.75f)</th>
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<tr>
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### COMBINATION OF SHEAR: INTERIOR GIRDER

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Flexural Reinforcement Design of Interior Girder

- Determine Effective width $b_{eff}$ for Interior Girder.
- Consider the Design moment for each section.
- Determine steel area $A_s$ for maximum design moment.
- Bar Cut-off will be done where required.

e) DESIGN OF EXTERIOR GIRDER

DESIGN OF EXTERIOR GIRDER

DEAD LOAD ANALYSIS

Loads from diaphragm $P_1, P_2$ etc. shall be halved. Trapezoidal load $w_v$ shall remain unchanged. Constant udl $w$ shall be recalculated.

Based on above loading, draw the SFD and BMD and determine values at 9 locations as before.

Fig. 45: Dead load on Exterior girder
DESIGN OF EXTERIOR GIRDER
LIVE LOAD ANALYSIS

Distribution factor for moment
\[ \alpha_{e,m} = e \alpha_{i,m} \]
\[ e = 0.77 + \frac{d_e}{9.1} \]
\[-1.0 \leq d_e \leq 5.5 \]

Distribution factor for shear
\[ \alpha_{e,v} = e \alpha_{i,v} \]
\[ e = 0.6 + \frac{d_e}{10} \]

Here \( d_e \) is in feet.

Now prepare the load combination tables for shear and moment. Dead load values shall be recalculated based on revised loading (DC and DW). Live load values may be directly copied from previous load combination tables and combinations may be performed with \( \alpha \) values for exterior girder.

Fig. 46: Live load on Exterior girder

Table 4:
### COMBINATION OF SHEAR: EXTERIOR GIRDER

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REINFORCEMENT DESIGN OF T-GIRDERS (AASHTO 2012 Section 5)

**Resistance factor $\phi$ [Sec. 5.5.4.2.1]**
- Moment: 0.90
- Shear: 0.90

**Positive Steel (T-section, bottom steel) [Sec. 5.7.3.2.1]**
\[
A_s \geq \frac{M_u}{\phi f_y \left( d - \frac{a}{2} \right)} \approx \frac{M_u}{\phi f_y (jd)}
\]
Assume $jd \approx 0.95d$

Check $A_s \geq A_{s,\text{min}} = \frac{200}{f_y} b_w d$

where, $f_y$ is in psi

Effective flange width, $b_e$ of T-Girder
- $b_e = $ Spacing of girders $= S$

Check, $a = \frac{A_s f_y}{0.85 f'_c b_e}$

Revise, $A_s = \frac{M_u}{\phi f_y \left( d - \frac{a}{2} \right)}$

Revise, $a = \frac{A_s f_y}{0.85 f'_c b_e}$

Determine $\beta_1$

Depth of neutral axis, $c = a / \beta_1$

Check $c < \frac{3}{8} d$ (tension controlled)

Finally, $\phi M_n = \phi A_s f_y \left( d - \frac{a}{2} \right) \geq M_u$

**Negative Steel (Rectangular Section, top steel)**

Design procedure same as before except that use beam web width $b_w$ instead of $b_e$.

REINFORCEMENT DESIGN OF T-GIRDERS (AASHTO 2012 Section 5)

**Design for Shear (Sec. 5.8.3.3)**

Shear reinforcement required when $V_u > 0.5 \phi V_c$ ($V_u$ and $V_c$ are in kip, Sec. 5.8.2.4)

If $V_u > 0.25 \phi f'_c b_w d$ then section has to be revised. ($f'_c$ in ksi, $b_w$ and $d$ are in inch)

Nominal shear resistance, $V_n = V_c + V_s$

where $V_c = 0.0316 \beta (\sqrt{f'_c}) b_w d$, where $\beta = 2.0$ (Eq. 5.8.3.3-3)

Stirrup spacing, $s = \frac{\phi A_v f_y d}{V_u - \phi V_c}$

Minimum transverse reinforcement (Eq. 5.8.2.5-1), $A_v \geq 0.0316 \sqrt{f'_c} b_w s f_y$

where, $A_v$ in in$^2$, $f'_c$ in ksi, $b_w$ is beam web width in inch, $s$ is stirrup spacing in inch, $f_y$ in ksi.

Shear stress in concrete $\nu_u = V_u / (\phi b_w d)$

**Maximum stirrup spacing [Sec 5.8.2.7]:**

$s_{\text{max}} = 0.8d \leq 24''$ when $\nu_u < 0.125 f'_c$

$s_{\text{max}} = 0.4d \leq 12''$ when $\nu_u \geq 0.125 f'_c$
SKIN REINFORCEMENT [Sec. 5.7.3.4]

If $d_e$ of non-prestressed or partially prestressed concrete members exceeds 3.0 ft, longitudinal skin reinforcement shall be uniformly distributed along both side faces of the component for a distance $d_e/2$ (inch) nearest the flexural tension reinforcement.

The area of skin reinforcement $A_{sk}$ in in$^2$/ft of height on each side face shall satisfy (Eq. 5.7.3.4-2):

$$A_{sk} \geq 0.012(d_l - 30) \leq \frac{A_s}{4}$$

However, the total area of longitudinal skin reinforcement (per face) need not exceed one-fourth of the required flexural tensile reinforcement $A_s$.

The maximum spacing of the skin reinforcement shall not exceed either $d_e/6$ or 12.0 in.

REINFORCEMENT DETAILING OF T-GIRDERS

**Haunch Steel**
Provide #3 or #4 bar @ 6” ~ 9” c/c along the length of the girder

**Link/Tie for skin reinforcement**
Provide #3 or #4 bars. Vertical and longitudinal spacing may not exceed 24”.

Fig.47: Reinforcement detailing of main girder
f) DESIGN OF CROSS GIRDER/ DIAPHRAM

NON LOAD BEARING.
FOR STABILITY OF MAIN GIRDERS.

Deck Slab

Stirrup: Provide #4 bars @ max. 18” on centers.
Ties: Provide #3 bars @ max. 24” on centre.

Skin reinforcement: Provide $A_s = 0.002bt$

Longitudinal steel: Provide $\rho = 200/f_y$ at top and bottom.

Fig.48 : Reinforcement detailing of cross girder
g) DESIGN OF ARTICULATION

Fig.49 : Articulation or halving joint
Fig.50: Widening of girder near articulation location

What is Articulation

- The connection between the suspended span and the edge of the cantilever is called ‘Articulation’.
- The bearings at articulations can be in the form of sliding plates, roller-rocker arrangement or elastomeric pads.
Fig. 51: Cracks at articulation

Fig. 52: Clearance requirement around bearing pad near articulation
$q = \text{Expansion gap} = 3''$

$p = \text{edge distance} \geq 7''$

3.5'' approx. considering standard clear cover and usual bar sizes used.

Fig. 53: Expansion gap and edge distance around bearing pad
DESIGN OF ARTICULATION

$V_A$ is increased by 2% to account for excess weight due to widening.

$F_A = 0.5V_{4,LL} \geq 0.2V_A$

$d_A =$ effective depth at sec. A

Width of pad, $b_b \geq 8''$

Therefore, $g = 2p+q+b_b$, $a_v = p+q+b_b/2$

**Bearing Criterion**

(AASHTO 2012 Sec. 5.7.5)

Bearing stress on the bearing area shall not exceed the concrete bearing strength, $f_b$.

Where, $f_b = 0.85\phi f_c'$

Therefore, $A_B f_b = V_A \Rightarrow b_A \geq \frac{V_A}{b_b f_b} + 2p$

---

DESIGN OF ARTICULATION

**Shear Friction Criterion**

(AASHTO 2012 Sec. 5.13.2.4.2)

Shear friction adequacy shall be checked at Section-A. When $a_v/d_A < 1.0$, $b_A$ must be adequate to resist $V_A$ ($d_A$ fixed).

Shear friction resistance of concrete on the plane of Sec-A is given by

$V_A \leq \phi V_n = \phi f_c' b_A d_A$

And $V_A \leq \phi V_n = \phi 0.8 b_A d_A$

Therefore,

$b_A \geq \frac{V_A}{0.2\phi f_c' d_A}$

or, $b_A \geq \frac{V_A}{\phi 0.8d_A}$

Final value of $b_A$ shall be the larger of the value obtained from bearing and shear friction criteria.
Design of Articulation

\[ M_A = V_A a_v + F_A \times d_A / 2 \]

- Flexural steel \( A_{s1} \) based on moment \( M_A \) for cracks 1, 3 & 4
- \( A_{sh} \) due to direct tension of \( V_A \) for crack 2
- \( A_{s2} \) due to \( F_A \) for cracks 1, 3 & 4

\( a_v \) and \( d_A \) are shown in the diagram.

\( A_{sh} \) may be provided by extending and bending the main top bar as required. Similarly, \( A_{s1} \) and \( A_{s2} \) may be provided by extending and bending the girder bottom steel.

Plan section
Flexural steel $A_{s1}$ based on moment $M_A$ for cracks 1, 3 & 4

$A_{sh}$ due to direct tension of $V_A$ for crack 2

$A_{s2}$ due to $F_A$ for cracks 1, 3 & 4 and $A_{vf}$ from shear friction criterion

For flexural steel $A_{s1}$

$$A_{s1} = \frac{M_A}{\varphi f_y (d_A-a/2)}$$

$$a = \frac{A_{s1} f_y}{0.85 f'_c b_A}, \quad \varphi = 0.9$$

For steel $A_{sh}$ and $A_{s2}$

$$A_{sh} = \frac{V_A}{\varphi f_y}, \quad \varphi = 0.85$$

$$A_{s2} = \frac{F_A}{\varphi f_y}$$
**DESIGN OF ARTICULATION**

Main girder steel

$A_{s2}$ due to $F_A$ for cracks 1, 3 & 4 and $A_{vf}$ from shear friction criterion

Stirrup based on $V_A$

---

**SHEAR FRICTION REINFORCEMENT [Sec 5.8.4]**

Shear friction criterion: $a_v/d_A < 1.0$

$V_A = \varphi \{ c A_{cv} + \mu A_{vf} f_y \}, V_A$ in lb [Eq. 5.8.4.1-3]

$\varphi = 0.9$ for shear, $c = 0.0$ psi [Sec.5.8.4.3]

$A_{cv} = b_A \times d_A = \text{shear area (in}^2\text{)}$

$\mu = 1.4 = \text{friction factor, } f_y \text{ in psi}$

$A_{vf} = \text{shear steel crossing the shear plane, in}^2$

$A_{vf} \geq 0.05A_{cv}/f_y$ [Eq. 5.8.4.4-1]

$A_{vf}$ may be merged with $A_{s2}$ and $A_{s1}$

---

**Conventional flexural shear criterion: $a_v/d_A > 1.0$**

$s = \frac{\varphi A_v f_v d_A}{V_A - \varphi V_c}$

$V_c = 0.0316 \beta (\sqrt{f_c'}) b_A d_A$

Check $s_{\text{max}} < 12''$ or $d_A/2$

Though conventional flexural shear steel is required only when $a_v/d_A > 1$, we shall, nevertheless, provide such steel even when $a_v/d_A < 1$. 
Design Steps

1. Determine flexural steel area $A_{s1}$ based on moment $M_A$.
2. Determine steel area $A_{s2}$ based on $F_A$.
3. Determine steel area $A_{sh}$ based on $V_A$
4. Determine required spacing $s$ for stirrup
5. Check spacing of stirrup with maximum spacing

Detailing of Articulation

![Fig.54 : Reinforcement detailing of articulation](image)
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3. AASHTO LRFD Bridge design Specifications, 6th edition, 2012, US.

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