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Existing Buildings 01
The Basic Concepts

Date: 26 Feb 2022
Session 38

A.S. Moghadam
International Institute of Earthquake Engineering and Seismology (IIEES)
and
S-04 Team Leader, JV of NKY-Protek-Sheltech

S-8A SEng PRP
Training Program

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1

Course Title: Existing Buildings
Track 1
The Basic Concepts

- **Course Topics:**

1. Why seismic design codes cannot be used for assessment of existing buildings?
2. Performance objectives for structural and nonstructural elements
3. Force-controlled and deformation-controlled parameters
4. Primary and secondary elements
5. Acceptance criteria
6. Introducing the general flow-chart of assessment of existing buildings

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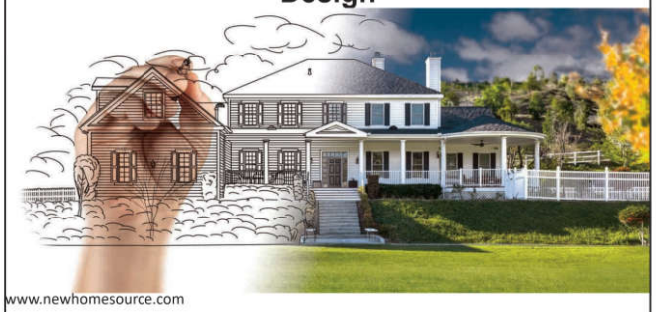
Building Design



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structural elements, non-structural elements

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কর্তৃপক্ষ কর্তৃক প্রকাশিত

বৃহস্পতিবার ফেব্রুয়ারি ১৩, ২০২১

Government of the People's Republic of Bangladesh
Ministry of Housing and Public Works
Notification
Date : 05-11-1426/18-02-2020

S.R.O. No.55-Law/2020. In exercise of the powers conferred under section 18A of the Building Construction Act 1952 (Act No. II of 1953) the Government is pleased to make the following Code by recalling the Bangladesh National Building Code, 2006, namely :—

PART I
Chapter 1
Title, Purpose, Scope, Etc

1. Title and commencement.—(1) This Code may be called the Bangladesh National Building Code (BNBC)2020.

(2) It shall come into force at once.

2. Purpose.—(1) The purpose of this Code is to establish minimum standards for design, construction, quality of materials, use and occupancy, location and maintenance of all buildings within Bangladesh in order to safeguard, within achievable limits, life,

বাংলাদেশ গেজেট, অতিরিক্ত, ফেব্রুয়ারি ১৩, ২০২১ ৩০৯৩

PART VI
Chapter 2
Loads on Buildings and Structures

2.1 Introduction

2.1.1 Scope

This Chapter specifies the minimum design forces including dead load, live load, wind and earthquake loads, miscellaneous loads and their various combinations. These loads shall be applicable for the design of buildings and structures in conformance with the general design requirements provided in Chapter 1.

2.7 Combinations of Loads

2.7.1 General

Buildings, foundations and structural members shall be investigated for adequate strength to resist the most unfavorable effect resulting from the various combinations of loads provided in this Section. The combination of loads may be selected using the provisions of either Sec 2.7.2 or Sec 2.7.3 whichever is applicable. However, once Sec 2.7.2 or Sec 2.7.3 is selected for a particular construction material, it must be

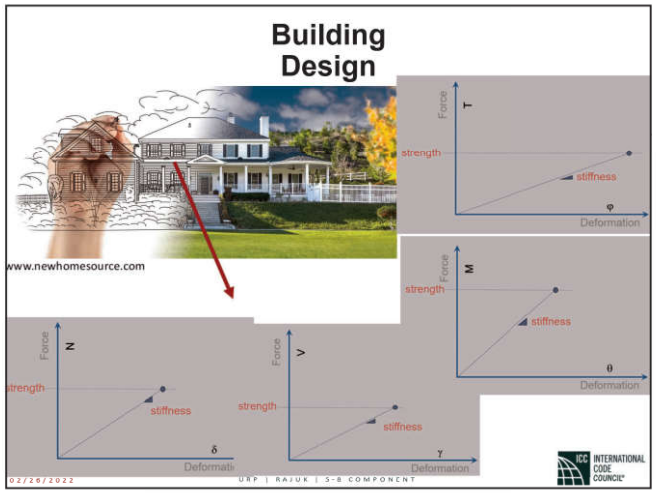
2.7.2 Combinations of Load effects for Allowable Stress/Strength Design Method

2.7.2.1 Basic combinations

Provisions of this Section shall apply to al construction materials permitting their use in proportioning structural members by allowable stress/strength design method. When this method is used in designing structural members, all loads listed herein shall be considered to act in the following combinations. The combination that produces the most unfavorable effect shall be used in design.

1. $D + F$
2. $D + H + F + L + T$
3. $D + H + F + (L_r \text{ or } R)$
4. $D + H + F + 0.75(L + T) + (L_r \text{ or } R)$
5. $D + H + F + (W \text{ or } 0.7E)$
6. $D + H + F + 0.75(W \text{ or } 0.7E) + 0.75L + 0.75(L_r \text{ or } R)$
7. $0.6D + W + H$
8. $0.6D + 0.7E + H$

4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Force
T
strength
stiffness
φ
Deformation
M
θ
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δ
V
γ
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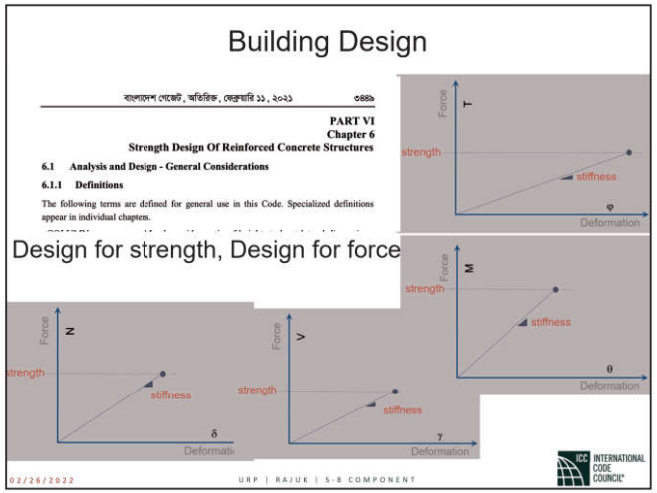
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৩৪৪৯
PART VI
Chapter 6
Strength Design Of Reinforced Concrete Structures
6.1 Analysis and Design - General Considerations
6.1.1 Definitions
The following terms are defined for general use in this Code. Specialized definitions appear in individual chaptens.
Design for strength, Design for force
Force
T
strength
stiffness
φ
Deformation
M
θ
N
δ
V
γ
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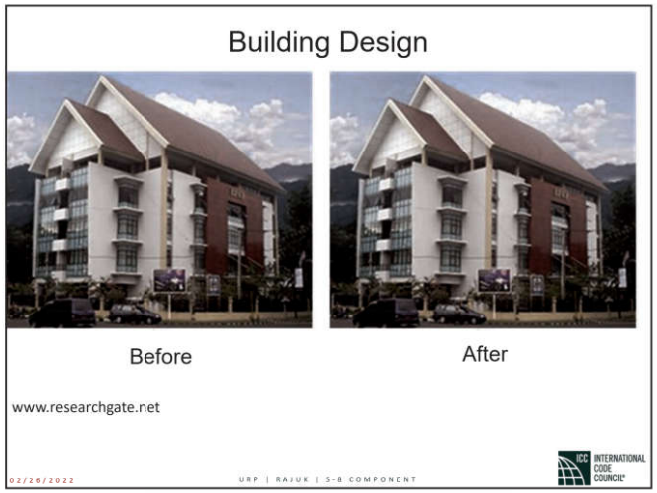
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Building Seismic Design
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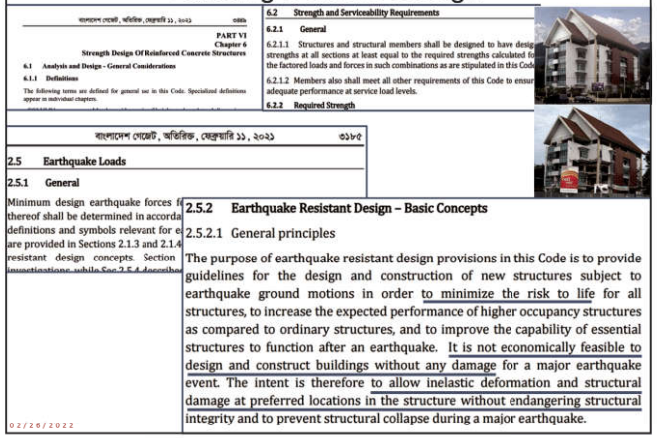
Building Seismic Design



বাংলাদেশ গেজেট, অতিরিক্ত, ফেব্রুয়ারি ১১, ২০২১ ৩১৮৫

PART VI
Chapter 6
Strength Design Of Reinforced Concrete Structures

6.1 Analysis and Design - General Considerations

6.1.1 Definitions

The following terms are defined for general use in this Code. Specialized definitions appear in individual chapters.

6.2 Strength and Serviceability Requirements

6.2.1 General

6.2.1.1 Structures and structural members shall be designed to have desig strengths at all sections at least equal to the required strengths calculated fo the factored loads and forces in such combinations as are stipulated in this Code

6.2.1.2 Members also shall meet all other requirements of this Code to ensur adequate performance at service load levels.

6.2.2 Required Strength

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2.5 Earthquake Loads

2.5.1 General

Minimum design earthquake forces f thereof shall be determined in accorda definitions and symbols relevant for e are provided in Sections 2.1.3 and 2.1.4 resistant design concepts. Section investigations while Sec 2.5.4 describes

2.5.2 Earthquake Resistant Design – Basic Concepts

2.5.2.1 General principles

The purpose of earthquake resistant design provisions in this Code is to provide guidelines for the design and construction of new structures subject to earthquake ground motions in order to minimize the risk to life for all structures, to increase the expected performance of higher occupancy structures as compared to ordinary structures, and to improve the capability of essential structures to function after an earthquake. It is not economically feasible to design and construct buildings without any damage for a major earthquake event. The intent is therefore to allow inelastic deformation and structural damage at preferred locations in the structure without endangering structural integrity and to prevent structural collapse during a major earthquake.

02/26/2022

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Building Seismic Design

Seismic Assessment of a 15-Story Building Damaged in the Chile Earthquake of February 27th 2010

M. Kohrangi
IUSS School, IUSS Pavia, Pavia, Italy

T.J. Sullivan
University of Pavia, Pavia, Italy

G.M. Calvi
IUSS Pavia, Pavia, Italy

15 WCEE
LISBOA 2012

2.5.2 Earthquake Resistant Design – Basic Concepts

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Building Seismic Design

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Building Seismic Design

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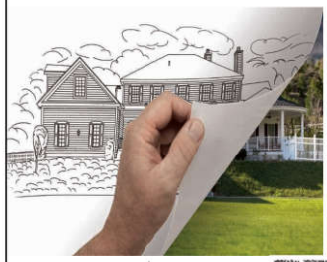
2.5.2 Earthquake Resistant Design – Basic Concepts

2.5.2.1 General principles

The purpose of earthquake resistant design provisions in this Code is to provide guidelines for the design and construction of new structures subject to earthquake ground motions in order to minimize the risk to life for all structures, to increase the expected performance of higher occupancy structures as compared to ordinary structures, and to improve the capability of essential structures to function after an earthquake. It is not economically feasible to design and construct buildings without any damage for a major earthquake event. The intent is therefore to allow inelastic deformation and structural damage at preferred locations in the structure without endangering structural integrity and to prevent structural collapse during a major earthquake.

13

Can we use the seismic design approach of NBCC for assessment of an existing building?



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Existing Building

14

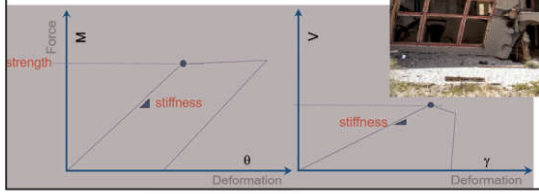
NBCC Seismic design for assessment of a building?

2.5.2 Earthquake Resistant Design – Basic Concepts

2.5.2.1 General principles

The purpose of earthquake resistant design provisions in this Code is to provide guidelines for the design and construction of new structures subject to earthquake ground motions in order to minimize the risk to life for all structures, to increase the expected performance of higher occupancy structures as compared to ordinary structures, and to improve the capability of essential structures to function after an earthquake. It is not economically feasible to design and construct buildings without any damage for a major earthquake event. The intent is therefore to allow inelastic deformation and structural damage at preferred locations in the structure without endangering structural integrity and to prevent structural collapse during a major earthquake.

How NBCC performs Seismic design for a new building?



15

Seismic **Design** Base Shear

BNBC 2020

$$V = S_a W$$

$$S_a = \frac{2}{3}\frac{ZI}{R}C_s$$

$S_a =$ Design spectral acceleration (in units of g)

$\beta =$ Coefficient used to calculate lower bound for S_a

$Z =$ Seismic zone coefficient

$I =$ Structure importance factor

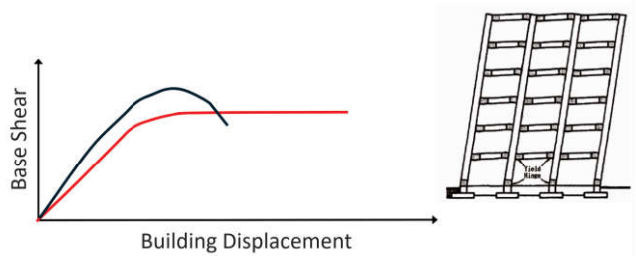
$R =$ Response reduction factor which depends on structural system

$C_s =$ Normalized acceleration response spectrum

16

Design based on **Ductility**

$R =$ Response reduction factor which depends on structural system



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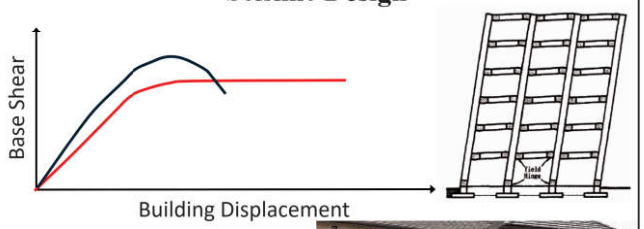
Seismic **Design**



- Ductility
- Damage
- Performance Level

$R =$ Response reduction factor which depends on structural system

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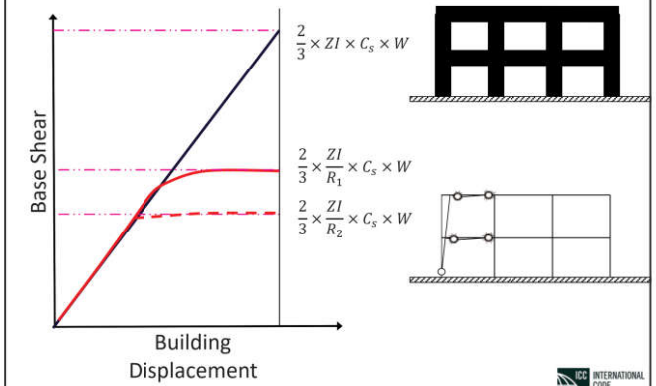
Relationship between **R-values** and **Damage**



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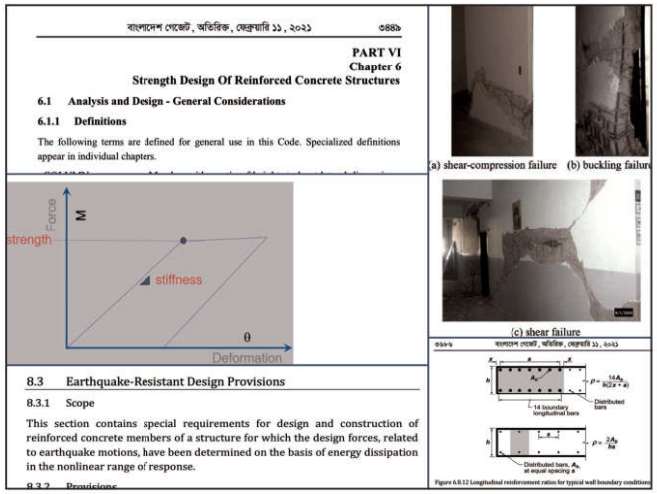

PART VI
Chapter 6
Strength Design Of Reinforced Concrete Structures

6.1 Analysis and Design - General Considerations

6.1.1 Definitions

The following terms are defined for general use in this Code. Specialized definitions appear in individual chapters.

8.3 Earthquake-Resistant Design Provisions

8.3.1 Scope

This section contains special requirements for design and construction of reinforced concrete members of a structure for which the design forces, related to earthquake motions, have been determined on the basis of energy dissipation in the nonlinear range of response.

(a) shear-compression failure (b) buckling failure

(c) shear failure

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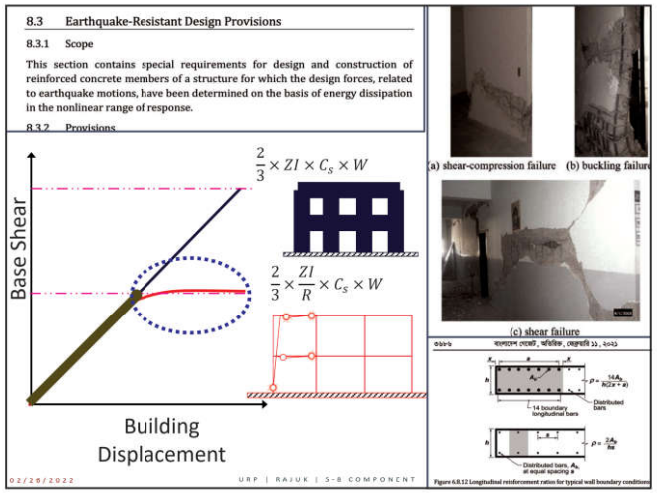

21

Design Base Shear verses **Assessment** Base Shear

BNBC 2020

$$V = S_a W$$

$$S_a = \frac{2}{3}\frac{ZI}{R}C_s$$

ASCE 41-17

$$V = C_1 C_2 C_m S_a W$$

$S_a =$ Design spectral acceleration (in units of g)

$\beta =$ Coefficient used to calculate lower bound for S_a

$Z =$ Seismic zone coefficient

$I =$ Structure importance factor

$R =$ Response reduction factor which depends on structural system

$C_s =$ Normalized acceleration response spectrum

22

To perform **Seismic Evaluation** and **Assessment** of an **Existing Building**

- Fragility Curves and Vulnerability functions
- Visual Screening

- Rapid Vulnerability Assessment (RVA)
- Preliminary Engineering Assessment (PEA)
- Detailed Engineering Assessment (DEA)

ASCE 41-17

- TIER 1 Screening
- TIER 2 Deficiency-Based Evaluation and Retrofit
- TIER 3 **Systematic** Evaluation and Retrofit

And many more methods …

23

Earthquake considerations for Buildings

- New Buildings → **BNBC**
- Existing Buildings
- Post earthquake Damaged Buildings

Seismic Vulnerability Assessment

24

To perform **Seismic Evaluation** and **Assessment** of an **Existing Building**

- Detailed Engineering Assessment (DEA)

ASCE 41-17

- TIER 3 **Systematic** Evaluation and Retrofit

Different Performance levels
For
Different seismic hazard levels

Performance Objective

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Performance Objective

Probability of Exceedance	Mean Return Period (years)
50%/30 years	43
50%/50 years	72
20%/50 years	225
10%/50 years	475
5%/50 years	975
2%/50 years	2,475

Earthquake Hazard Level	Target Building Performance Levels: Operational Level (1-A)	Immediate Occupancy Performance Level (1-B)	Life Safety Performance Level (3-C)	Collapse Prevention Performance Level (5-E)
50%/50 year	a	b		d
20%/50 year	e	f		h
BSE-1 (~10%/50 year)				l
BSE-2 (~2%/50 year)	m	n	o	p

Life Safety Level (3-C): Moderate

Some residual strength and stiffness left in all stories. Gravity-load-bearing elements function. No out-of-plane failure of walls. Some permanent drift. Damage to partitions. Continued occupancy might not be likely before repair. Building might not be economical to repair.

Falling hazards, such as parapets, mitigated, but many architectural, mechanical, and electrical systems are damaged.

Somewhat more damage and slightly higher life-safety risk.

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Table C2-2. Performance Objectives

Seismic Hazard Level	Target Building Performance Levels: Operational Performance Level (1-A)	Immediate Occupancy Performance Level (1-B)	Life Safety Performance Level (3-C)	Collapse Prevention Performance Level (5-D)
50%/50 years	a	b	c	d
BSE-1E(20%/50 years)	e	f	g	h
BSE-2E(5%/50 years)	i	j	k	l
BSE-2 N(ASCE 7 MCE_R)	m	n	o	p

Nonstructural Performance Levels	Structural Performance Levels: Immediate Occupancy (S-1)	Damage Control (S-2)	Life Safety (S-3)	Limited Safety (S-4)	Collapse Prevention (S-5)	Not Considered (S-6)
Operational (N-A)	Operational 1-A	2-A	NR[a]	NR[a]	NR[a]	NR[a]
Position Retention (N-B)	Immediate Occupancy 1-B	2-B	3-B	4-B	NR[a]	NR[a]
Life Safety (N-C)	1-C	2-C	Life Safety 3-C	4-C	5-C	6-C
Hazards Reduced (N-D)	NR[a]	NR[a]	3-D	4-D	Collapse Prevention 5-D	6-D
Not Considered (N-E)	NR[a]	NR[a]	3-E	4-E	5-E	No evaluation or retrofit

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Nonstructural Performance Levels	Structural Performance Levels: Immediate Occupancy (S-1)	Damage Control (S-2)	Life Safety (S-3)	Limited Safety (S-4)	Collapse Prevention (S-5)	Not Considered (S-6)
Operational (N-A)	Operational 1-A	2-A	NR[a]	NR[a]	NR[a]	NR[a]
Position Retention (N-B)	Immediate Occupancy 1-B	2-B	3-B	4-B	NR[a]	NR[a]
Life Safety (N-C)	1-C	2-C	Life Safety 3-C	4-C	5-C	6-C
Hazards Reduced (N-D)	NR[a]	NR[a]	3-D	4-D	Collapse Prevention 5-D	6-D
Not Considered (N-E)	NR[a]	NR[a]	3-E	4-E	5-E	No evaluation or retrofit

Seismic-Force-Resisting System	Type	Structural Performance Levels: Collapse Prevention (S-5)	Life Safety (S-3)	Immediate Occupancy (S-1)
Concrete frames	Primary elements	Extensive cracking and hinge formation in ductile elements. Limited cracking or splice failure in some nonductile columns. Severe damage in short columns.	Extensive damage to beams. Spalling of cover and shear cracking in ductile columns. Minor spalling in nonductile columns. Joint cracks.	Minor cracking. Limited yielding possible at a few locations. Minor spalling of concrete cover.
	Secondary elements	Extensive spalling in columns and beams. Limited column shortening. Severe joint damage. Some reinforcing buckled.	Major cracking and hinge formation in ductile elements. Limited cracking or splice failure in some nonductile columns. Severe damage in short columns.	Minor spalling in a few places in ductile columns and beams. Flexural cracking in beams and columns. Shear cracking in joints.
	Drift	Transient drift sufficient to cause extensive nonstructural damage. Extensive permanent drift.	Transient drift sufficient to cause nonstructural damage. Noticeable permanent drift.	Transient drift that causes minor or no nonstructural damage. Negligible permanent drift.

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Nonstructural Performance Levels	Structural Performance Levels					
	Immediate Occupancy (S-1)	Damage Control (S-2)	Life Safety (S-3)	Limited Safety (S-4)	Collapse Prevention (S-5)	Not Considered (S-6)
Operational (N-A)	Operational 1-A	2-A	NR[a]	NR[a]	NR[a]	NR[a]
Position Retention (N-B)	Immediate Occupancy 1-B	2-B	3-B	4-B	NR[a]	NR[a]
Life Safety (N-C)	1-C	2-C	Life Safety 3-C	4-C	5-C	6-C
Hazards Reduced (N-D)	NR[a]	NR[a]	3-D	4-D	Collapse Prevention 5-D	6-D
Not Considered (N-E)	NR[a]	NR[a]	3-E	4-E	5-E	No evaluation or retrofit

Component Group	Nonstructural Performance Levels		
	Life Safety (N-C)	Position Retention (N-B)	Operational (N-A)
Cladding Panels	Distorton in connections and damage to cladding components, including loss of weather-tightness and security. Overhead panels do not fall.	Distortion in connections and damage to cladding components, including loss of weather-tightness and security. Overhead panels do not fall.	Negligible damage to panels and connections. No loss of function or weather-tightness.
Glazing	Some cracked panes; none broken. Limited loss of weather-tightness.	Some cracked panes; none broken. Limited loss of weather-tightness.	No cracked or broken panes. No loss of function or weather-tightness.
Heavy partitions (masonry and hollow clay tile or stud walls with tile or masonry veneer)	Distributed damage; cracking, crushing, and dislodging of veneer or parge coat in some areas. Damage to adjacent ceiling, but no wall failure.	Distributed damage; cracking, crushing, and dislodging of veneer or parge coat in some areas.	Minor crushing and cracking at corners. Limited dislodging of veneer or parge coat.
Light partitions (plaster and gypsum)	Distributed damage; some severe cracking of sheathing and racking in some areas.	Cracking at openings. Minor cracking of sheathing.	Minor cracking.
Ceilings	Extensive damage to suspended acoustical ceilings and grids. Plaster ceilings cracked and spalled but do not drop as a unit. Tiles in grid ceilings dislodged and falling; grids distorted and pulled apart. Plaster and gypsum board ceilings cracked and spalled but did not drop as a unit.	Limited damage. Plaster ceilings cracked and spalled but did not drop as a unit. Suspended ceiling grids largely undamaged, though individual tiles falling.	Generally negligible damage with no impact on reoccupancy or functionality.
Parapets and ornamentation	Minor damage; some falling of unreinforced elements in unoccupied areas.	Minor damage.	Negligible damage.
Canopies and marquees	Some damage to the elements, but essentially in place.	Some damage to the elements, but essentially in place.	Minor damage to the elements.
Chimneys and stacks	Minor damage. No collapse.	Minor damage. No collapse.	Negligible damage.

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To perform **Seismic Evaluation** and **Assessment** of an **Existing Building**

- Detailed Engineering Assessment (DEA)

ASCE 41-17

- TIER 3 **Systematic** Evaluation and Retrofit

Simple or Complex?

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Components that affect the lateral stiffness or distribution of forces in a structure, or are loaded as a result of lateral deformation of the structure are classified as:

Primary or **Secondary Components**

A structural component that is required to resist seismic forces and accommodate deformations for the structure to achieve the selected Performance Level shall be classified as primary.

A structural component that accommodates seismic deformations and is not required to resist seismic forces for the structure to achieve the selected Performance Level shall be permitted to be classified as secondary.

Table 10-10b. Numerical Acceptance Criteria for Linear Procedures—Reinforced Concrete Circular Columns with Spiral Reinforcement or Seismic Hoops as Defined in ACI 318

			m-Factors[a]				
			Performance Level				
			Component Type				
			Primary			Secondary	
$\left(\frac{N_{UD}}{A_g f_{cE}}\right)$	ρ_t	V_{yE}/V_{ColOE}	IO	LS	CP	LS	CP
Columns not controlled by inadequate development or splicing along the clear height[b]							

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Difference between response parameters (**Deformation-Controlled** actions and **force-controlled** actions) and difference in estimation of their capacity and demand

Primary component actions exhibiting this behavior shall be classified as deformation controlled if the plastic range is such that $d \geq 2g$; otherwise, they shall be classified as force controlled. Secondary component actions exhibiting this behavior shall be classified as deformation controlled for any d/g ratio.

Primary component actions exhibiting this behavior shall be classified as deformation controlled if the plastic range is such that $e \geq 2g$; otherwise, they shall be classified as force controlled. Secondary component actions exhibiting this behavior shall be classified as deformation controlled if $f \geq 2g$; otherwise, they shall be classified as force controlled.

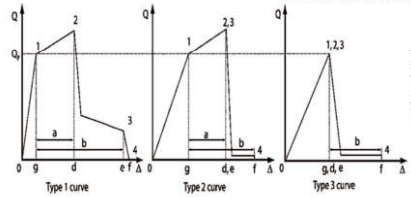

Figure 7-4. Component Force Versus Deformation Curves

Notes:

1. Only secondary component actons permitted between points 2 and 4.
2. The force, *Q*, after point 3 diminishes to approximately zero.

Primary component actions exhibiting this behavior shall be classified as force controlled. Secondary component actions exhibiting this behavior shall be classified as deformation controlled if $f \geq 2g$; otherwise, they shal be classified as force controlled.

For nonlinear procedures, force-controlled components defined in Chapters 8 through 12 may be reclassified as Type 3 deformation-controlled components, provided the following criteria are met:

1. The component action being reclassified exhibits the Type 3 deformation-controlled performance defined in this section;
2. The gravity-load-resisting load path is not altered, or if it is altered, an alternate load path is provided to ensure that local stability is maintained in accordance with the load combinations of Section 7.2.2 at the anticipated maximum displacements predicted by the analysis;
3. The total gravity load supported by all components that are reclassified from force controlled to deformation controlled does not exceed 5% of the total gravity load being supported at that story; and
4. All remaining deformation-controlled components meet the acceptance criteria to achieve the target performance level and all remaining force-controlled components are not overstressed.

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Difference between response parameters
(**Deformation-Controlled** actions and **force-controlled** actions)
and difference in estimation of their **capacity** and demand

Q_{CE} = Expected strength of component deformation-controlled action

Q_{CL} = Lower-bound strength of a force-controlled action

7.5.1.4 Material Properties. Expected material properties shall be based on mean values of tested material properties. Lower-bound material properties shall be based on mean values of tested material properties minus one standard deviation, σ.

Nominal material properties, or properties specified in construction documents, shall be taken as lower-bound material properties unless otherwise specified…

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Difference between **Deformation-Controlled** based on their **Demand**

Table 10-6. Component Ductility Demand Classification

Maximum Value of DCR or Displacement Ductility	Descriptor
<2	Low ductility demand
2 to 4	Moderate ductility demand
>4	High ductility demand

Example:

columns, lap-spliced transverse reinforcement shall be assumed to be not more than 50% effective in regions of moderate ductility demand and ineffective in regions of high ductility demand, and

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Difference between **Force-Controlled** based on their **Criticality**

Table 7-8. Load Factor for Force-Controlled Behaviors

Action Type	γ
Critical	1.3
Ordinary	1.0
Noncritical	1.0

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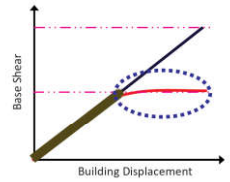
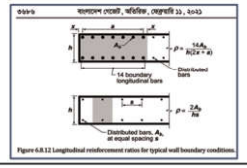
Summary 1/5

- The main two categories of Design requirements of a building in earthquake prone regions.



- Seismic design codes cannot be used for assessment of existing buildings.



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Summary 2/5

- The main characteristic of "Systematic" Evaluation.

Probability of Exceedance	Mean Return Period (years)
50%/30 years	43
50%/50 years	72
20%/50 years	225
10%/50 years	475
5%/50 years	975
2%/50 years	2,475

Earthquake Hazard Level	Target Building Performance Levels: Operational Performance Level (1-A)	Immediate Occupancy Performance Level (1-B)	Life Safety Performance Level (3-C)	Collapse Prevention Performance Level (5-E)
50%/50 year	a	b		d
20%/50 year	e	f		h
BSE-1 (~10%/50 year)				i
BSE-2 (~2%/50 year)	m	n	o	p

Life Safety
Level (3-C)

Structure

Some residual strength and stiffness left in all stories. Gravity-load-bearing elements function. No out-of-plane failure of walls. Some permanent drift. Damage to partitions. Continued occupancy might not be likely before repair. Building might not be economical to repair.

Falling hazards, such as parapets mitigated, but many architectural, mechanical, and electrical systems are damaged.

Comparing more damage and slightly higher life safety risk.

Different Performance levels For Different seismic hazard levels

- "Primary Component".

A structural component that is required to resist seismic forces and accommodate deformations

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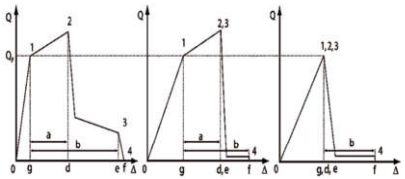
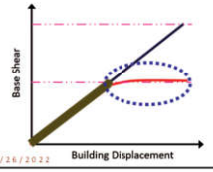
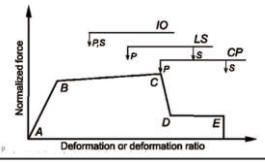
Summary 3/5

- "Deformation-Controlled actions" and "force-controlled actions".



- Satisfy a performance level such as "Life Safety (LS)".





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Summary 4/5

- Factor in the Seismic Design Base Shear formula of BNBC-2020 that is not applicable to existing buildings.

$$V = S_a W \qquad S_a = \frac{2}{3}\frac{ZI}{R}C_s$$

- Calculations of capacities for "Deformation-Controlled actions" and "force-controlled actions".

Q_{CE} = Expected strength of component deformation-controlled action

Q_{CL} = Lower-bound strength of a force-controlled action

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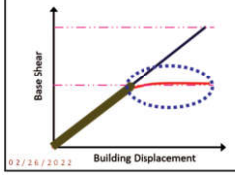
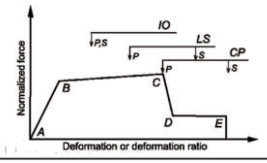
Summary 5/5

- Systematic Assessment are often complicated and complex.

$$V = S_a W \qquad S_a = \frac{2}{3}\frac{ZI}{R}C_s$$

- Meaning of the "Performance Level" of a structural member.





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Course Title: Existing Buildings
Track 1
The Basic Concepts

- **Course Outcomes:**

1. Understand the main concepts in systematic retrofitting of the existing buildings.
2. Have a clear view on differences between seismic design of a new building and seismic assessment of an existing building.
3. Differentiate between different response parameters and the way they are treated in estimation of capacity and demand.
4. Relate the concept of performance of a building to accept levels of damages.
5. Understand why modelling is the main issue in systematic assessment, and how it is different with ordinary design procedures.

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Questions?
Thank you

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THE WORLD BANK

Existing Buildings 02
Linear Analyses Procedures

Date: 27 Feb 2022
Session 39

A.S. Moghadam
International Institute of Earthquake Engineering and Seismology (IIEES)
and
S-04 Team Leader, JV of NKY-Protek-Sheltech

S-8A SEng PRP Training Program

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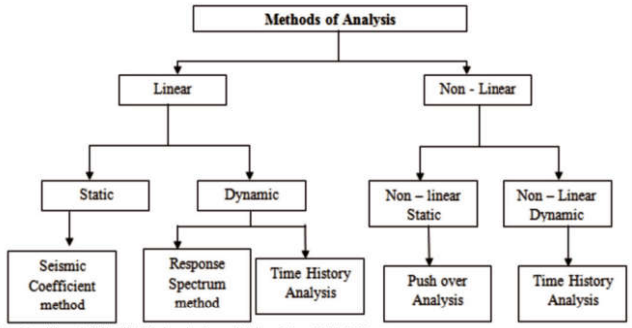

nternational Journal of Scientific & Engineering Research Volume 8, Issue 7, July-2017

2

The Applicability of the Linear Procedures

If the building does not have any of these:

1. In-Plane Discontinuity Irregularity
2. Out-of-Plane Discontinuity Irregularity
3. Weak Story Irregularity*
4. Torsional Strength Irregularity**

If either * or ** exists and
If a component DCR exceeds the lesser of 3.0 and the m-factor
for the component action then linear
procedures are not applicable and shall not be used.

$$\text{DCR} = \frac{Q_{UD}}{Q_{CE}}$$

Q_{UD} = Force caused by gravity loads and earthquake forces
Q_{CE} = Expected strength of the component or element

3

The Applicability of the Linear Static Procedure

If the building does not have any of these:

1. The fundamental period of the building, T, is greater than or equal to 3.5 times Ts.
2. The ratio of the horizontal dimension at any story to an adjacent story exceeds 1.4.
3. The building has a torsional stiffness irregularity in any story.
4. The building has a vertical stiffness irregularity.
5. The building has a nonorthogonal seismic-force-resisting system.

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Course Title: Existing Buildings
Track 2
Linear Analyses Procedures

- **Course Topics:**

1. The steps in Linear Analyses Procedures.
2. The limitations of each linear analysis method.
3. What are the issues for linear procedures in estimating the deformation-controlled parameters?
4. What are the issues for linear procedures in estimating the force-controlled parameters?
5. Base shear estimation formula and its coefficients.
6. The acceptance criteria for linear analysis.

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Performance Objective

Probability of Exceedance	Mean Return Period (years)
50%/30 years	43
50%/50 years	72
20%/50 years	225
10%/50 years	475
5%/50 years	975
2%/50 years	2,475

Earthquake Hazard Level	Target Building Performance Levels: Operational Level (1-A)	Immediate Occupancy Performance Level (1-B)	Life Safety Performance Level (3-C)	Collapse Prevention Performance Level (5-E)	Life Safety Level (3-C) Moderate
50%/50 year	a	b		d	Some residual strength and stiffness left in all stories. Gravity-load-bearing elements function. No out-of-plane failure of walls. Some permanent drift. Damage to partitions. Continued occupancy might not be likely before repair. Building might not be economical to repair. Falling hazards, such as parapets, mitigated, but many architectural, mechanical, and electrical systems are damaged.
20%/50 year	e	f		h	
BSE-1 (~10%/50 year)				l	Somewhat more damage and slightly higher life-safety risk.
BSE-2 (~2%/50 year)	m	n	o	p	

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Design Base Shear verses **Assessment** Base Shear

BNBC 2020

$$V = S_a W$$

$$S_a = \frac{2}{3}\frac{ZI}{R}C_s$$

ASCE 41-17

$$V = C_1 C_2 C_m S_a W$$

$S_a =$ Design spectral acceleration (in units of g)

$\beta =$ Coefficient used to calculate lower bound for S_a

$Z =$ Seismic zone coefficient

$I =$ Structure importance factor

$R =$ Response reduction factor which depends on structural system

$C_s =$ Normalized acceleration response spectrum

7

Design Base Shear verses **Assessment** Base Shear

BNBC 2020

$$V = S_a W$$

$$S_a = \frac{2}{3}\frac{ZI}{R}C_s$$

ASCE 41-17

$$V = C_1 C_2 C_m S_a W$$

$$m\kappa Q_{CE} > Q_{UD}$$

$m =$ Component capacity modification factor to account for expected ductility associated with this action at the selected Structural Performance Level.

$R =$ Response reduction factor which depends on structural system

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Pseudo Seismic Force

$$V = C_1 C_2 C_m S_a W$$

$$V = C_1 C_2 C_3 C_m S_a W$$

Probability of Exceedance	Mean Return Period (years)
50%/30 years	43
50%/50 years	72
20%/50 years	225
10%/50 years	475
5%/50 years	975
2%/50 years	2,475

Occupancy Performance

Verification Level

Earthquake Hazard Level: 50%/50 year; 20%/50 year; BSE-1 (~10%/50 year); BSE-2 (~2%/50 year)

বাংলাদেশ গেজেট, অতিরিক্ত, ফেব্রুয়ারি ১১, ২০২১ ৩১৯৭

Table 6.2.16: Site Dependent Soil Factor and Other Parameters Defining Elastic Response Spectrum

Soil type	S	T_B(s)	T_C(s)	T_D(s)
SA	1.0	0.15	0.40	2.0
SB	1.2	0.15	0.50	2.0
SC	1.15	0.20	0.60	2.0
SD	1.35	0.20	0.80	2.0
SE	1.4	0.15	0.50	2.0

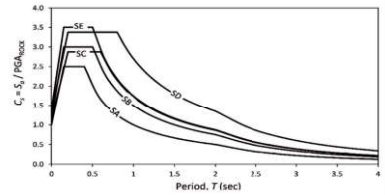

Figure 6.2.26 Normalized design acceleration response spectrum for different site classes.

S_a = Response spectrum acceleration, at the fundamental period and damping ratio of the building in the direction under consideration.

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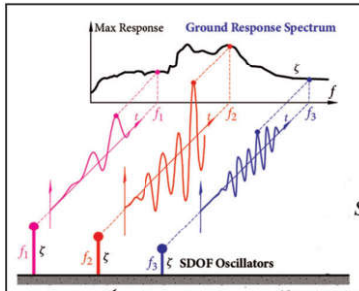
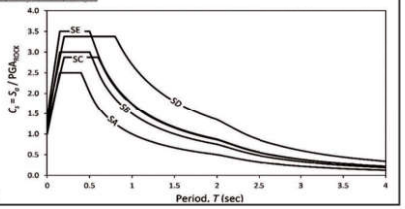
Pseudo Seismic Force



$$V = C_1 C_2 C_m S_a W$$

$S_a =$ Response spectrum acceleration



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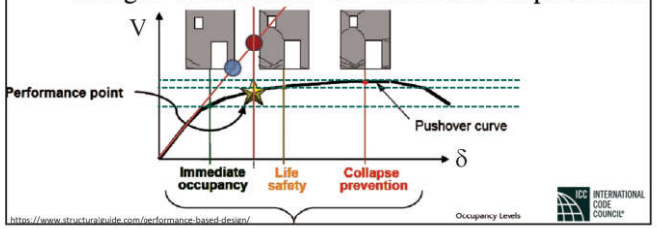
10

Pseudo Seismic Force

$$V = C_1 C_2 C_m S_a W$$

C_1 = Modification factor to relate expected maximum inelastic displacements to displacements calculated for linear elastic response.

C_2 = Modification factor to represent the effect of pinched hysteresis shape, cyclic stiffness degradation, and strength deterioration on maximum displacement.



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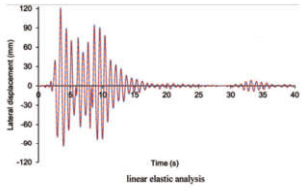
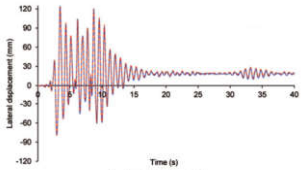
C_1 = Modification factor to relate expected maximum inelastic displacements to displacements calculated for linear elastic response.



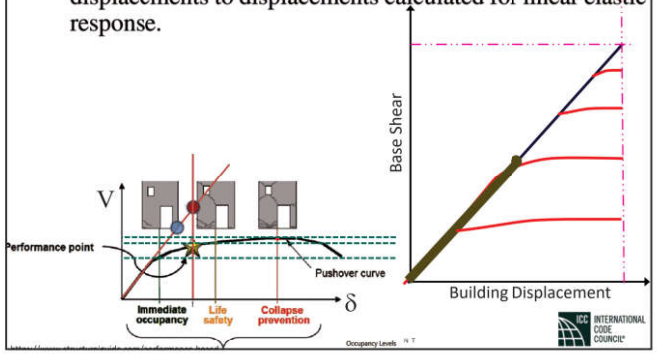


12

Pseudo Seismic Force

$$V = C_1 C_2 C_m S_a W$$

C_1 = Modification factor to relate expected maximum inelastic displacements to displacements calculated for linear elastic response.



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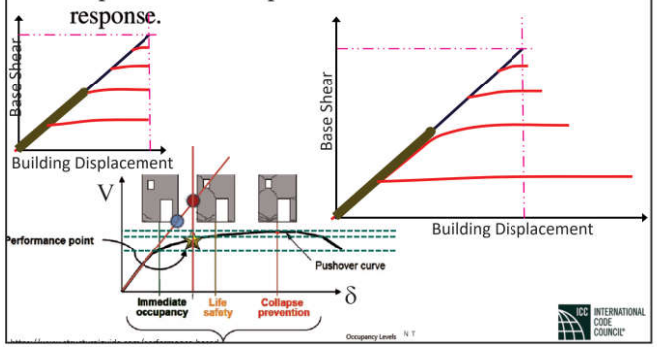
Pseudo Seismic Force

$$V = C_1 C_2 C_m S_a W \qquad C_1 = 1 + \frac{\mu_{strength} - 1}{aT^2}$$

C_1 = Modification factor to relate expected maximum inelastic displacements to displacements calculated for linear elastic response.



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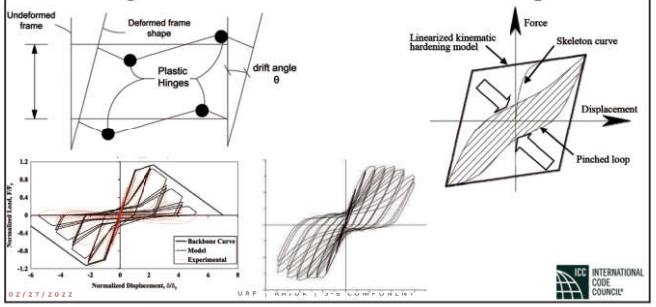
Pseudo Seismic Force

$$V = C_1 C_2 C_m S_a W$$

$$C_2 = 1 + \frac{1}{800}\left(\frac{\mu_{strength} - 1}{T}\right)^2$$

C_2 = Modification factor to represent the effect of pinched hysteresis shape, cyclic stiffness degradation, and strength deterioration on maximum displacement.



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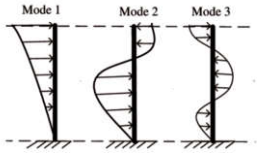
Pseudo Seismic Force

$$V = C_1 C_2 C_m S_a W$$



C_m = Effective mass factor to account for higher modal mass participation effects obtained from Table 7-4.

C_m shall be taken as 1.0 if the fundamental period, T, is greater than 1.0 s;

Table 7-4. Values for Effective Mass Factor C_m

No. of Stories	Concrete Moment Frame	Concrete Shear Wall	Concrete Pier-Spandrel	Steel Moment Frame	Steel Concentrically Braced Frame	Steel Eccentrically Braced Frame	Other
1–2	1.0	1.0	1.0	1.0	1.0	1.0	1.0
3 or more	0.9	0.8	0.8	0.9	0.9	0.9	1.0

Note: C_m shall be taken as 1.0 if the fundamental period, T, in the direction of response under consideration is greater than 1.0 s.

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Pseudo Seismic Force

$$V = C_1 C_2 C_m S_a W$$

Vertical Distribution of Seismic Forces

$$F_x = C_{vx} V \qquad (7\text{-}24)$$

$$C_{vx} = \frac{w_x h_x^k}{\sum_{i=1}^{n} w_i h_i^k} \qquad (7\text{-}25)$$

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The load combinations in Linear Static Procedure

Gravity Loads

$$Q_G = 1.1(Q_D + Q_L + Q_S) \qquad (7\text{-}1)$$

$$Q_G = 0.9 Q_D \qquad (7\text{-}2)$$

$$Q_G = Q_D + Q_L + Q_S \qquad (7\text{-}3)$$

Q_D = Action caused by dead loads;
Q_L = Action caused by live load, equal to 25% of the unreduced live load obtained in accordance with ASCE 7 but not less than the actual live load; and
Q_S = Action caused by effective snow load.

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Actions (responses) caused by loads

$$V = C_1 C_2 C_m S_a W$$

$$Q_G = 1.1(Q_D + Q_L + Q_S)$$

$$Q_G = 0.9 Q_D$$

$$Q_G = Q_D + Q_L + Q_S$$

Q_E Q_G

Q_G = Action caused by gravity loads
Q_E = Action caused by the response to the selected Seismic Hazard Level

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Acceptance Criteria for **Deformation-Controlled** actions

estimation of their Demand and checking with their Capacity

$$Q_{UD} = Q_G + Q_E$$

Q_G = Action caused by gravity loads
Q_E = Action caused by the response to the selected Seismic Hazard Level

$$m\kappa Q_{CE} > Q_{UD}$$

m = Component capacity modification factor to account for expected ductility associated with this action at the selected Structural Performance Level.
Q_{CE} = Expected strength of component deformation-controlled action
κ = Knowledge factor

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Table 10-13. Numerical Acceptance Criteria for Linear Procedures—Reinforced Concrete Beams

			m-Factors[a]				
			Performance Level				
			Component Type				
			Primary			Secondary	
Conditions			IO	LS	CP	LS	CP
Condition i. Beams controlled by flexure[b]							
$\frac{\rho - \rho'}{\rho_{bal}}$	Transverse reinforcement[c]	$\frac{V^d}{b_w d \sqrt{f'_{ce}}}$					
≤0.0	C	≤3 (0.25)	3	6	7	6	10
≤0.0	C	≥6 (0.5)	2	3	4	3	5
≥0.5	C	≤3 (0.25)	2	3	4	3	5
≥0.5	C	≥6 (0.5)	2	2	3	2	4
≤0.0	NC	≤3 (0.25)	2	3	4	3	5
≤0.0	NC	≥6 (0.5)	1.25	2	3	2	4
≥0.5	NC	≤3 (0.25)	2	3	3	3	4
≥0.5	NC	≥6 (0.5)	1.25	2	2	2	3
Condition ii. Beams controlled by shear[b]							
Stirrup spacing ≤ d/2			1.25	1.5	1.75	3	4
Stirrup spacing > d/2			1.25	1.5	1.75	2	3
Condition iii. Beams controlled by inadequate development or splicing along the span[b]							
Stirrup spacing ≤ d/2			1.25	1.5	1.75	3	4
Stirrup spacing > d/2			1.25	1.5	1.75	2	3
Condition iv. Beams controlled by inadequate embedment into beam–column joint[b]			2	2	3	3	4

m = Component capacity modification factor to account for expected ductility associated with this action at the selected Structural Performance Level.

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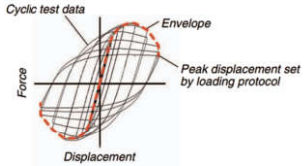
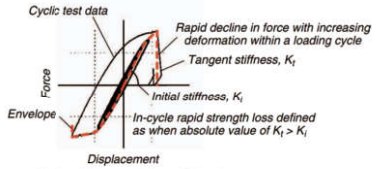
m = Component capacity modification factor to account for expected ductility associated with this action at the selected Structural Performance Level.



(a) Ductile behavior



(b) Rapid Strength Loss Behavior

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Acceptance Criteria for **Deformation-Controlled** actions
estimation of their Demand and checking with their Capacity

$$Q_{UD} = Q_G + Q_E \longrightarrow m\kappa Q_{CE} > Q_{UD}$$

κ = Knowledge factor

Table 6-1. Data Collection Requirements

Data	Level of Knowledge: Minimum		Usual		Comprehensive	
Performance Level	Life Safety (S-3) or lower		Damage Control (S-2) or lower		Immediate Occupancy (S-1) or lower	
Analysis Procedures	LSP, LDP		All		All	
Testing	No tests[a]		Usual testing		Comprehensive testing	
Drawings	Design drawings	Field survey drawings prepared in absence of design drawings	Design drawings	Field survey drawings prepared in absence of design drawings	Design drawings	Field survey drawings prepared in absence of design drawings
Condition Assessment[b]	Visual	Comprehensive	Visual	Comprehensive	Visual	Comprehensive
Material Properties	From design drawings (or documents)[c]	From default values	From design drawings (or documents) and tests	From usual tests	From design drawings (or documents) and tests	From comprehensive tests
Knowledge Factor (κ)[d]	0.9[e,f]	0.75	1.00	1.00	1.00	1.00

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Acceptance Criteria for **Deformation-Controlled** actions
estimation of their Demand and checking with their Capacity

$$Q_{UD} = Q_G + Q_E \longrightarrow m\kappa Q_{CE} > Q_{UD}$$

κ = Knowledge factor

Table 7-7. Calculation of Component Action Capacity: Nonlinear Procedures

Parameter	Deformation Controlled	Force Controlled
Deformation capacity (existing component)	$\kappa \times$ Deformation limit	N/A
Deformation capacity (new component)	Deformation limit	N/A
Strength capacity (existing component)	N/A	$\kappa \times Q_{CL}$
Strength capacity (new component)	N/A	Q_{CL}

Table 7-6. Calculation of Component Action Capacity: Linear Procedures

Parameter	Deformation Controlled	Force Controlled
Existing material strength	Expected mean value with allowance for strain hardening	Lower bound value (approximately mean value minus 1σ level)
Existing action capacity	κQ_{CE}	κQ_{CL}
New material strength	Expected material strength	Specified material strength
New action capacity	Q_{CE}	Q_{CL}

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Acceptance Criteria for **Force-Controlled** actions
estimation of their Demand and checking with their Capacity

1. Q_{UF} shall be taken as the maximum action that can be developed in a component based on a limit-state analysis considering the expected strength of the components delivering force to the component under consideration, or the maximum action developed in the component as limited by the nonlinear response of the building.
2. Alternatively,

$$Q_{UF} = Q_G \pm \frac{\chi Q_E}{C_1 C_2 J}$$

χ = Factor for adjusting action caused by response for the selected Structural Performance Level;
= 1.0 where J is taken as smallest DCR; otherwise,
= 1.0 for Collapse Prevention;
= 1.3 for Life Safety or Immediate Occupancy;

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Acceptance Criteria for **Force-Controlled** actions
estimation of their Demand and checking with their Capacity

$$Q_{UF} = Q_G \pm \frac{\chi Q_E}{C_1 C_2 J}$$

J = Force-delivery reduction factor, greater than or equal to 1.0, taken as the smallest demand–capacity ratio (DCR) of the components in the load path delivering force to the component in question.

Alternatively, values of J equal to 2.0 for a high level of seismicity, 1.5 for a moderate level of seismicity, and 1.0 for a low level of seismicity shall be permitted where not based on calculated DCRs. J shall be taken as 1.0 for the Immediate Occupancy Structural Performance Level in these instances.

In any case where the forces contributing to Q_{UF} are delivered by components of the seismic-force-resisting system that remain elastic, J shall be taken as 1.0.

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Acceptance Criteria for **Force-Controlled** actions
estimation of their Demand and checking with their Capacity

$$Q_{UF} = Q_G \pm \frac{\chi Q_E}{C_1 C_2 J}$$

7.5.2.2.2 Acceptance Criteria for Force-Controlled Actions for LSP or LDP. Force-controlled actions in primary and secondary components shall satisfy Eq. (7-37):

$$\kappa Q_{CL} > Q_{UF} \qquad (7\text{-}37)$$

where

Q_{CL} = Lower-bound strength of a force-controlled action of an element at the deformation level under consideration.

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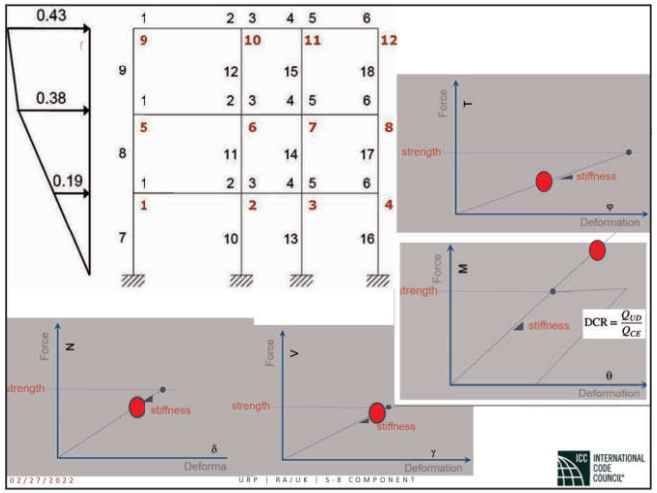

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Summary 1/5

- The two main parts of each "Performance Objective".

Earthquake Hazard Level	Target Building Performance Levels: Operational Performance Level (1-A)	Immediate Occupancy Performance Level (1-B)	Life Safety Performance Level (3-C)	Collapse Prevention Performance Level (5-E)
50%/50 year	a	b		d
20%/50 year	e	f		h
BSE-1 (~10%/50 year)				l
BSE-2 (~2%/50 year)	m	n	o	p

- The difference between R-factor in "BNBC 2020 base shear formula" and m-factor in "systematic evaluation" by linear analysis.

Conditions			IO	LS	CP
Condition i. Beams controlled by flexure[b]					
$\frac{\rho - \rho'}{\rho_{bal}}$	Transverse reinforcement[c]	$\frac{V^d}{b_w d \sqrt{f'_{cE}}}$			
≤0.0	C	≤3 (0.25)	3	6	7

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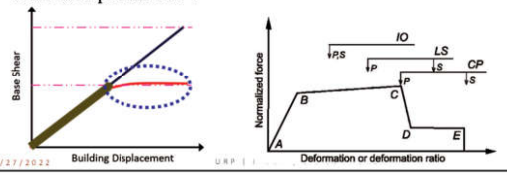
Summary 2/5

- In the base shear formula for systematic linear analysis, the factor "C1" is "the Modification factor to relate maximum inelastic displacements to displacements calculated for linear elastic response". The factor "C1" is mainly dependent on?

- The linear analysis method is not capable of determining the "deformation-controlled parameters".



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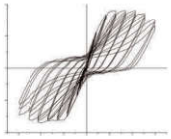
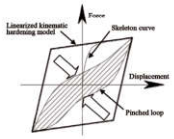
Summary 3/5

- In the base shear formula for systematic linear analysis, the factor "C2" is "the modification factor to represent the effect of pinched hysteresis shape, and strength deterioration on maximum displacement". The factor "C2" is mainly dependent on?





- If a "Force-Controlled Action" does not satisfy the acceptance criteria.

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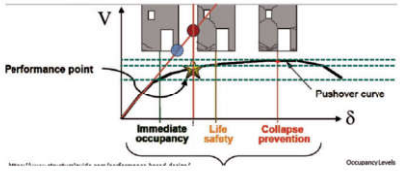
Summary 4/5

- Why the base shear increased by the factors "C1" and "C2"?



- In the base shear formula for linear analysis, the "Earthquake Hazard Level" is represented by?

$$V = C_1 C_2 C_m S_a W$$

S_a = Response spectrum acceleration

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Summary 5/5

- The linear analysis method "is not" capable of determining the "force-controlled parameters".

$$Q_{UD} = Q_G + Q_E \qquad Q_{UF} = Q_G \pm \frac{\chi Q_E}{C_1 C_2 J}$$

- For a linear analysis, when "dynamic methods" should be used instead of "static methods"?

Mass and stiffness irregularity

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Course Title: Existing Buildings
Track 2
Linear Analyses Procedures

- **Course Outcomes:**

1. Understand what the shortcomings of the linear analysis methods are.
2. Can calculate the base shear of buildings for earthquakes with different return periods.
3. Differentiate between the way force-controlled parameters are treated compared to deformation-controlled response parameters.
4. Determine the load combinations needed for estimation of force-controlled and deformation-controlled response parameters.
5. Apply the linear procedures for assessment of existing buildings.

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Questions?
Thank you

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Existing Buildings 03
Nonlinear Analyses Procedures

Date: 1 March 2022
Session 41

A.S. Moghadam
International Institute of Earthquake Engineering and Seismology (IIEES)
and
S-04 Team Leader, JV of NKY-Protek-Sheltech

S-8A SEng PRP Training Program

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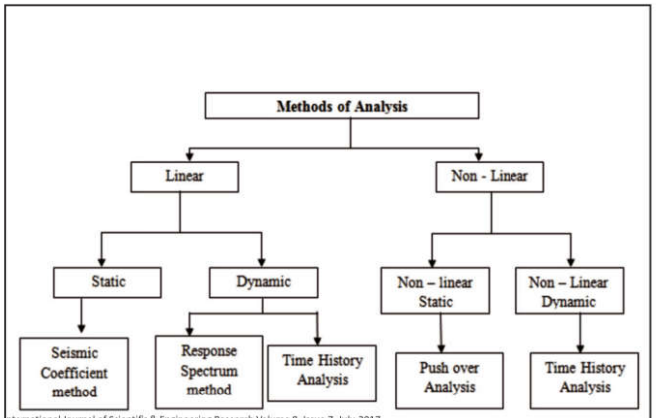

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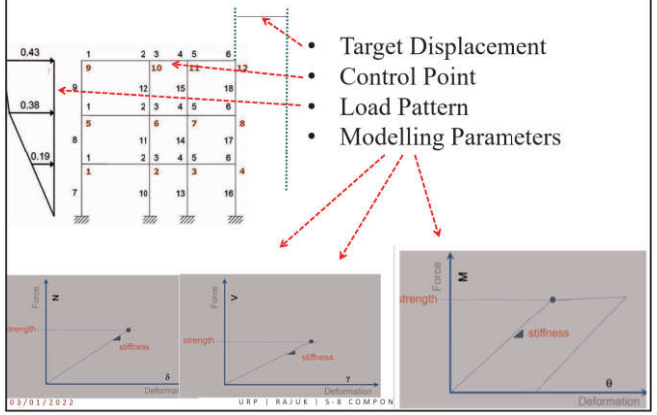
Course Title: Existing Buildings Track 3 Nonlinear Analyses Procedures

- **Course Topics:**

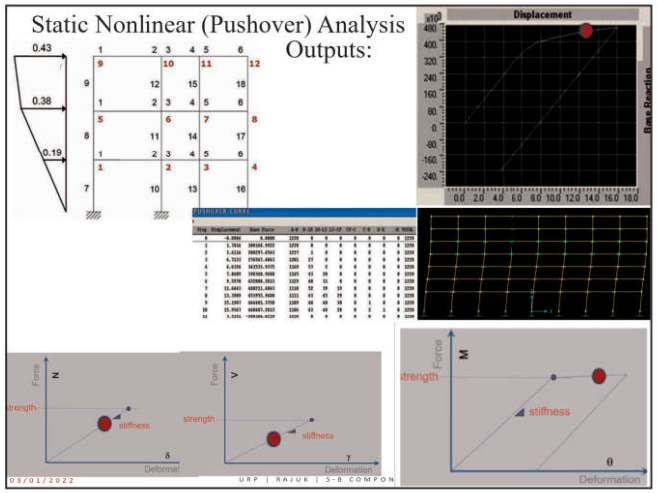
1. The steps in Nonlinear Analyses Procedures.
2. The limitations of Nonlinear analysis methods.
3. Modelling of deformation-controlled parameters for Nonlinear analyses.
4. Modelling of force-controlled parameters for Nonlinear analyses.
5. Target displacement estimation formula and its coefficients.
6. The acceptance criteria for Nonlinear analysis.

Static Nonlinear (Pushover) Analysis Inputs:



- Target Displacement
- Control Point
- Load Pattern
- Modelling Parameters

Static Nonlinear (Pushover) Analysis Outputs:



Performance Objective

Probability of Exceedance	Mean Return Period (years)
50%/30 years	43
50%/50 years	72
20%/50 years	225
10%/50 years	475
5%/50 years	975
2%/50 years	2,475

Earthquake Hazard Level	Target Building Performance Levels: Operational Level (1-A)	Immediate Occupancy Performance Level (1-B)	Life Safety Performance Level (3-C)	Collapse Prevention Performance Level (5-E)
50%/50 year	a	b		d
20%/50 year	e	f		h
BSE-1 (~10%/50 year)				l
BSE-2 (~2%/50 year)	m	n	o	p

Life Safety Level (3-C)

Moderate

Some residual strength and stiffness left in all stories. Gravity-load-bearing elements function. No out-of-plane failure of walls. Some permanent drift. Damage to partitions. Continued occupancy might not be likely before repair. Building might not be economical to repair.

Faling hazards, such as parapets, mitigated, but many architectural, mechanical, and electrical systems are damaged.

Somewhat more damage and slightly higher life-safety risk.

Target Displacement

$$\delta_t = C_0 C_1 C_2 S_a \frac{T_e^2}{4\pi^2} g$$

C_0 = Modification factor to relate spectral displacement of an equivalent single-degree-of-freedom (SDOF) system to the roof displacement of the building multiple-degree-of-freedom (MDOF) system.

C_1 = Modification factor to relate expected maximum inelastic displacements to displacements calculated for linear elastic response.

C_2 = Modification factor to represent the effect of pinched hysteresis shape, cyclic stiffness degradation, and strength deterioration on maximum displacement.

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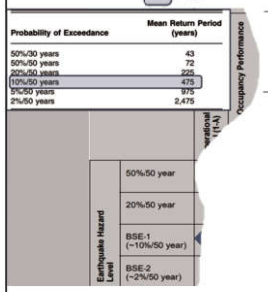
Target Displacement

$$\delta_t = C_0 C_1 C_2 S_a \frac{T_e^2}{4\pi^2} g$$



বাংলাদেশ গেজেট, অতিরিক্ত, ফেব্রুয়ারি ১১, ২০২১ ৩১৯৭

Table 6.2.16: Site Dependent Soil Factor and Other Parameters Defining Elastic Response Spectrum

Soil type	S	T_B(s)	T_C(s)	T_D(s)
SA	1.0	0.15	0.40	2.0
SB	1.2	0.15	0.50	2.0
SC	1.15	0.20	0.60	2.0
SD	1.35	0.20	0.80	2.0
SE	1.4	0.15	0.50	2.0

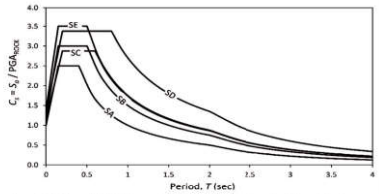


Figure 6.2.26 Normalized design acceleration response spectrum for different site classes.

S_a = Response spectrum acceleration, at the fundamental period and damping ratio of the building in the direction under consideration.

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Target Displacement

$$\delta_t = C_0 C_1 C_2 S_a \frac{T_e^2}{4\pi^2} g$$

Displacement $= y = A \sin \frac{2\pi}{T} t \quad = A \sin \omega t$

Velocity $= \dot{y} = \frac{2\pi A}{T} \cos \frac{2\pi}{T} t = \omega A \cos \omega t$

Acceleration $= \ddot{y} = \frac{4\pi^2 A}{-T^2} \sin \frac{2\pi}{T} t = -\omega^2 A \sin \omega t$
$= -\omega^2 y$

- Peak spectral acceleration, A, is related to peak spectral displacement, D, by the equation:

$$A = \omega_n^2 D = \left(\frac{2\pi}{T_n}\right)^2 D$$

$S_a \rightarrow S_d$

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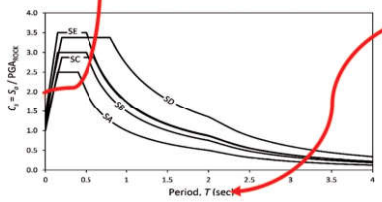
Target Displacement

$$\delta_t = C_0 C_1 C_2 S_a \frac{T_e^2}{4\pi^2} g$$

$$T_e = T_i \sqrt{\frac{K_i}{K_e}}$$



$$T = 2\pi \sqrt{\frac{m}{k}}$$

$$T_i = 2\pi \sqrt{\frac{m}{k_i}}$$

$$T_e = 2\pi \sqrt{\frac{m}{k_e}}$$

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Target Displacement

$$\delta_t = C_0 C_1 C_2 S_a \frac{T_e^2}{4\pi^2} g$$

$$T_e = T_i \sqrt{\frac{K_i}{K_e}}$$

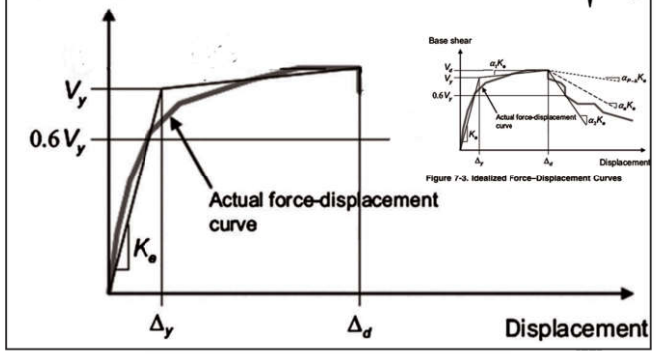

Figure 7-3. Idealized Force–Displacement Curves

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Target Displacement

$$\delta_t = C_0 C_1 C_2 S_a \frac{T_e^2}{4\pi^2} g$$

C_0 = Modification factor to relate spectral displacement of an equivalent single-degree-of-freedom (SDOF) system to the roof displacement of the building multiple-degree-of-freedom (MDOF) system.

Table 7-5. Values for Modification Factor C_0

	Shear Buildings[a]		Other Buildings
Number of Stories	Triangular Load Pattern (1.1, 1.2, 1.3)	Uniform Load Pattern (2.1)	Any Load Pattern
1	1.0	1.0	1.0
2	1.2	1.15	1.2
3	1.2	1.2	1.3
5	1.3	1.2	1.4
10+	1.3	1.2	1.5

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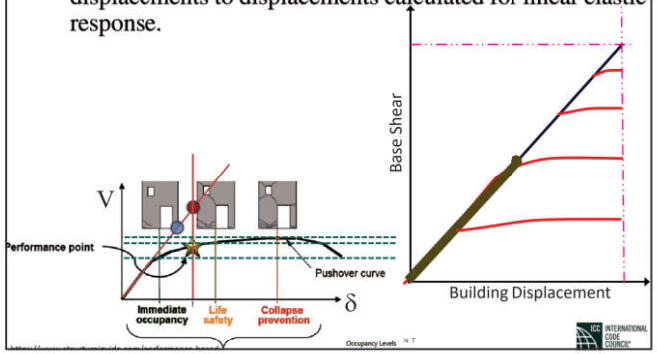
Target Displacement

$$\delta_t = C_0 C_1 C_2 S_a \frac{T_e^2}{4\pi^2} g$$

$$C_1 = 1 + \frac{\mu_{strength} - 1}{aT_e^2}$$

C_1 = Modification factor to relate expected maximum inelastic displacements to displacements calculated for linear elastic response.



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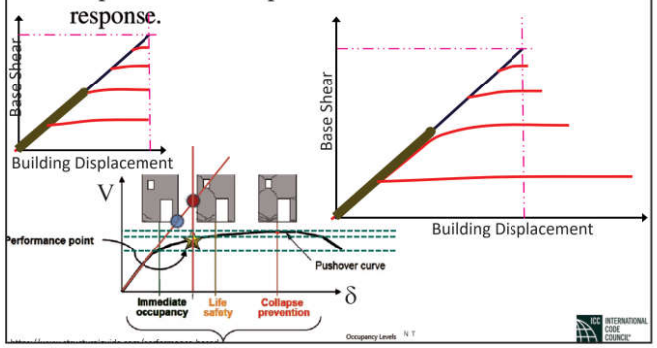
Target Displacement

$$\delta_t = C_0 C_1 C_2 S_a \frac{T_e^2}{4\pi^2} g$$

$$C_1 = 1 + \frac{\mu_{strength} - 1}{aT_e^2}$$

C_1 = Modification factor to relate expected maximum inelastic displacements to displacements calculated for linear elastic response.



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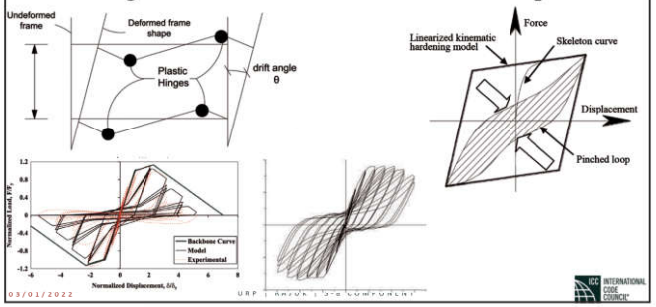
Target Displacement

$$\delta_t = C_0 C_1 C_2 S_a \frac{T_e^2}{4\pi^2} g \qquad C_2 = 1 + \frac{1}{800}\left(\frac{\mu_{strength} - 1}{T_e}\right)^2$$

C_2 = Modification factor to represent the effect of pinched hysteresis shape, cyclic stiffness degradation, and strength deterioration on maximum displacement.



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Target Displacement

A modification factor relating expected maximum inelastic displacements to displacements calculated for linear elastic response

A factor to relate the spectral displacement of the equivalent SDF and building roof displacement

The effective period of the building in the direction under consideration

Acceleration due to gravity, 9.81 m/sec²

Target Displacement $\delta_T = C_0 C_1 C_2 S_a \frac{T_e^2}{4\pi^2} g$

A modification factor to represent the effect of hysteresis shape on the maximum displacement response

The spectral acceleration at the effective period and damping ratio of the building in the direction under consideration

https://link.springer.com/content/pdf/10.1007F978-3-319-61914-9_15.pdf

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The load combinations in Nonlinear Static Procedure

Gravity Loads

$$Q_G = 1.1(Q_D + Q_L + Q_S) \qquad (7\text{-}1)$$

$$Q_G = 0.9 Q_D \qquad (7\text{-}2)$$

$$Q_G = Q_D + Q_L + Q_S \qquad (7\text{-}3)$$

Q_D = Action caused by dead loads;
Q_L = Action caused by live load, equal to 25% of the unreduced live load obtained in accordance with ASCE 7 but not less than the actual live load; and
Q_S = Action caused by effective snow load.

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Actions (responses) caused by loads

$$\delta_t = C_0 C_1 C_2 S_a \frac{T_e^2}{4\pi^2} g$$

$$Q_G = 1.1(Q_D + Q_L + Q_S)$$

$$Q_G = 0.9 Q_D$$

$$Q_G = Q_D + Q_L + Q_S$$

Q_E Q_G

Q_G = Action caused by gravity loads
Q_E = Action caused by the response to the selected Seismic Hazard Level

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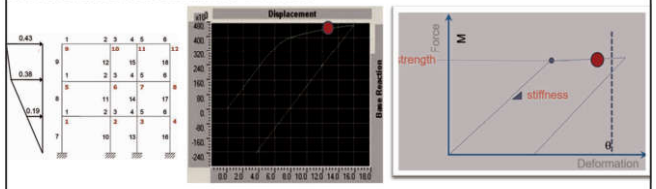
Acceptance Criteria for **Deformation-Controlled** actions

7.5.3.2.2 Acceptance Criteria for Deformation-Controlled Actions for NSP or NDP. Primary and secondary components shall have expected deformation capacities not less than maximum deformation demands calculated at target displacements. Primary and secondary component demands shall be within the acceptance criteria for nonlinear components at the selected Structural Performance Level.



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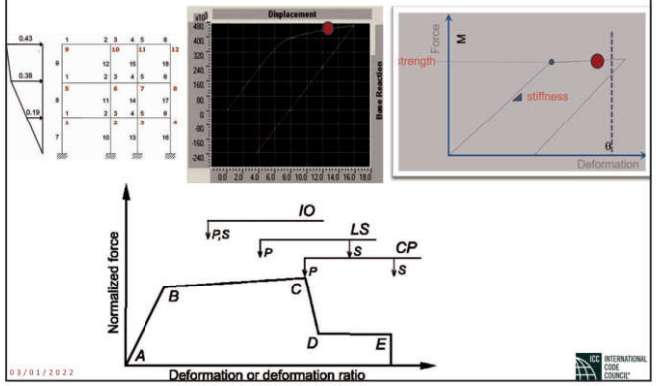
Acceptance Criteria for **Deformation-Controlled** actions

7.5.3.2.2 Acceptance Criteria for Deformation-Controlled Actions for NSP or NDP.



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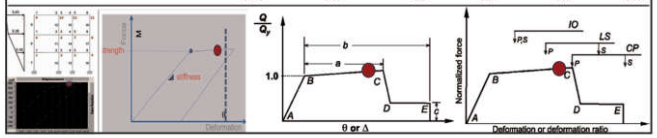
Acceptance Criteria for **Deformation-Controlled** actions

Table 10-7. Modeling Parameters and Numerical Acceptance Criteria for Nonlinear Procedures—Reinforced Concrete Beams

			Modeling Parameters[a]			Acceptance Criteria[a]		
			Plastic Rotation Angle (radians)		Residual Strength Ratio	Plastic Rotation Angle (radians)		
						Performance Level		
Conditions			a	b	c	IO	LS	CP
Condition i. Beams controlled by flexure[b]								
$\frac{\rho - \rho'}{\rho_{bal}}$	Transverse reinforcement[d]	$\frac{V^{e}}{b_w d \sqrt{f'_{ce}}}$						
≤0.0	C	≤3 (0.25)	0.025	0.05	0.2	0.010	0.025	0.05
≤0.0	C	≥6 (0.5)	0.02	0.04	0.2	0.005	0.02	0.04
≥0.5	C	≤3 (0.25)	0.02	0.03	0.2	0.005	0.02	0.03
≥0.5	C	≥6 (0.5)	0.015	0.02	0.2	0.005	0.015	0.02
≤0.0	NC	≤3 (0.25)	0.02	0.03	0.2	0.005	0.02	0.03
≤0.0	NC	≥6 (0.5)	0.01	0.015	0.2	0.0015	0.01	0.015
≥0.5	NC	≤3 (0.25)	0.01	0.015	0.2	0.005	0.01	0.015
≥0.5	NC	≥6 (0.5)	0.005	0.01	0.2	0.0015	0.005	0.01
Condition ii. Beams controlled by shear[b]								
Stirrup spacing ≤ d/2			0.0030	0.02	0.2	0.0015	0.01	0.02
Stirrup spacing > d/2			0.0030	0.01	0.2	0.0015	0.005	0.01
Condition iii. Beams controlled by inadequate development or splicing along the span[b]								
Stirrup spacing ≤ d/2			0.0030	0.02	0.0	0.0015	0.01	0.02
Stirrup spacing > d/2			0.0030	0.01	0.0	0.0015	0.005	0.01
Condition iv. Beams controlled by inadequate embedment into beam–column joint[b]								
			0.015	0.03	0.2	0.01	0.02	0.03



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Acceptance Criteria for **Force-Controlled** actions

7.5.3.2.3 Acceptance Criteria for Force-Controlled Actions for NSP or NDP. Force-controlled components that are not explicitly included in the mathematical model with nonlinear force-deformation properties per Section 7.5.1.2 shall satisfy:

$$\gamma\chi(Q_{UF} - Q_G) + Q_G \leq Q_{CL}$$

Q_{UF} = the force-controlled demand
Q_G = Gravity load demand
Q_{CL} = Lower-bound component strength
γ = Load factor obtained from Table 7-8:

Table 7-8. Load Factor for Force-Controlled Behaviors

Action Type	γ
Critical	1.3
Ordinary	1.0
Noncritical	1.0

χ is taken as 1.0 for Collapse Prevention or 1.3 for Life Safety and Immediate Occupancy.

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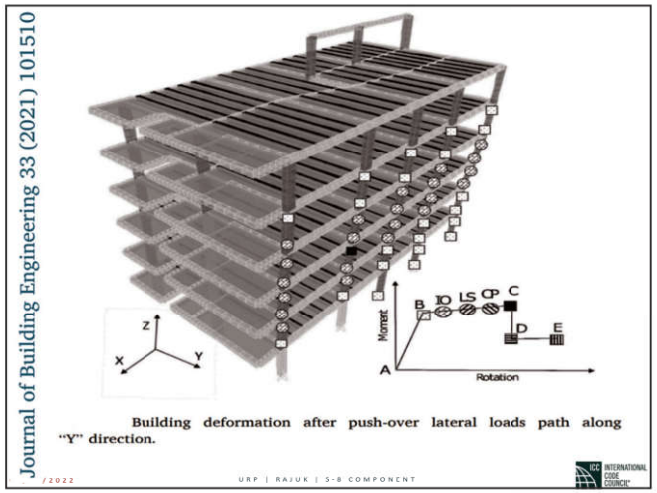

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Nonlinear Static Analysis

versus

Nonlinear Dynamic Analysis

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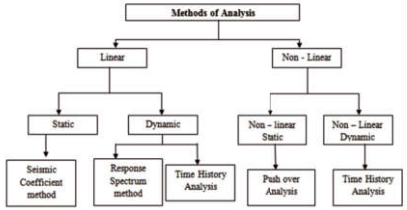
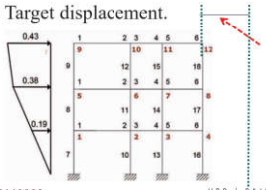
Summary 1/5

- Pushover analysis.



- Target displacement.



- Target Displacement

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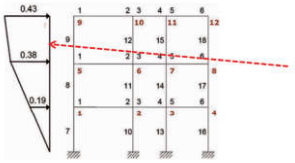
Summary 2/5

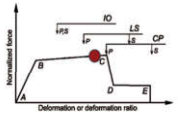
- Load Pattern in the Pushover Analysis?



- Load Pattern

- Acceptance Criteria for "Deformation-Controlled actions" in nonlinear analysis.

Deformation capacity $\geq$ Deformation Demand



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Summary 3/5

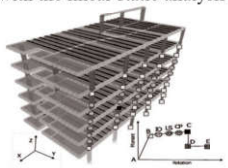
- Acceptance Criteria for "Force-Controlled actions" in nonlinear analysis.

Lower-bound strength $\geq$ Force Demand

$$\gamma\chi(Q_{UF} - Q_G) + Q_G \leq Q_{CL}$$

- The main difference of pushover with the linear static analysis.

Effects of damages are considered



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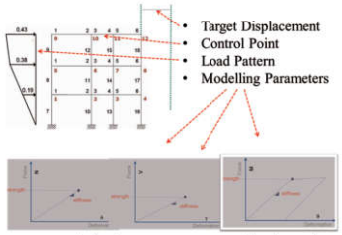
Summary 4/5

- Inputs in pushover analysis.



- If a "Force-Controlled action", does not satisfy the acceptance criteria, what is the building performance level?

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Summary 5/5

- In linear analysis the load combinations for "load-controlled" and "deformation-controlled actions" are different. Why in nonlinear analysis they are the same?

Effects of damages are considered

- Why nonlinear dynamic analysis has not been the preferred approach of analysis yet?

What will be the characteristics of the next earthquake? How to select the earthquake records?

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Course Title: Existing Buildings
Track 3
Nonlinear Analyses Procedures

- **Course Outcomes:**

1. Understand what the advantages and shortcomings of the Nonlinear analysis methods are.
2. Can calculate the Target Displacement of buildings for earthquakes with different return periods.
3. Differentiate between the way force-controlled parameters are treated compared to deformation-controlled response parameters in Nonlinear analyses.
4. Determine the load combinations needed for estimation of force-controlled and deformation-controlled response parameters in Nonlinear analyses.
5. Apply the Nonlinear procedures for assessment of existing buildings.

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Questions?
Thank you

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Existing Buildings 04 Modelling Parameters and Acceptance Criteria

Date: 5 March 2022
Session 43

A.S. Moghadam
International Institute of Earthquake Engineering and Seismology (IIEES)
and
S-04 Team Leader, JV of NKY-Protek-Sheltech

S-8A SEng PRP Training Program

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Course Title: Existing Buildings Track 4 Modelling Parameters and Acceptance Criteria

- **Course Topics:**

1. Implementing different deficiencies of the existing buildings in their modelling parameters.
2. The factors controlling the Modelling Parameters and Acceptance Criteria.
3. Different standard shapes of Modelling Parameters and their characteristics.
4. Introducing the tables of structural systems for deriving their Modelling Parameters and Acceptance Criteria values.
5. Importance of using different analysis methods in systematic evaluation.

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10 CONCRETE . . . . . .
10.1 Scope . . . . . .
10.2 Material Properties and Condition Assessment
10.3 General Assumptions and Requirements . . .
10.4 Concrete Moment Frames . . . . . .
10.5 Precast Concrete Frames . . . . . .
10.6 Concrete Frames with Infills . . . . . .
10.7 Concrete Structural Walls . . . . . .
10.8 Precast Concrete Structural Walls . . . . . .
10.9 Concrete Braced Frames . . . . . .
10.10 Cast-in-Place Concrete Diaphragms . . . . . .
10.11 Precast Concrete Diaphragms . . . . . .
10.12 Concrete Foundations . . . . . .

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3

10 CONCRETE . . . . . .
10.1 Scope . . . . . .
10.2 Material Properties and Condition Assessment

Properties of In-Place Materials and Components

Table 6-1. Data Collection Requirements

	Level of Knowledge					
Data	**Minimum**		**Usual**		**Comprehensive**	
Performance Level	Life Safety (S-3) or lower		Damage Control (S-2) or lower		Immediate Occupancy (S-1) or lower	
Analysis Procedures	LSP, LDP		All		All	
Testing	No tests[a]		Usual testing		Comprehensive testing	
Drawings	Design drawings	Field survey drawings prepared in absence of design drawings	Design drawings	Field survey drawings prepared in absence of design drawings	Design drawings	Field survey drawings prepared in absence of design drawings
Condition Assessment[b]	Visual	Comprehensive	Visual	Comprehensive	Visual	Comprehensive
Material Properties	From design drawings (or documents)[c]	From default values	From design drawings (or documents) and tests	From usual tests	From design drawings (or documents) and tests	From comprehensive tests
Knowledge Factor (κ)[d]	0.9[e,f]	0.75	1.00	1.00	1.00	1.00

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Table 10-1. Factors to Translate Lower-Bound Material Properties to Expected Strength Material Properties

Material Property	Factor
Concrete compressive strength	1.50
Reinforcing steel tensile and yield strength	1.25
Connector steel yield strength	1.50

Table 10-2. Default Lower-Bound Compressive Strength of Structural Concrete, lb/in.^{2} (MPa)

Time Frame	Footings	Beams	Slabs	Columns	Walls
1900–1919	1,000 to 2,500 (7 to 17)	2,000 to 3,000 (14 to 21)	1,500 to 3,000 (10 to 21)	1,500 to 3,000 (10 to 21)	1,000 to 2,500 (7 to 17)
1920–1949	1,500 to 3,000 (10 to 21)	2,000 to 3,000 (14 to 21)	2,000 to 3,000 (14 to 21)	2,000 to 4,000 (14 to 28)	2,000 to 3,000 (14 to 21)
1950–1969	2,500 to 3,000 (17 to 21)	3,000 to 4,000 (21 to 28)	3,000 to 4,000 (21 to 28)	3,000 to 6,000 (21 to 40)	2,500 to 4,000 (17 to 28)
1970–present	3,000 to 4,000 (21 to 28)	3,000 to 5,000 (21 to 35)	3,000 to 5,000 (21 to 35)	3,000 to 10,000 (21 to 70)	3,000 to 5,000 (21 to 35)

Table 10-3. Default Lower-Bound Tensile and Yield Properties of Reinforcing Steel for Various Periods

	Structural[a]	Intermediate[a]	Hard[a]				
Grade	33	40	50	60	65	70	75
Minimum Yield, lb/in.^{2} (MPa)	33,000 (230)	40,000 (280)	50,000 (350)	60,000 (420)	65,000 (450)	70,000 (485)	75,000 (520)
Minimum Tensile, lb/in.^{2} (MPa)	55,000 (380)	70,000 (485)	80,000 (550)	90,000 (620)	75,000 (520)	80,000 (550)	100,000 (690)
Year							
1911–1959	x	x	x		x		
1959–1966	x	x	x	x	x	x	x
1966–1972		x	x	x	x	x	
1972–1974		x	x	x	x	x	
1974–1987		x	x	x	x	x	
1987–present		x	x	x	x	x	x

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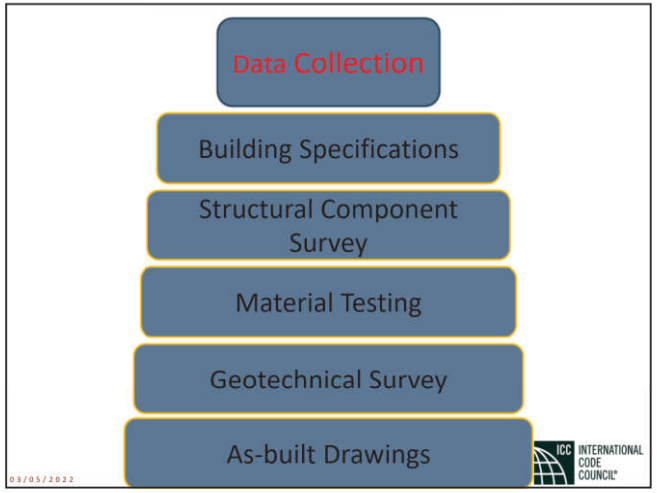
7



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κ = Knowledge factor

Table 7-6. Calculation of Component Action Capacity: Linear Procedures

Parameter	Deformation Controlled	Force Controlled
Existing material strength	Expected mean value with allowance for strain hardening	Lower bound value (approximately mean value minus 1σ level)
Existing action capacity	κQ_{CE}	κQ_{CL}
New material strength	Expected material strength	Specified material strength
New action capacity	Q_{CE}	Q_{CL}

Table 7-7. Calculation of Component Action Capacity: Nonlinear Procedures

Parameter	Deformation Controlled	Force Controlled
Deformation capacity (existing component)	$\kappa \times$ Deformation limit	N/A
Deformation capacity (new component)	Deformation limit	N/A
Strength capacity (existing component)	N/A	$\kappa \times Q_{CL}$
Strength capacity (new component)	N/A	Q_{CL}

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10 CONCRETE . . . . . .
10.1 Scope . . . . . .
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Table 10-5. Effective Stiffness Values

Component	Flexural Rigidity	Shear Rigidity	Axial Rigidity
Beams—nonprestressed[a]	$0.3E_{cE}I_g$	$0.4E_{cE}A_w$	—
Beams—prestressed[a]	$E_{cE}I_g$	$0.4E_{cE}A_w$	—
Columns with compression caused by design gravity loads $\geq 0.5A_g f'_{cE}$[b]	$0.7E_{cE}I_g$	$0.4E_{cE}A_w$	$E_{cE}A_g$
Columns with compression caused by design gravity loads $\leq 0.1A_g f'_{cE}$ or with tension[b]	$0.3E_{cE}I_g$	$0.4E_{cE}A_w$	$E_{cE}A_g$ (compression) $E_{sE}A_s$ (tension)
Beam–column joints	Refer to Section 10.4.2.2.1		$E_{cE}A_g$
Flat slabs—nonprestressed	Refer to Section 10.4.4.2	$0.4E_{cE}A_g$	—
Flat slabs—prestressed	Refer to Section 10.4.4.2	$0.4E_{cE}A_g$	—
Walls—cracked[c]	$0.35E_{cE}A_g$	$0.4E_{cE}A_w$	$E_{cE}A_g$ (compression) $E_{sE}A_s$ (tension)

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10

10 CONCRETE . . . . . .
10.1 Scope . . . . . .
10.2 Material Properties and Condition Assessment
10.3 General Assumptions and Requirements . . .

Implementing different deficiencies

2. Where existing deformed straight bars, hooked bars, and lap-spliced bars do not meet the development requirements of (1) above, the capacity of existing reinforcement shall be calculated using Eq. (10-1):

$$f_s = 1.25\left(\frac{l_b}{l_d}\right)^{2/3} f_{yL} \le f_{yL/E} \qquad \text{(10-1a)}$$

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Implementing different deficiencies

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$$f_s = 1.25\left(\frac{l_b}{l_d}\right)^{2/3} f_{yL} \le f_{yL/E} \qquad \text{(10-1a)}$$

Table 10-7. Modeling Parameters and Numerical Acceptance Criteria for Nonlinear Procedures—Reinforced Concrete Beams

			Modeling Parameters[a]			Acceptance Criteria[a]		
			Plastic Rotation Angle (radians)		Residual Strength Ratio	Plastic Rotation Angle (radians) Performance Level		
Conditions			a	b	c	IO	LS	CP
Condition i. Beams controlled by flexure[b]								
$\frac{\rho - \rho'}{\rho_{bal}}$	Transverse reinforcement[c]	$\frac{V^*}{b_w d \sqrt{f'_{ce}}}$						
≤0.0	C	≤3 (0.25)	0.025	0.05	0.2	0.010	0.025	0.05
≤0.0	C	≥6 (0.5)	0.02	0.04	0.2	0.005	0.02	0.04
≥0.5	C	≤3 (0.25)	0.02	0.03	0.2	0.005	0.02	0.03
≥0.5	C	≥6 (0.5)	0.015	0.02	0.2	0.005	0.015	0.02
≤0.0	NC	≤3 (0.25)	0.02	0.03	0.2	0.005	0.02	0.03
≤0.0	NC	≥6 (0.5)	0.01	0.015	0.2	0.0015	0.01	0.015
≥0.5	NC	≤3 (0.25)	0.01	0.015	0.2	0.005	0.01	0.015
≥0.5	NC	≥6 (0.5)	0.005	0.01	0.2	0.0015	0.005	0.01
Condition ii. Beams controlled by shear[b]								
Stirrup spacing ≤ d/2			0.0030	0.02	0.2	0.0015	0.01	0.02
Stirrup spacing > d/2			0.0030	0.01	0.2	0.0015	0.005	0.01
Condition iii. Beams controlled by inadequate development or splicing along the span[b]								
Stirrup spacing ≤ d/2			0.0030	0.02	0.0	0.0015	0.01	0.02
Stirrup spacing > d/2			0.0030	0.01	0.0	0.0015	0.005	0.01
Condition iv. Beams controlled by inadequate embedment into beam–column joint[b]			0.015	0.03	0.2	0.01	0.02	0.03

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Implementing different deficiencies

Table 10-6. Component Ductility Demand Classification

Maximum Value of DCR or Displacement Ductility	Descriptor
<2	Low ductility demand
2 to 4	Moderate ductility demand
>4	High ductility demand

Example:

columns, lap-spliced transverse reinforcement shall be assumed to be not more than 50% effective in regions of moderate ductility demand and ineffective in regions of high ductility demand, and

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10.4 Concrete Moment Frames . . . . . .

10.4.1 Types of Concrete Moment Frames . . . . . .
10.4.1.1 Reinforced Concrete Beam–Column Moment Frames . . . . . .
10.4.1.2 Post-tensioned Concrete Beam–Column Moment Frames . . . . . .
10.4.1.3 Slab–Column Moment Frames. . . . . .
10.4.2 Reinforced Concrete Beam–Column Moment Frames . . . . . .
10.4.2.1 General . . . . . .
10.4.2.2 Stiffness of Reinforced Concrete Beam–Column Moment Frames . . . . .
10.4.2.3 Strength of Reinforced Concrete Beam–Column Moment Frames . . . . .
10.4.2.4 Acceptance Criteria for Reinforced Concrete Beam–Column Moment Frames
10.4.2.5 Retrofit Measures for Reinforced Concrete Beam–Column Moment Frames .
10.4.3 Post-tensioned Concrete Beam–Column Moment Frames . . . . . .
10.4.3.1 General . . . . . .
10.4.3.2 Stiffness of Post-tensioned Concrete Beam–Column Moment Frames . . .
10.4.3.3 Strength of Post-tensioned Concrete Beam–Column Moment Frames . . .
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10.4.4.4 Acceptance Criteria for Slab–Column Moment Frames. . . . . .
10.4.4.5 Retrofit Measures for Slab–Column Moment Frames. . . . . .

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The most important contributions of Systematic Evaluation

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			Modeling Parameters[a]			Acceptance Criteria[a]		
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≤0.0	NC	≥6 (0.5)	0.01	0.015	0.2	0.0015	0.01	0.015
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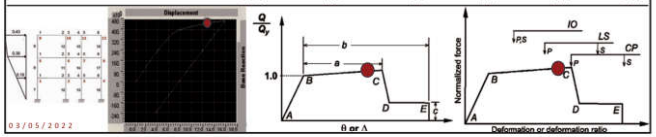
Condition ii. Beams controlled by shear[b]								
Stirrup spacing ≤ d/2			0.0030	0.02	0.2	0.0015	0.01	0.02
Stirrup spacing > d/2			0.0030	0.01	0.2	0.0015	0.005	0.01
Condition iii. Beams controlled by inadequate development or splicing along the span[b]								
Stirrup spacing ≤ d/2			0.0030	0.02	0.0	0.0015	0.01	0.02
Stirrup spacing > d/2			0.0030	0.01	0.0	0.0015	0.005	0.01
Condition iv. Beams controlled by inadequate embedment into beam–column joint[b]			0.015	0.03	0.2	0.01	0.02	0.03



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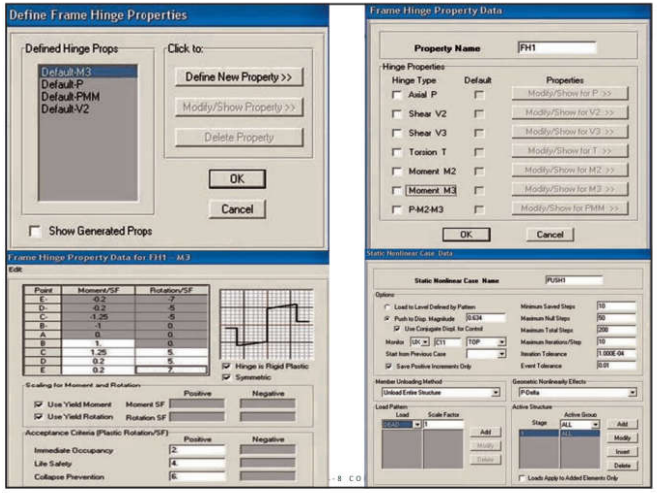

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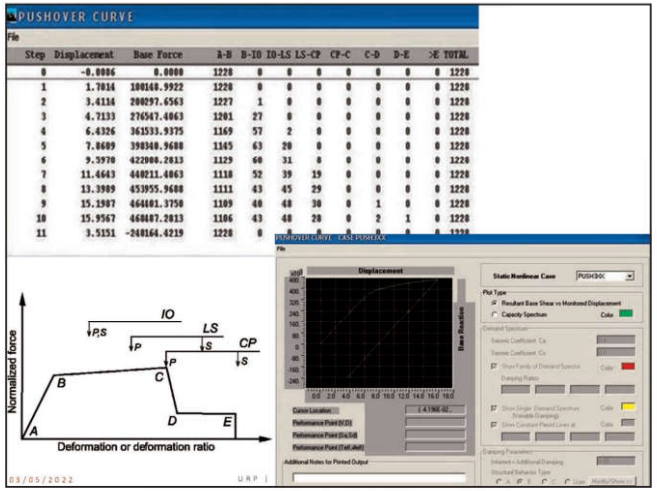

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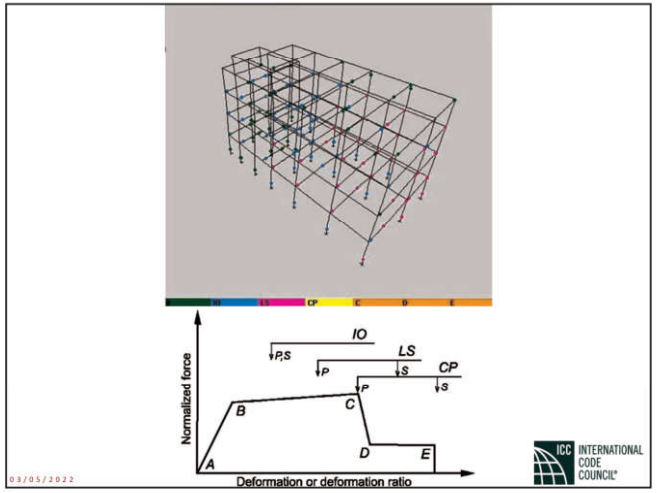

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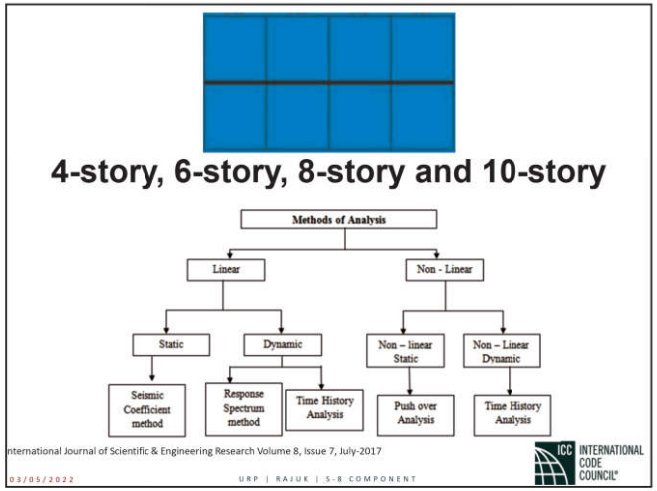
International Code Council (ICC)



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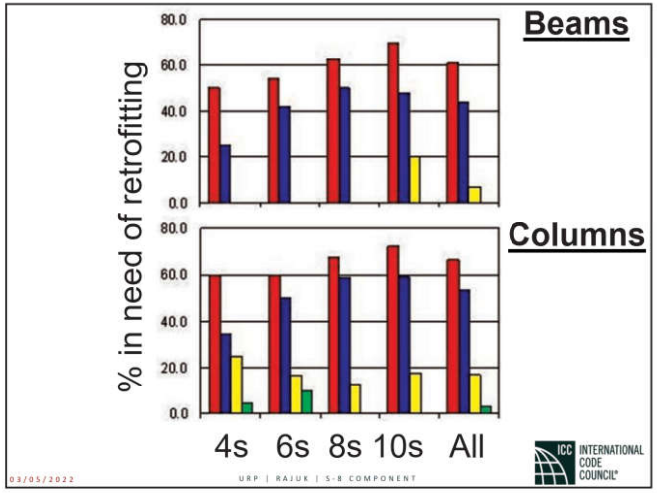

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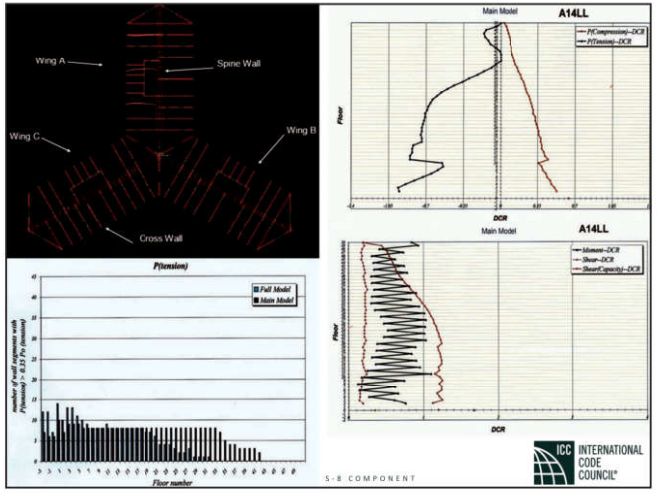

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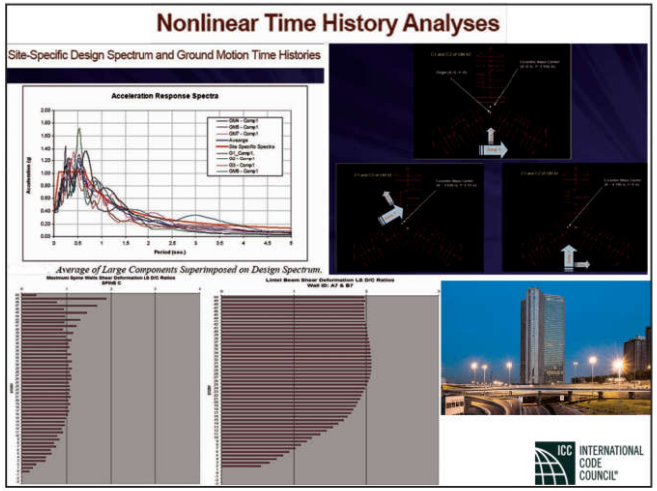

25

Existing Buildings

a) Seismic Evaluation

- Fragility Curves and Vulnerability functions
- Visual Screening
- Rapid Vulnerability Assessment (RVA)
- Preliminary Engineering Assessment (PEA)
- ✓ Detailed Engineering Assessment (DEA)
  ASCE 41-17
- TIER 1 Screening
- TIER 2 Deficiency-Based Evaluation and Retrofit
- ✓ TIER 3 **Systematic** Evaluation and Retrofit

And many more methods …

b) Seismic Retrofitting Technique Selection and Detailing

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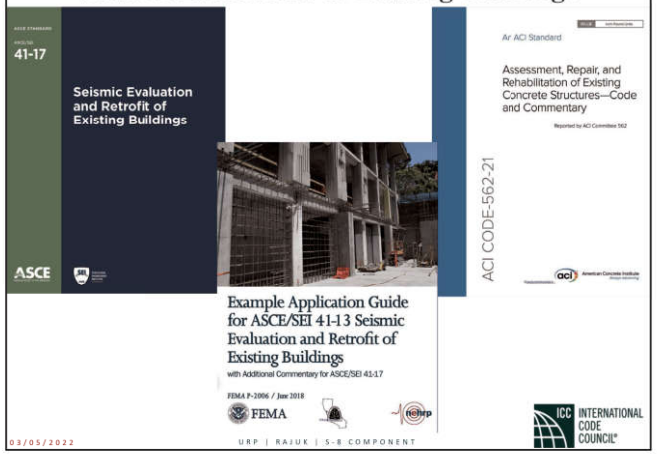
Seismic Evaluation of Existing Buildings



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Seismic Retrofitting Technique Selection and Detailing

Deficiency		Rehabilitation Technique				
Category	Deficiency	Add New Elements	Enhance Existing Elements	Improve Connections Between Elements	Reduce Demand	Remove Selected Components
Global Strength	Insufficient number of frames or weak frames	Concrete/masonry shear wall [12.4.2] Steel braced frame [12.4.1] Concrete or steel moment frame Steel moment frame	Increase size of columns and/or beams [12.4.5]		Remove upper story or stories [24.2] Seismically isolate [24.3] Supplemental damping [24.4]	
Global Stiffness	Insufficient number of frames or frames with inadequate stiffness	Concrete/masonry shear wall [12.4.2] Steel braced frame [12.4.1] Concrete or steel moment frame	Increase size of columns and/or beams [12.4.5] Fiber composite wrap of gravity columns [12.4.4] Concrete/steel jacket of gravity columns [12.4.5] Provide detailing of all other elements to accept drift		Supplemental damping [24.4]	Remove components causing shear columns
Configuration	Soft story or weak story	Add strength or stiffness in story to match balance of floors				
	Re-entrant corner Torsional layout	Add floor area to minimize effect of corner Add balancing walls, braced frames, or moment frames		Provide chords in diaphragm		

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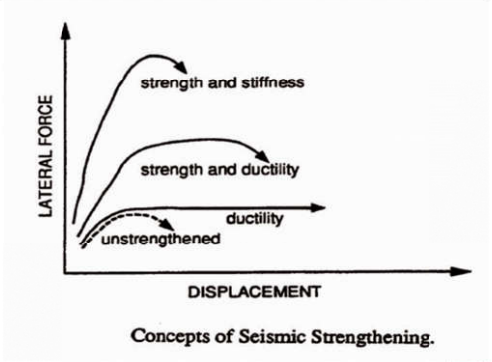
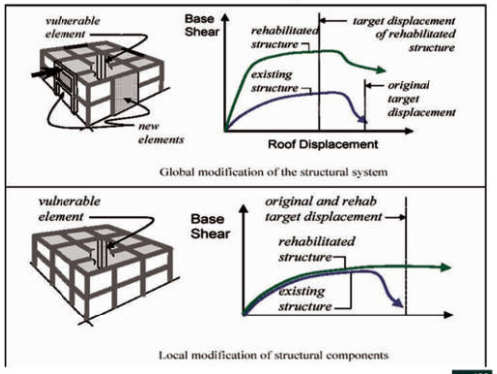
Seismic Retrofitting Technique Selection and Detailing



Concepts of Seismic Strengthening.

29

Seismic Retrofitting Technique Selection and Detailing



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Seismic Retrofitting Technique Selection and Detailing

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Seismic Retrofitting Technique Selection and Detailing

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Summary 1/5

- The major innovation of "systematic assessment and retrofitting approaches".

Table 10-7. Modeling Parameters and Numerical Acceptance Criteria for Nonlinear Procedures—Reinforced Concrete Beams

			Modeling Parameters[a]			Acceptance Criteria[a]		
			Plastic Rotation Angle (radians)		Residual Strength Ratio	Plastic Rotation Angle (radians) Performance Level		
Conditions			a	b	c	IO	LS	CP
Condition i. Beams controlled by flexure[b]								
$\frac{\rho - \rho'}{\rho_{bal}}$	Transverse reinforcement[c]	$\frac{V}{b_w d \sqrt{f'_c}}$						
≤0.0	C	≤3 (0.25)	0.025	0.05	0.2	0.010	0.025	0.05
≤0.0	C	≥6 (0.5)	0.02	0.04	0.2	0.005	0.02	0.04
≥0.5	C	≤3 (0.25)	0.02	0.03	0.2	0.005	0.02	0.03
≥0.5	C	≥6 (0.5)	0.015	0.02	0.2	0.005	0.015	0.02
≤0.0	NC	≤3 (0.25)	0.02	0.03	0.2	0.005	0.02	0.03
≤0.0	NC	≥6 (0.5)	0.01	0.015	0.2	0.0015	0.01	0.015
≥0.5	NC	≤3 (0.25)	0.01	0.015	0.2	0.005	0.01	0.015
≥0.5	NC	≥6 (0.5)	0.005	0.01	0.2	0.0015	0.005	0.01
Condition ii. Beams controlled by shear[b]								
Stirrup spacing ≤ d/2			0.0030	0.02	0.2	0.0015	0.01	0.02
Stirrup spacing > d/2			0.0030	0.01	0.2	0.0015	0.005	0.01
Condition iii. Beams controlled by inadequate development or splicing along the span[d]								
Stirrup spacing ≤ d/2			0.0030	0.02	0.0	0.0015	0.01	0.02
Stirrup spacing > d/2			0.0030	0.01	0.0	0.0015	0.005	0.01
Condition iv. Beams controlled by inadequate embedment into beam-column joint[d]								
			0.015	0.03	0.2	0.01	0.02	0.03

- Most difficult and time-consuming part of a "systematic evaluation" project.

Building Configuration

Material Testing

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Summary 2/5

- The "systematic evaluation" approach toward deficient elements in a building.

Table 10-7. Modeling Parameters and Numerical Acceptance Criteria for Nonlinear Procedures—Reinforced Concrete Beams

			Modeling Parameters[a]			Acceptance Criteria[a]		
			Plastic Rotation Angle (radians)		Residual Strength Ratio	Plastic Rotation Angle (radians) Performance Level		
Conditions			a	b	c	IO	LS	CP
Condition i. Beams controlled by flexure[b]								
$\frac{\rho - \rho'}{\rho_{bal}}$	Transverse reinforcement[c]	$\frac{V}{b_w d \sqrt{f'_c}}$						
≤0.0	C	≤3 (0.25)	0.025	0.05	0.2	0.010	0.025	0.05
≤0.0	C	≥6 (0.5)	0.02	0.04	0.2	0.005	0.02	0.04
≥0.5	C	≤3 (0.25)	0.02	0.03	0.2	0.005	0.02	0.03
≥0.5	C	≥6 (0.5)	0.015	0.02	0.2	0.005	0.015	0.02
≤0.0	NC	≤3 (0.25)	0.02	0.03	0.2	0.005	0.02	0.03
≤0.0	NC	≥6 (0.5)	0.01	0.015	0.2	0.0015	0.01	0.015
≥0.5	NC	≤3 (0.25)	0.01	0.015	0.2	0.005	0.01	0.015
≥0.5	NC	≥6 (0.5)	0.005	0.01	0.2	0.0015	0.005	0.01
Condition ii. Beams controlled by shear[b]								
Stirrup spacing ≤ d/2			0.0030	0.02	0.2	0.0015	0.01	0.02
Stirrup spacing > d/2			0.0030	0.01	0.2	0.0015	0.005	0.01
Condition iii. Beams controlled by inadequate development or splicing along the span[d]								
Stirrup spacing ≤ d/2			0.0030	0.02	0.0	0.0015	0.01	0.02
Stirrup spacing > d/2			0.0030	0.01	0.0	0.0015	0.005	0.01
Condition iv. Beams controlled by inadequate embedment into beam-column joint[d]								
			0.015	0.03	0.2	0.01	0.02	0.03

- Modelling Parameters presented in the tables of existing building codes are only provided for the deformation-controlled actions.

Force-controlled actions should remain linear.

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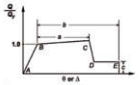
Summary 3/5

- "Modelling Parameters"

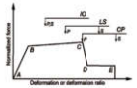


Nonlinear capacity curve of a Deformation-Controlled action

Table 10-7. Modeling Parameters and Numerical Acceptance Criteria for Nonlinear Procedures—Reinforced Concrete Beams

Conditions			Modeling Parameters[a]			Acceptanc	
			Plastic Rotation Angle (radians)		Residual Strength Ratio	Plastic Rotation	
			a	b	c	IO	
Condition i. Beams controlled by flexure[a]							
$\frac{\rho-\rho'}{\rho_{bal}}$	Transverse reinforcement[c]	$\frac{V}{b_w d\sqrt{f'_c}}$					
≤0.0	C	≤3 (0.25)	0.025	0.05	0.2	0.010	0.
≤0.0	C	≥6 (0.5)	0.02	0.04	0.2	0.005	0.
≥0.5	C	≤3 (0.25)	0.02	0.03	0.2	0.005	0.
≥0.5	C	≥6 (0.5)	0.015	0.02	0.2	0.005	0.
≤0.0	NC	≤3 (0.25)	0.02	0.03	0.2	0.005	0.

- "Acceptance Criteria"



Limits of performance levels for a Deformation-Controlled action

Nonlinear Procedures—Reinforced Concrete Beams

Modeling Parameters[a]		Acceptance Criteria[a]		
		Plastic Rotation Angle (radians)		
ngle (radians)	Residual Strength Ratio	Performance Level		
b	c	IO	LS	CP
0.05	0.2	0.010	0.025	0.05
0.04	0.2	0.005	0.02	0.04
0.03	0.2	0.005	0.02	0.03
0.02	0.2	0.005	0.015	0.02

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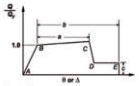
Summary 4/5

- Larger or smaller values for "Modelling Parameters".

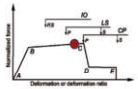


Level of Ductile behaviour

Table 10-7. Modeling Parameters and Numerical Acceptance Criteria for Nonlinear Procedures—Reinforced Concrete Beams

Conditions			Modeling Parameters[a]			Acceptanc	
			Plastic Rotation Angle (radians)		Residual Strength Ratio	Plastic Rotation	
			a	b	c	IO	
Condition i. Beams controlled by flexure[a]							
$\frac{\rho-\rho'}{\rho_{bal}}$	Transverse reinforcement[c]	$\frac{V}{b_w d\sqrt{f'_c}}$					
≤0.0	C	≤3 (0.25)	0.025	0.05	0.2	0.010	0.
≤0.0	C	≥6 (0.5)	0.02	0.04	0.2	0.005	0.
≥0.5	C	≤3 (0.25)	0.02	0.03	0.2	0.005	0.
≥0.5	C	≥6 (0.5)	0.015	0.02	0.2	0.005	0.
≤0.0	NC	≤3 (0.25)	0.02	0.03	0.2	0.005	0.

- Larger or smaller values for "Acceptance Criteria".



More chance of Satisfying the performance level

Nonlinear Procedures—Reinforced Concrete Beams

Modeling Parameters[a]		Acceptance Criteria[a]		
		Plastic Rotation Angle (radians)		
ngle (radians)	Residual Strength Ratio	Performance Level		
b	c	IO	LS	CP
0.05	0.2	0.010	0.025	0.05
0.04	0.2	0.005	0.02	0.04
0.03	0.2	0.005	0.02	0.03
0.02	0.2	0.005	0.015	0.02

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Summary 5/5

- Main usage of non-destructive tests.

Uniformity of the material

- The "Knowledge factor" is being used to modify component capacities.

Capacities of the existing components

Table 7-7. Calculation of Component Action Capacity: Nonlinear Procedures

Parameter	Deformation Controlled	Force Controlled
Deformation capacity (existing component)	$\kappa \times$ Deformation limit	N/A
Deformation capacity (new component)	Deformation limit	N/A
Strength capacity (existing component)	N/A	$\kappa \times Q_{CL}$
Strength capacity (new component)	N/A	Q_{CL}

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Course Title: Existing Buildings
Track 4
Modelling Parameters and Acceptance Criteria

- **Course Outcomes:**

1. Understand the relation between level of deficiency in an element with values of different modelling parameters for that element.
2. Can derive the modelling parameters for different structural systems from the tables.
3. Have an idea about alternative ways of estimating the modelling parameters and acceptance criteria for members of a structural system.
4. Recognize the performance level of a building based on the values of acceptance criteria of the elements of the system.
5. Understand consequence of different analysis methods in systematic evaluation.

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Questions?
Thank you

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Preliminary Bridge Design (AASHTO LRFD 2017)

Date: 23 February 2022
Session 37 – Bridges 01

Nur Yazdani, Ph.D. P.E., F.ASCE, F.ACI, F.SEI
Professor of Civil Engineering
The University of Texas at Arlington, USA

S-8A SEng PRP
Training Program

BASIC CONCRETE BRIDGE TYPES

In several thousand years since we started building bridges, we have built only

FOUR TYPES OF BRIDGES

Girder bridges
Arch bridges
Suspension bridges
Cable-stayed bridges

BASIC BRIDGE TYPES

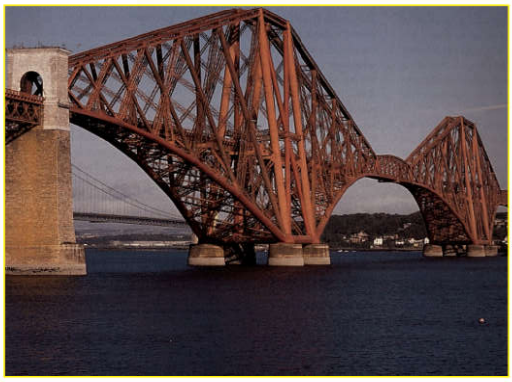
Cable-stayed Bridge

Girder Bridge

Suspension Bridge

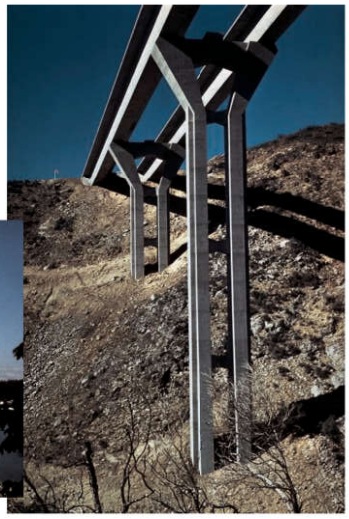
Arch Bridge

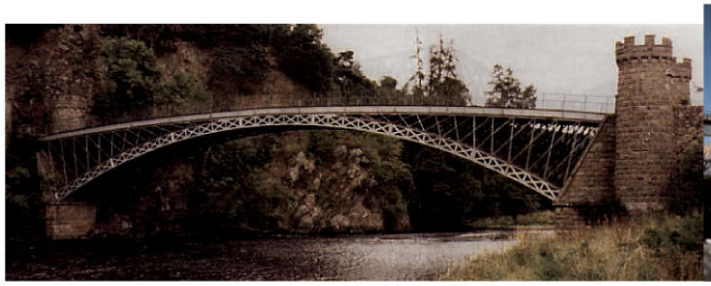
BASIC BRIDGE TYPES



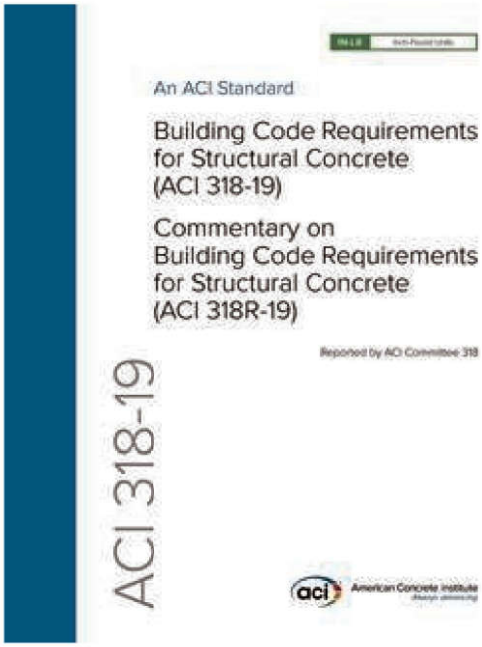
Today's bridges

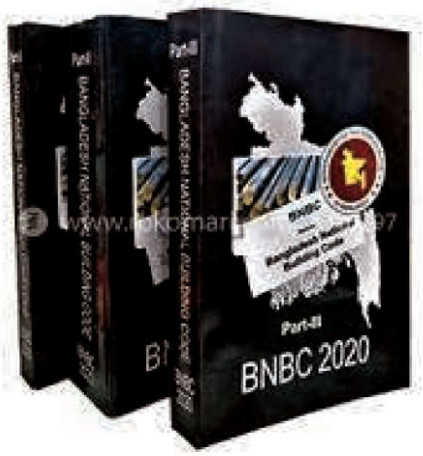
more sophisticated, bigger, stronger, and more durable





WHAT ENGINEERS MAY BE USED TO WORKING WITH

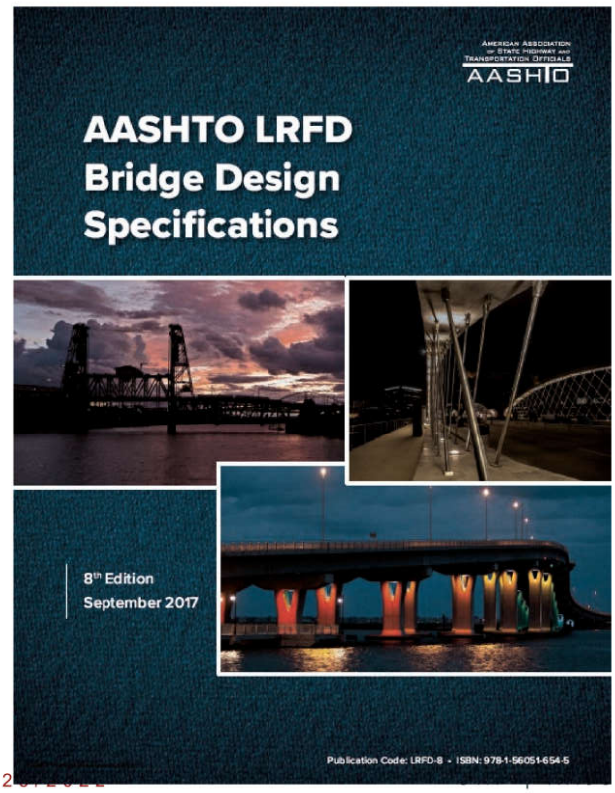


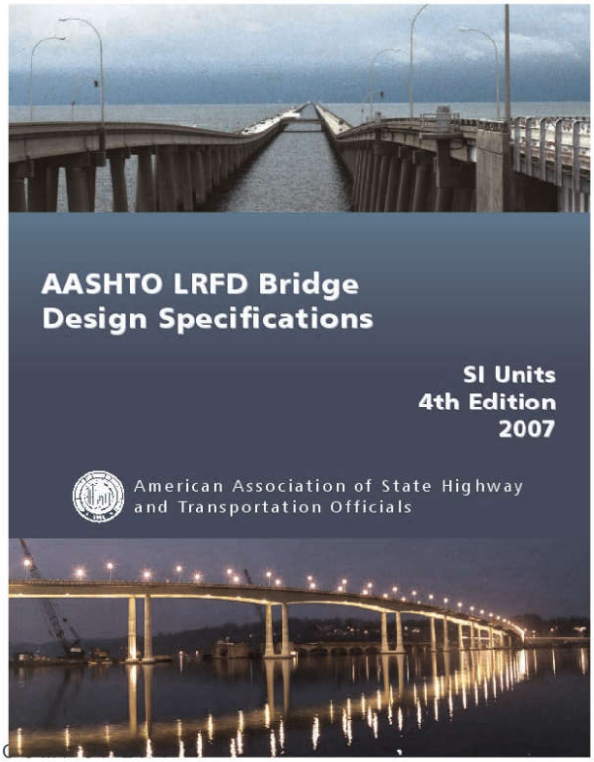


BRIDGE DESIGN CODES

- BNBC is only applicable to building design.
- ACI 318 Code is also applicable to building design.
- There is currently no dedicated bridge design, construction or evaluation code in Bangladesh.
- However, it is customary in Bangladesh to use the following codes:
  - AASHTO LRFD Specifications for Bridge Design (2017, U.S. units)
  - AASHRO LRFD Specifications for Bridge Design (2007, SI units)

AASHTO LRFD BRIDGE DESIGN Specifications

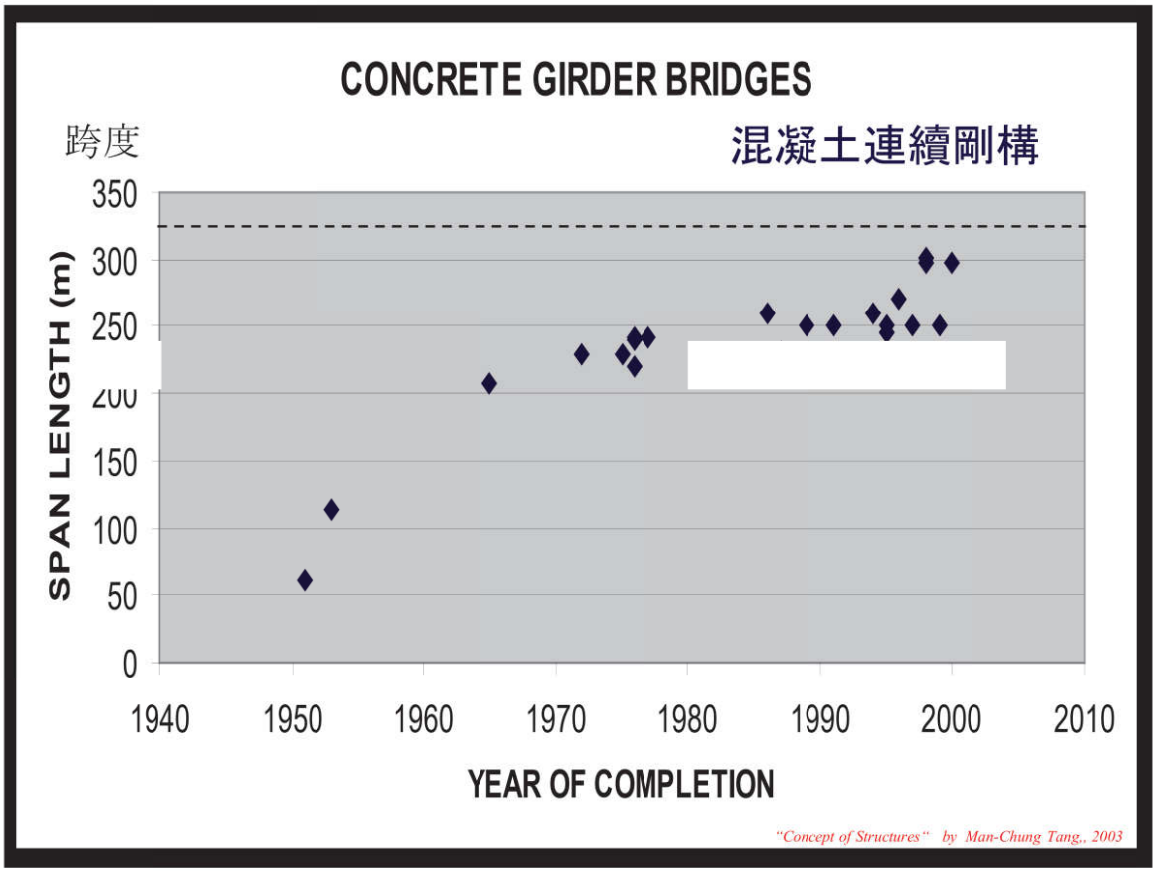




Preliminary Bridge Design

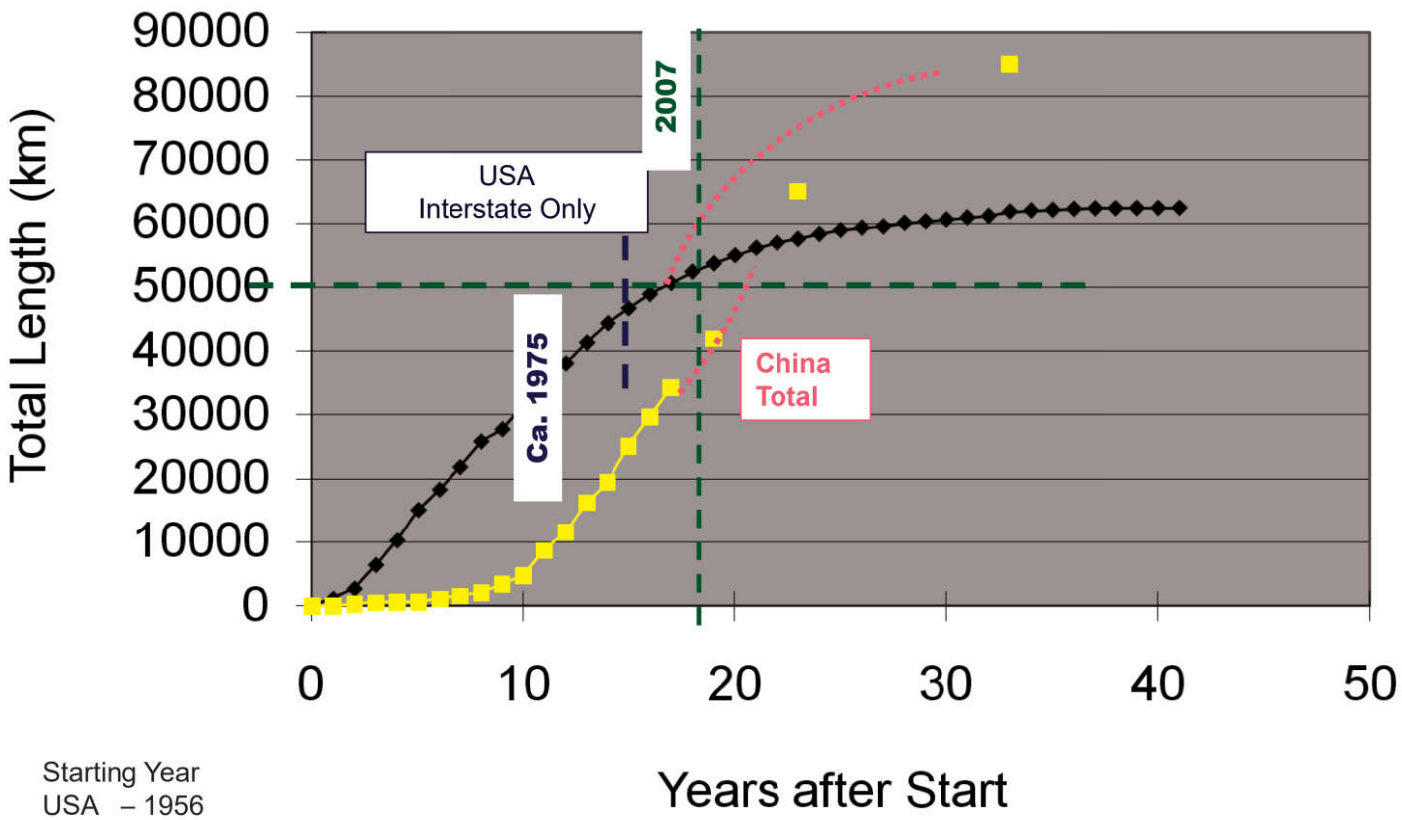
- Preliminary design is the first step in designing an economical bridge.
- AASHTO does not provide any such guidelines.

So, we shall look at other sources.



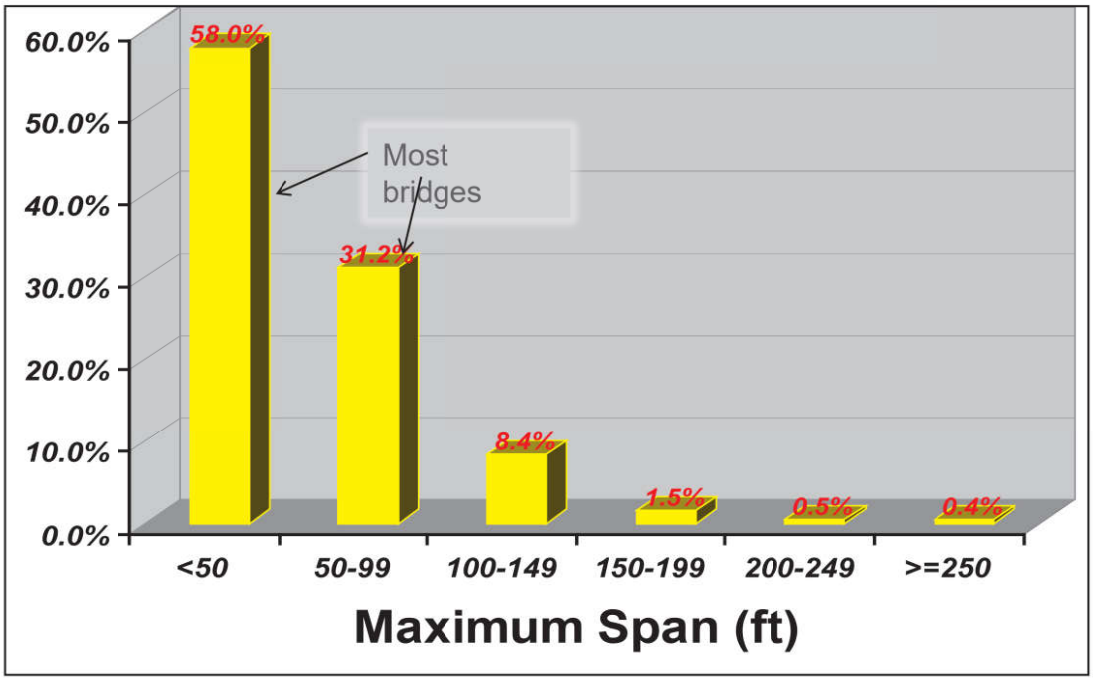
FREEWAY CONSTRUCTION

China vs. USA



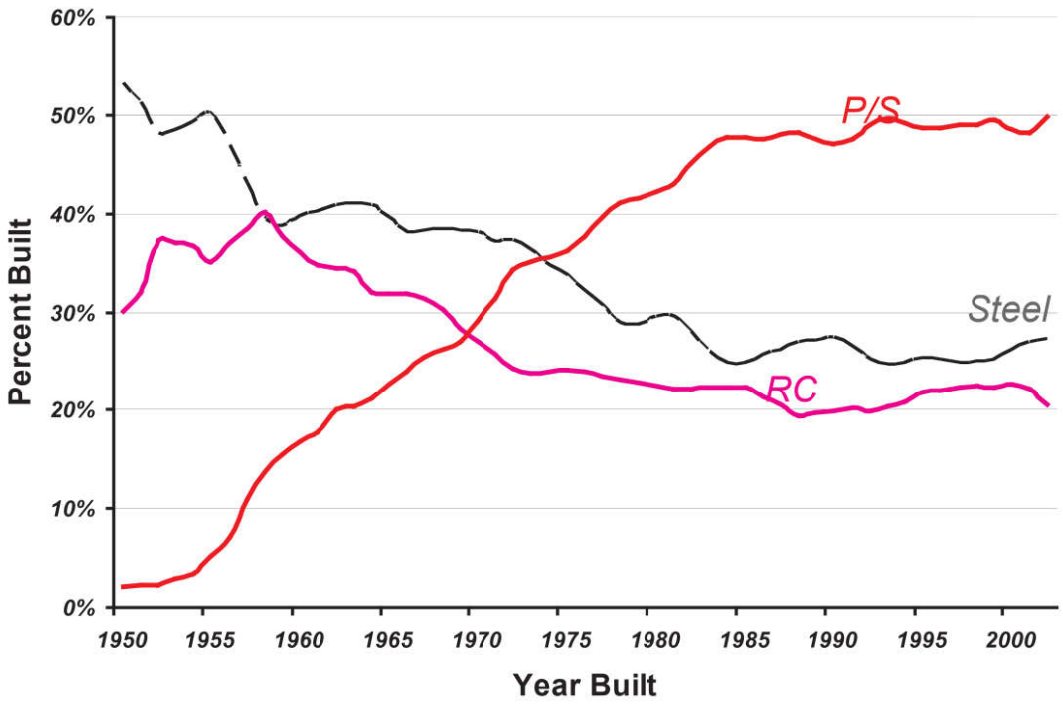
Starting Year
USA – 1956
China – 1987

All Existing U.S. Bridges
NBI Data, 2018



Total Built = 475,000 Bridges

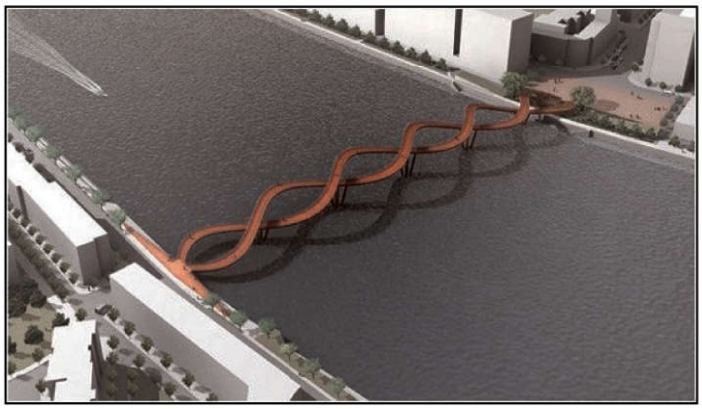
Bridges Built, 2018 NBI Data



Next Innovations in Bridges?

- Composite bridges
- Entirely modular bridges
- 3D printing – precast mold building
- BIM in general
- Laser and drones for bridge monitoring
- Jointless bridges

Unusual Bridges



Unusual Bridges

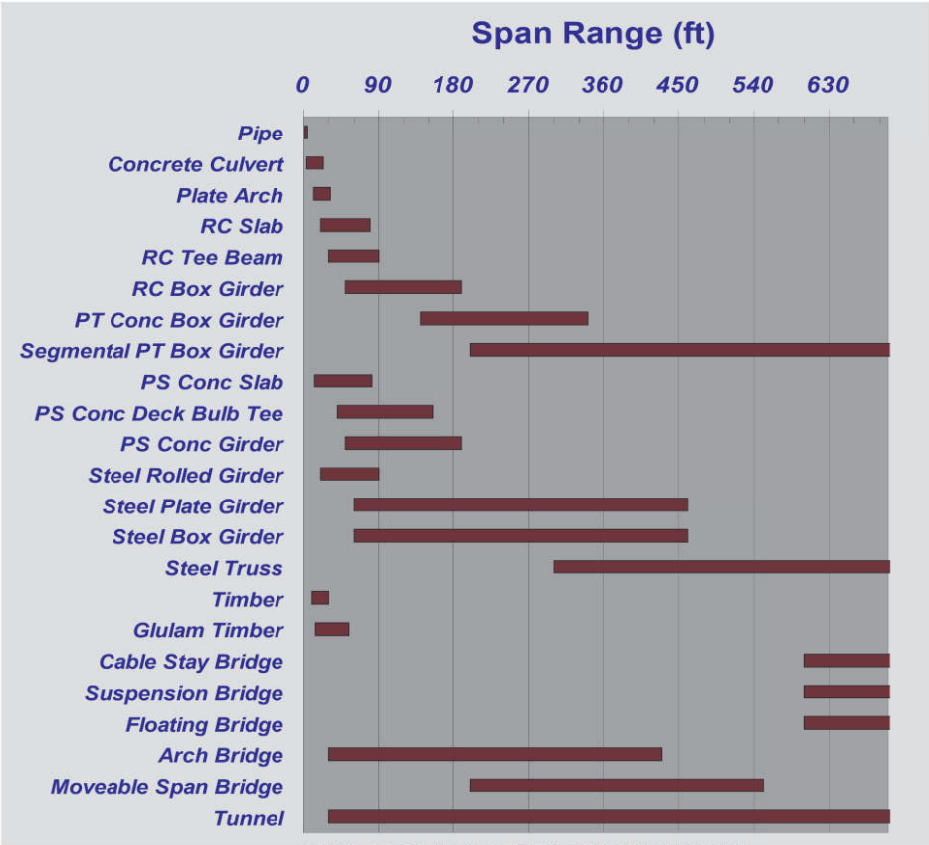
Common Concrete Bridge Types

- Slab
- I-Girder
- Box Girder
- Segmental
- U-Girder
- Spliced Girder
- Arch
- Cable-Stayed

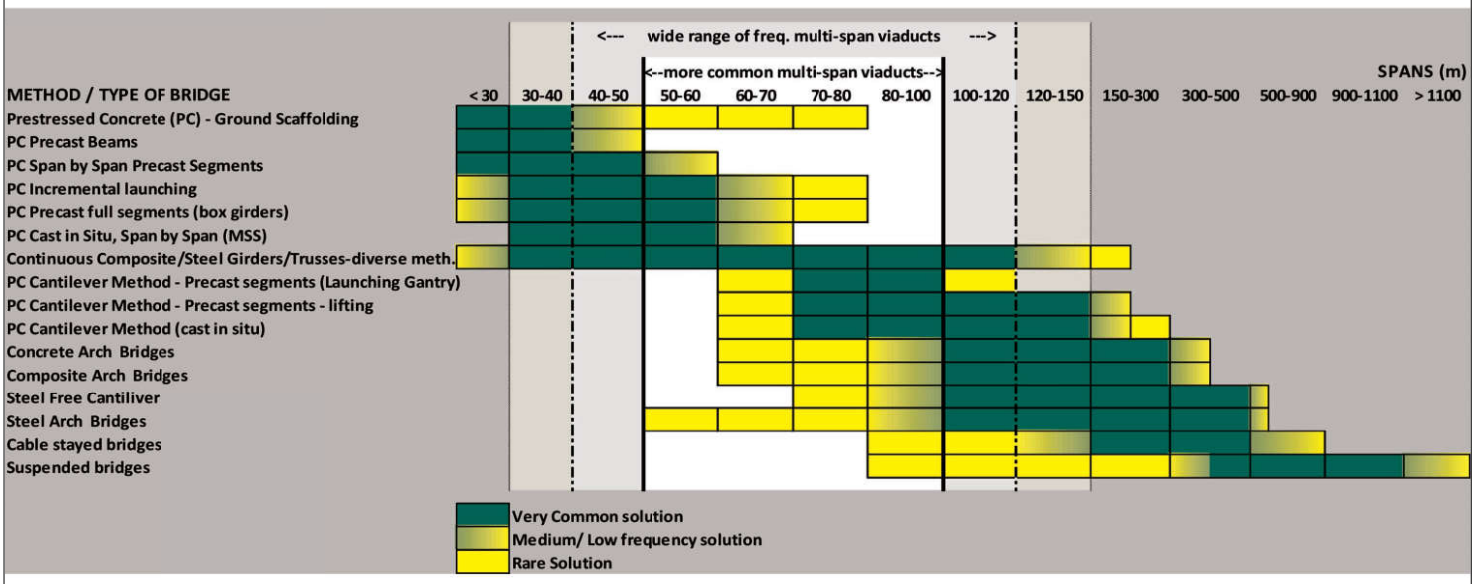
Preliminary Design (most common shapes)

- CIP Short Span Bridges
  - »Slab
  - »T-Beam
- Precast, Prestressed
  - »Standard AASHTO/PCI/ Girders
    - – I-Girders
    - – Box Girders

Preliminary Selection Guide (WSDOT)



Preliminary Selection Guide (Portugal)



Slab Bridges

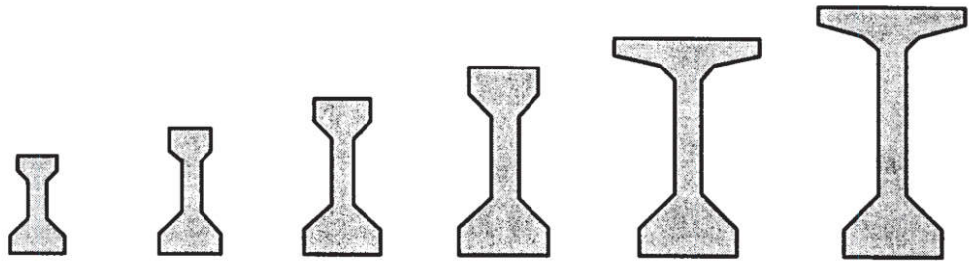
- Simple, easy to construct
- Well-suited for spans up to about 15 m.
- Cast-in-place or precast. Reinforced or prestressed.
- Can be made continuous with abutments and piers to mobilize frame action.

I-Girder Bridges

- Most popular bridge type in the USA, also used in Bangladesh.
- For spans up to about 55 m.
- Common sizes: AASHTO/PCI Type I-VI (710 mm to 1.8 m), Bulb-Tee (1.4 m, 1.6 m, and 1.8 m).

Walnut Lane Bridge, Philadelphia, PA

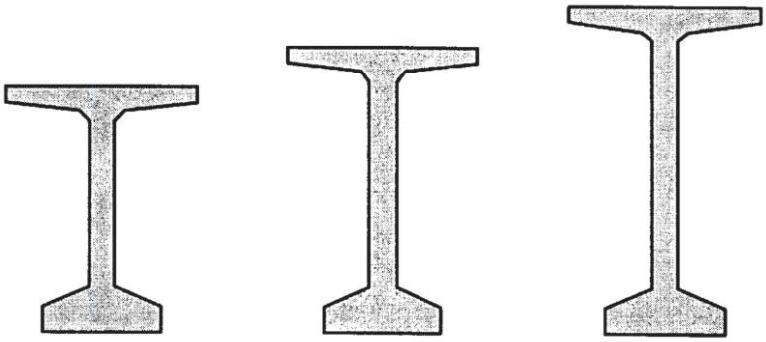
Properties, Dimensions and Maximum Spans for AASHTO-PCI I-Girders



Type I Type II Type III Type IV Type V Type VI

Beam Properties and Basic Dimensions							
					Width		
Type	Area	Centroid to Btm	Moment of Inertia	Height	Top Flange	Web	Bottom Flange
	(in. ^{2})	(in.)	(in. ^{4})	(in.)	(in.)	(in.)	(in.)
I	276	12.59	22,750	28	12	6	16
II	369	15.83	50,980	36	12	6	18
III	560	20.27	125,390	45	16	7	22
IV	789	24.73	260,730	54	20	8	26
V	1,013	31.96	521,180	63	42	8	28
VI	1,085	36.38	733,320	72	42	8	28

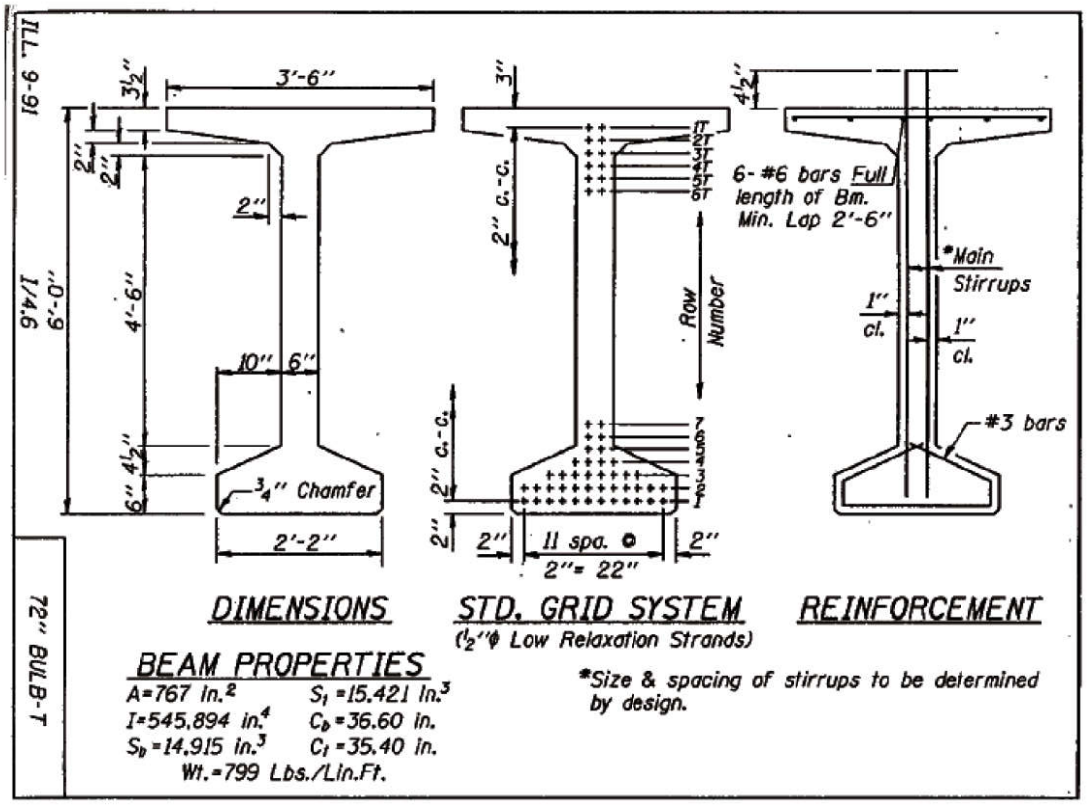
Properties, Dimensions and Maximum Spans for PCI Bulb Tee Girders



BT-54 BT-63 BT-72

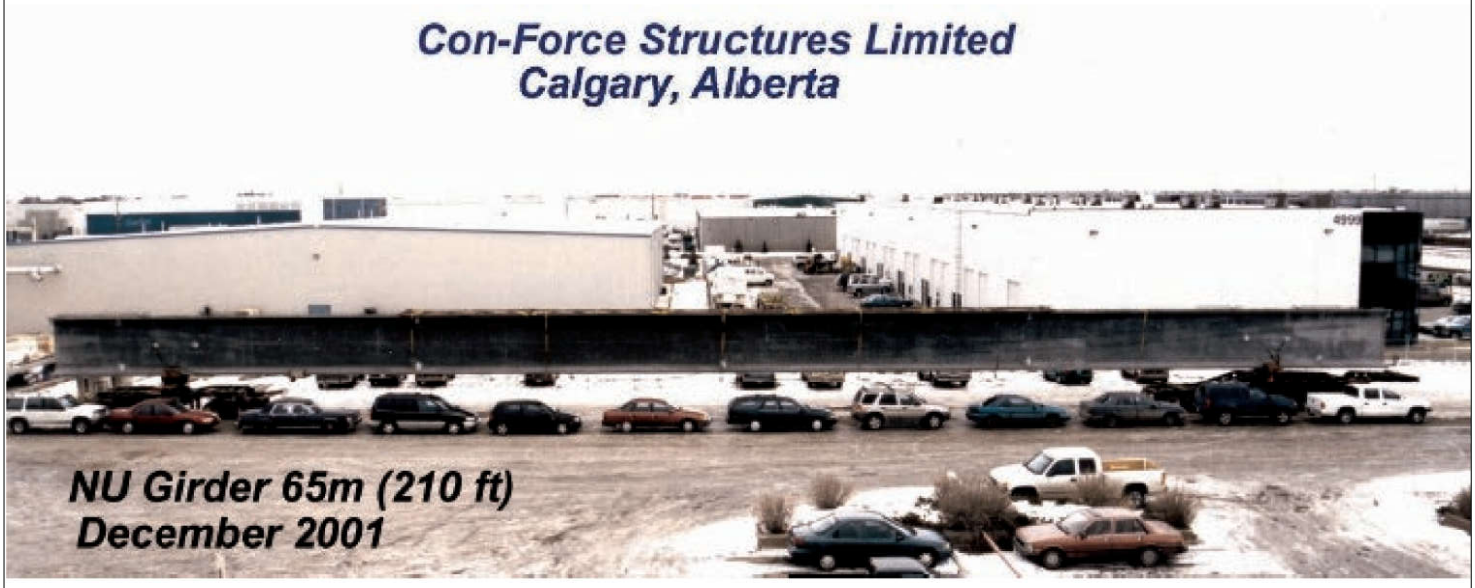
Beam Properties and Basic Dimensions							
					Width		
Type	Area	Centroid to Btm	Moment of Inertia	Height	Top Flange	Web	Bottom Flange
	(in. ^{2})	(in.)	(in. ^{4})	(in.)	(in.)	(in.)	(in.)
BT-54	659	27.63	268,077	54	42	6	26
BT-63	713	32.12	392,638	63	42	6	26
BT-72	767	36.60	545,894	72	42	6	26

Properties, Dimensions and Maximum Spans for PCI Bulb Tee Girders



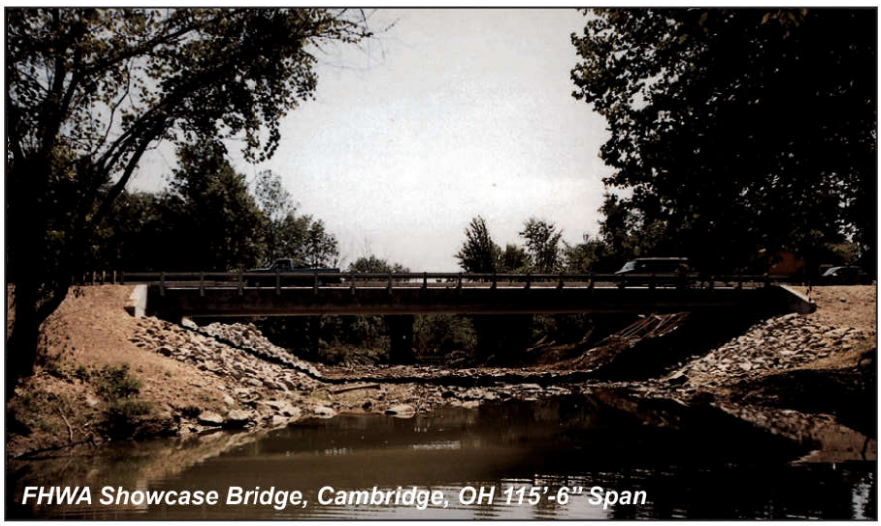
Bow River Bridge, Calgary, Alberta

Bow River Bridge, Calgary, Alberta (transportation could be a challenge for very long girders)

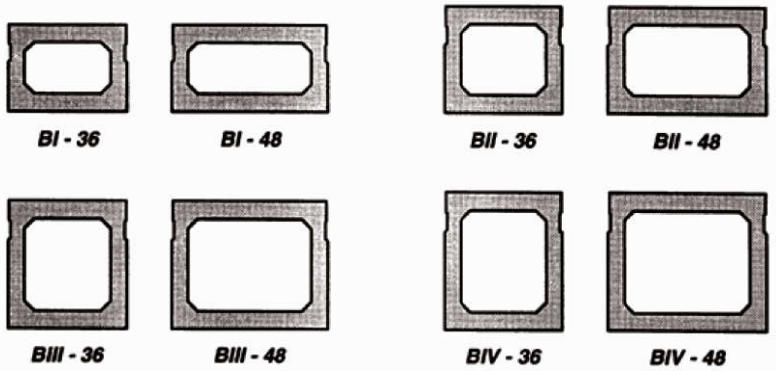


Box Girder Bridges

- Second-most popular after the I-girder bridges
- Common sizes: AASHTO/PCI Type BI-BIV
- Span Range: 18 m – 32 m.
- Use of side-by-side boxes without a wearing course offers speedy construction


FHWA Showcase Bridge, Cambridge, OH 115'-6" Span

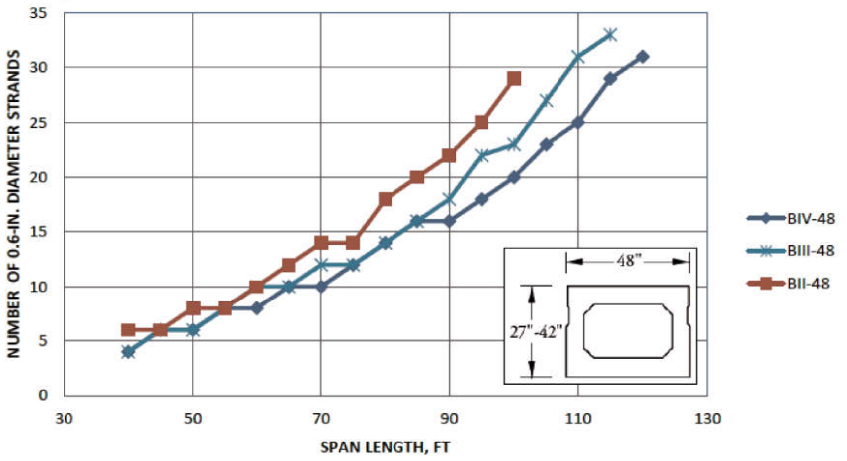
Properties, Dimensions and Maximum Spans for AASHTO - PCI Box Girders

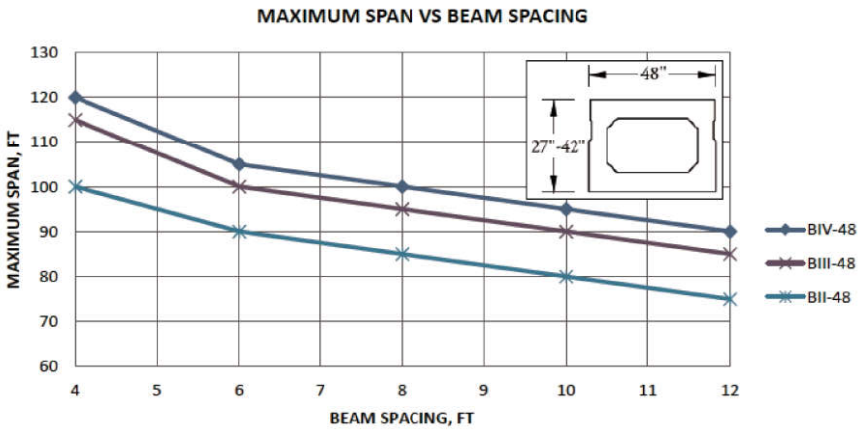


Beam Properties and Basic Dimensions

Type	Area	Centroid to Btm	Moment of Inertia	Height	Width: Top Flange	Width: Webs	Top & Bottom Flange
	(in. ^{2})	(in.)	(in. ^{4})	(in.)	(in.)	(in.)	(in.)
BI-36	560.5	13.35	50,334	27	36	5	5.5
BI-48	692.5	13.37	65,941	27	48	5	5.5
BII-36	620.5	16.29	85,153	33	36	5	5.5
BII-48	752.5	16.33	110,499	33	48	5	5.5
BIII-36	680.5	19.25	131,145	39	36	5	5.5
BIII-48	812.5	19.29	168,367	39	48	5	5.5
BIV-36	710.5	20.73	158,644	42	36	5	5.5
BIV-48	842.5	20.78	203,088	42	48	5	5.5

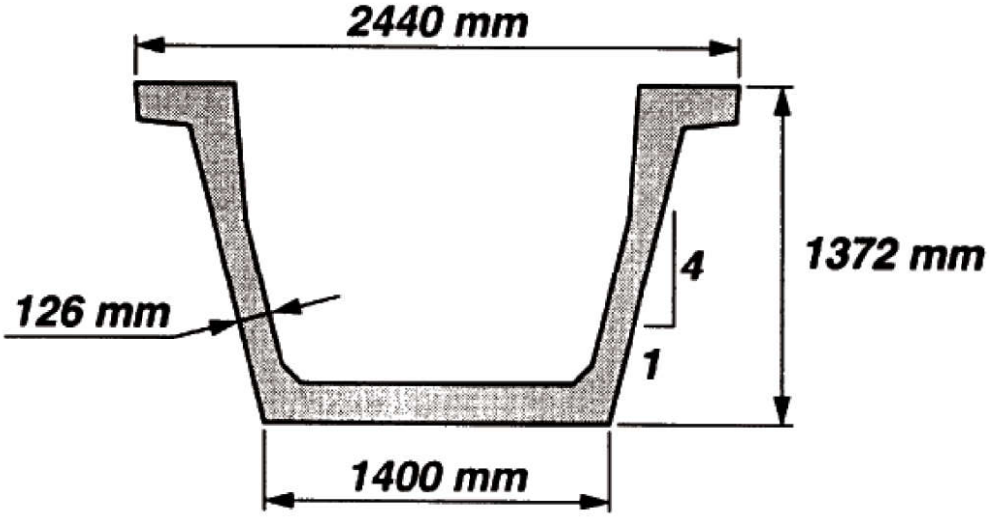
Chart BB-2
AASHTO Adjacent Box Beams 48 in. Wide





U-Girder Bridges

U-Girder Bridges



- Closed cellular section.
- Good strength-weight ratio.
- Combination of box-girder and double-tee beam.

U-Girder Bridges

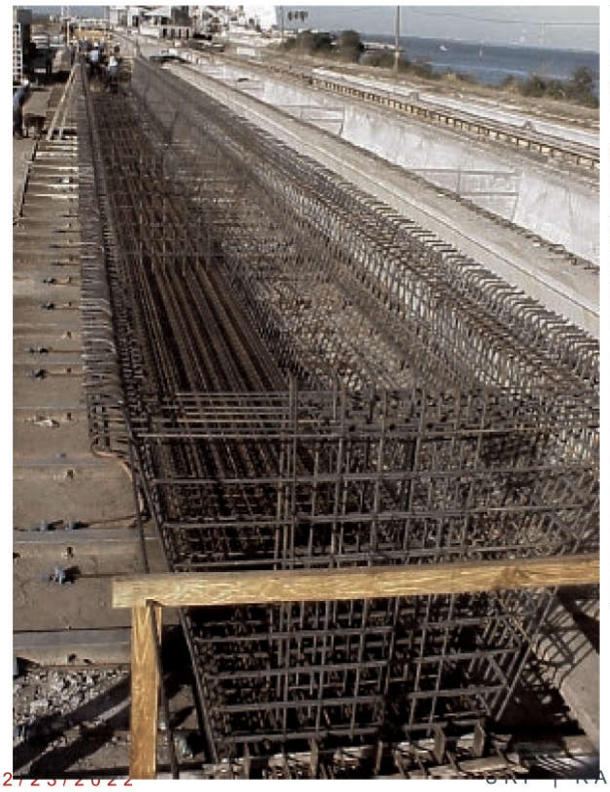


Chart U-1
U-Beams

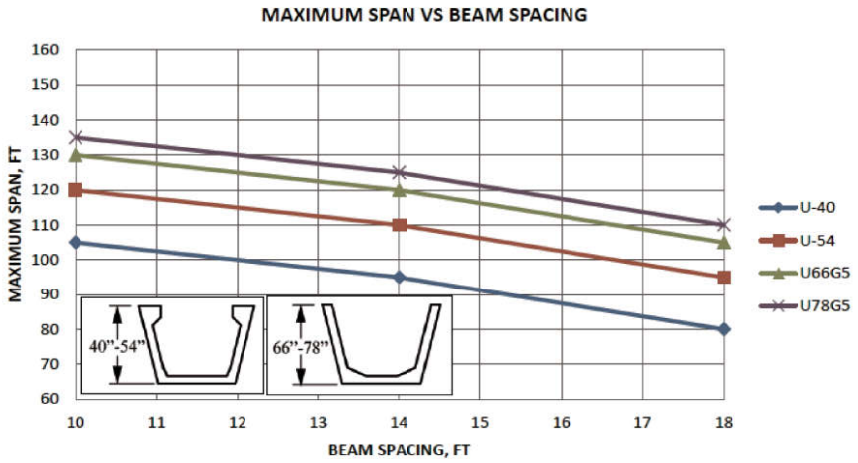
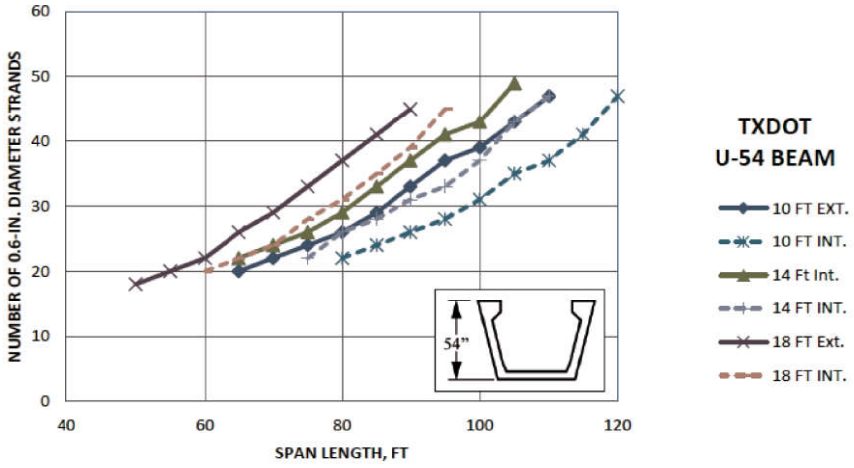


Chart U-3
Texas U-54 Beams



Louetta Road Bridge Deck Panels

Louetta Road Bridge

Chart IB-1
AASHTO I-Beams

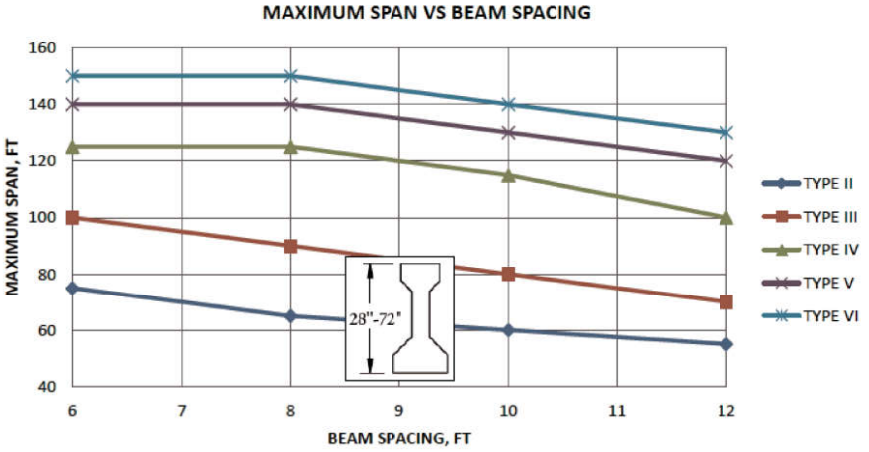
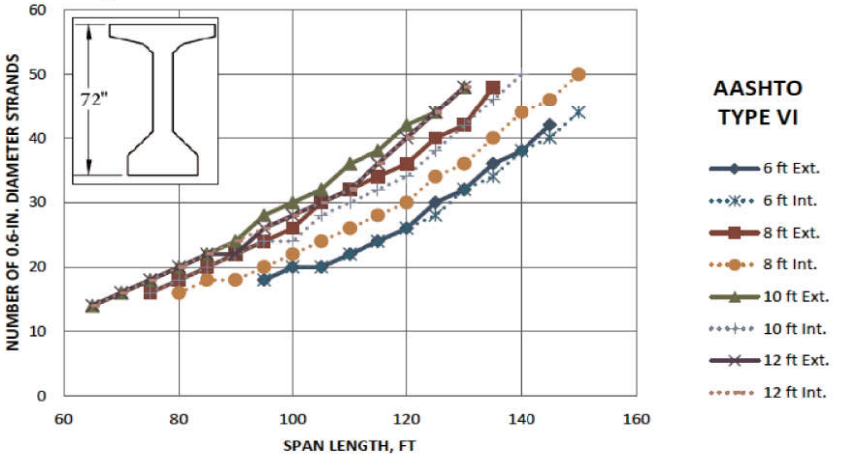


Chart IB-6
AASHTO I-Beams Type VI



Design a simple span bridge for HL-93 loading with 120 ft. span. The width is 51 ft. with a 6 in. thick cast-in-place deck

Product Options for Example No. 2[1]

Products		Depth in.	Spacing ft	Deck Thickness in.	Number of Strands	Design Chart
AASHTO I-Beams	I V	54	8	8.0	42	IB-4
		54	6	8.0	36	IB-4
	V	63	12	9.0	46	IB-5
		63	10	8.0	48	IB-5
	VI	63	8	8.0	42	IB-5
		63	6	8.0	32	IB-5
		72	12	9.0	40	IB-6
		72	10	8.0	42	IB-6
		72	8	8.0	36	IB-6
		72	6	8.0	26	IB-6
AASHTO-PCI Bulb-Tees	BT-54	54	6	8.0	34	BT-2
	BT-63	63	6	8.0	28	BT-3
	BT-72	72	6	8.0	24	BT-4
		72	8	8.0	34	BT-4
		72	10	8.0	38	BT-4
		72	12	9.0	36	BT-4
Deck Bulb-Tees 6-ft-Wide Flange		53	6	None	30	DBT-2
		65	6	None	23	DBT-2
AASHTO Box Beams	BIV-36	39	3	6.0	27	BB-7
	BIV-48	42	4	6.0	31	BB-5
Washington U-Beams	U66G5	66	10	8.0	47	U-4
	U78G5	78	14	8.0	49	U-5
		78	10	8.0	43	U-5

Preliminary Design Hints

- Beneficial to use the widest possible spacing to minimize the number of girder lines.
- Clearance requirements may dictate the structure depth.
- If no depth limitations, the most economical will be the deepest. Minimize number of strands.

Girder Options	72 in. AASHTO Type VI	72 in. AASHTO-PCI Bulb-Tee	AASHTO Box BIV-48	U-Beam U78G5
Spacing (ft.)	8	8	4	10
Number of girders	7	7	13	5
Girder depth (in.)	72	72	42	66
Cross section area in^2)	1085	767	842	767
Number of strands	36	34	31	47

Segmental Bridges

- ✓ The first long-span segmental concrete bridge in the U.S. was built over the Gulf-Intracoastal Waterway in Texas in 1973.
- ✓ Over 460 segmental bridges have been built in the U.S.
- ✓ **M**aximum span length: 760 ft. for segmental box girder and 1300 ft. for cable-stayed bridges.
- ✓ Advantages**: r**epetitive & economical construction, durable structure, can be placed on small footprint**, can** be used on a small radius**,** aesthetically pleasing, concrete can be colored to blend into the environment.

Segmental Bridges

- ✓ Precast segments are usually erected using the span-by-span, balanced cantilever, or progressive placement methods.

Precast, Span-by-Span Method

Precast, balanced cantilever method

Segmental Bridges

Precast, progressive placement method

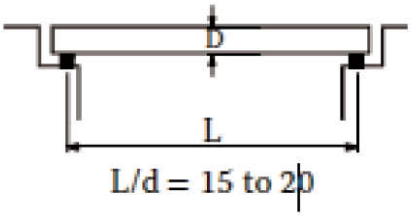
Cast-in-place, balanced cantilever method

Segmental Bridges

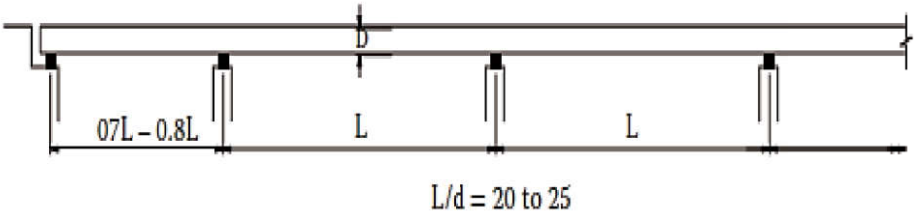
- ✓ AASHTO LRFD adopted segmental bridge design provisions from AASHTO Guide Specifications for Design and Construction of Segmental Bridges.
- ✓ ***Considerations during preliminary*** design stage:
  - Span arrangement and configuration should be closely studied by considering ***the*** location of the bridge.
  - A bridge crossing over a navigable waterway is very much dictated by the horizontal and vertical clearance requirements.
  - Uniformity of the span lengths is critical in order to maximize the benefit of precasting the segments. Uniform spans produce economical design.
  - For spans up to 250 ft. , constant depth section is typically utilized.
  - For span length greater than 250 ft., a variable depth section is more economical and efficient.
  - **Span length over depth ratio (L/D) plays an important role in preliminary design.**

Segmental Bridges

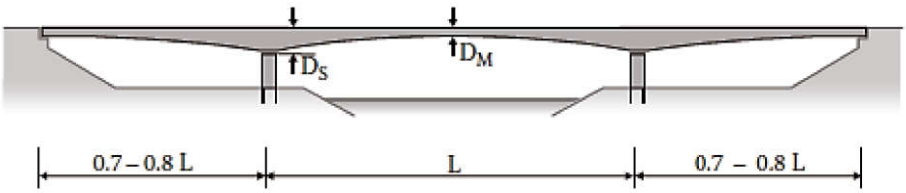
- The preliminary superstructure depth is selected on the basis of the *L/D* ratio.
- The initial depth selected is continuously refined in the preliminary and final design.



Simply supported girder



Continuous girder

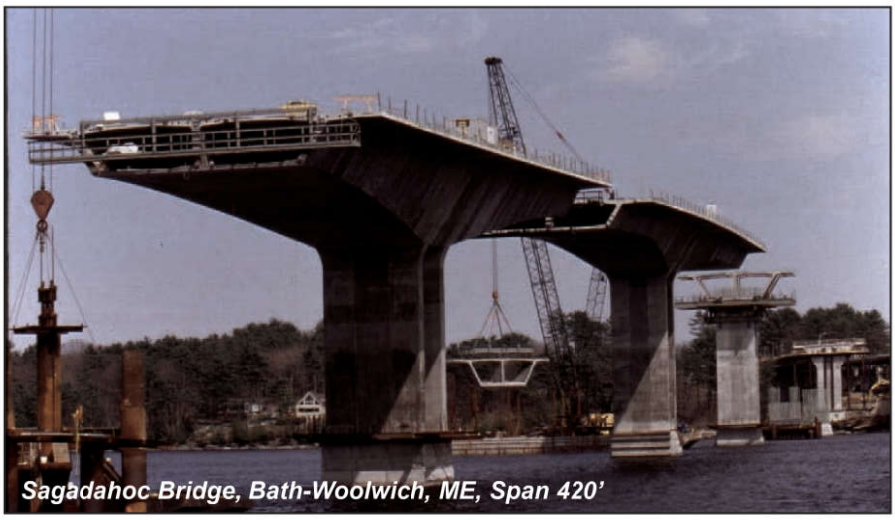


Span/Depth ratio: L/D_S = 15 to 18
L/D_M = 35 to 45

Variable-depth bridge girder (for span ≥ 250 ft.)

Segmental Bridges

- Economical, durable, aesthetically pleasing
- Post-tensioned or/and cable-stayed
- Typical segment type: Concrete box
- Cast-in-place (CIP) or precast
- Perfectly suited for gradual and sharply curved alignments


Sagadahoc Bridge, Bath-Woolwich, ME, Span 420'

Hanging Lake Viaduct

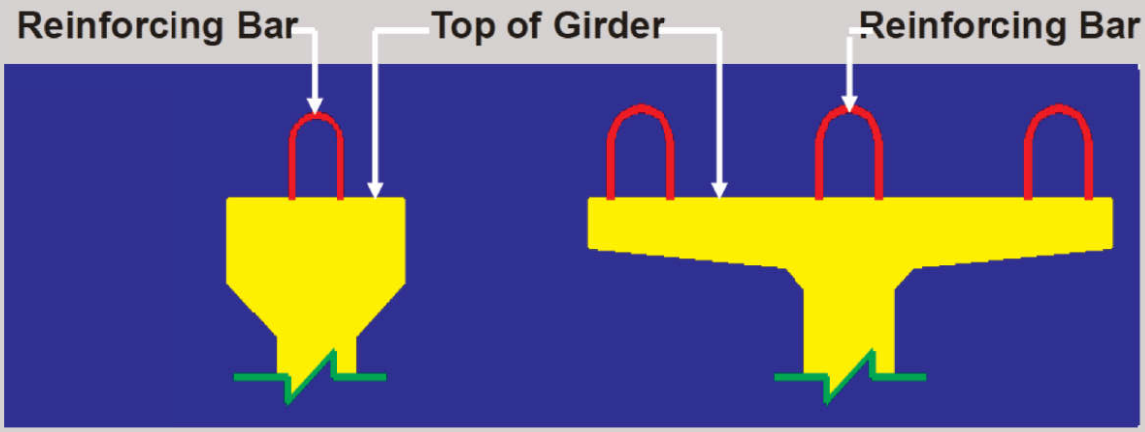
- I-70 Highway in Glenwood, Colorado
- Segmental precast concrete bridge
- Balanced Cantilever construction

Spliced Girder Bridges

- Innovative technique for very long spans
- Long-segment precast prestressed girders spliced
- Spans of more than 91 m have been achieved

Spliced Girder Bridges

Example 2- Ease of Construction AASHTO Girders vs. PCEF Bulb Tee



Top of AASHTO IV Girder

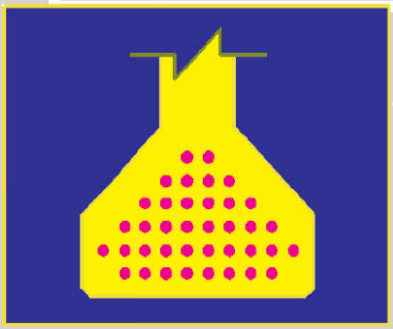
1'-8" Wide Balance Beam

Top of PCEF Bulb Tee

4'-0" Wide Runway

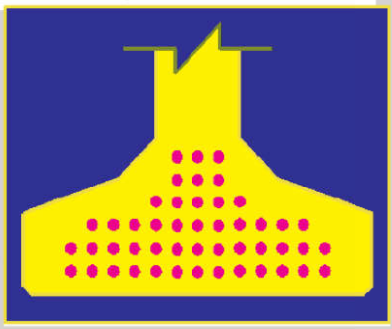
Comparison of Section Properties

Type of Girder	AASHTO III	45" Bulb Tee
Girder Depth	45 inches	45 inches
Moment of Inertia	125,390 in^4	215,151 in^4
Maximum Strands Bottom Flange	38 strands	48 strands



AASHTO III Bottom Flange

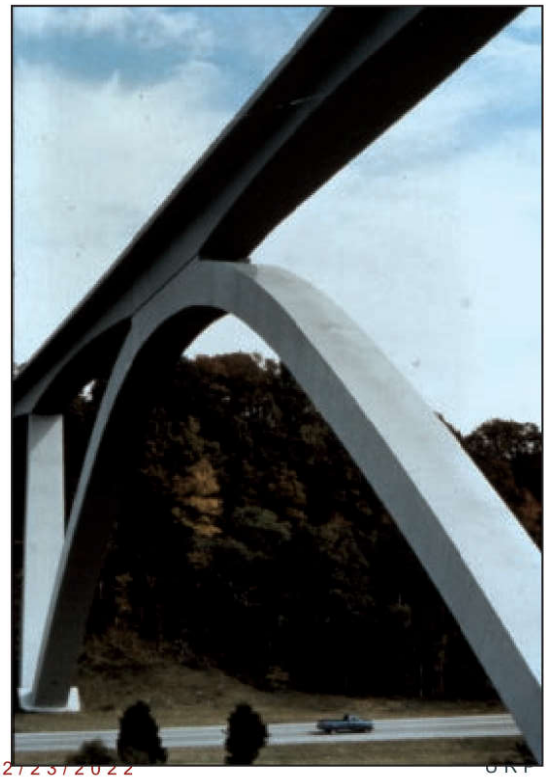
PCEF Bulb Tee Bottom Flange



Why Prestressed Concrete Girders?

- Steel girders need paint!
- Concrete girders are generally maintenance free.
- Concrete over water, steel over highways
- Difficult to paint girders over water.
- Steel girders more resilient to vehicle hits.

Arch Bridges



- Most efficient shape for supporting gravity loading
- CIP or precast
- Longest existing concrete arch bridge: Wanxian Bridge, China. Span = 420 m.

Cable-Stayed Bridges

- Structurally efficient use of materials.
- Concrete in compression and steel stays in tension.
- Economical and aesthetically pleasing.
- Most popular type for signature bridges.
- Longest concrete cable-stayed bridge in the U.S.: Dames Point, Jacksonville, Fl. Main Span = 396 m.

Zakim Bunker Hill Bridge, Boston, MA

Over Charles River.
Part of the Central Artery Project.

Design Considerations

- Engineers tend to use familiar designs that worked in the past.
- Clients' desires, architects' design, relevant codes, accepted practice, available materials, contractors' capabilities, economics, environmental concerns, legal factors, political factors.
- Data gathering - topography, function, soil condition, available materials, hydrology, etc.
- Preliminary design
- Design - Build option

Bridge Aesthetics Hints

- About 2% additional cost for short bridges; 5% for longer bridges.
- Equal spans for deep valleys. Odd span numbers.
- Relative proportion of piers and girders.
- Limit pier width to about 1/8 of span length.
- Use order and rhythm.
- Contrast and texture.
- Light and shadow.

Evolution of BRIDGE Design Methodologies

3. LRFD Methodology

Load and Resistance Factor Design

» Recognizes Variability in Loads and Resistances

- Loads Multiplied by a Load Factor → Increases Loads
- Strengths (Resistances) Multiplied by a Resistance Factor → Decreases Resistances

» Consistent Reliability Index, β, at Strength Limit State

» Calibrated Load and Resistance Factors

- 1.25D + 1.75(L+I)

BRIDGE SPECIFICATIONS: USA

- All Public Bridges in the USA must be designed in conformance with the AASHTO Specifications (with any local additions).

AASHTO
THE VOICE OF TRANSPORTATION

- AASHTO ?

American Association of State Highway and Transportation Officials

AASHTO Bridge Design Specifications

- Standard Specifications for Highway Bridges, 17th Edition, 2002—phased out in 2007 (ASD).
- All new bridges for which States initiate preliminary engineering after October 1, 2007, shall be designed by the LRFD Specifications.
- For modifications to existing structures, use LRFD Specifications or the specifications originally used.

Other Manuals, Guides, Guide Specifications

- AASHTO LRFD Bridge Construction Specifications.
- AASHTO Guide Specifications for LRFD Seismic Bridge Design.
- AASHTO Guide Specification for Seismic Isolation Design.
- AASHTO Guide Specifications for Bridges Vulnerable to Coast Storms.
- AASHTO Guide Specifications and Commentary for Vessel Collision Design of Highway Bridges.

Other Manuals, Guides, Guide Specifications

- AASHTO LRFD Moveable Highway Bridge Design Specifications.
- AASHTO LRFD Guide Specifications for Design of Pedestrian Bridges.
- AASHTO LRFD Bridge Design Guide Specifications for GFRP-Reinforced Concrete Bridge Decks and Traffic Railings.
- AASHTO Technical Manual for Design and Construction of Road Tunnels—Civil Elements.
- AASHTO Manual for Bridge Evaluation.

Other Manuals, Guides, Guide Specifications

- AASHTO Bridge Element Inspection Manual.
- Bridge Aesthetics Sourcebook.
- Construction Handbook for Bridge Temporary Works.
- Guide Specifications for Bridge Temporary Works.
- AASHTO Standard Specifications for Transportation Materials and Methods of Sampling and Testing.
- AREMA Manual for Railway Engineering (by American Railway Engineering and Maintenance-of- Way Association [AREMA]).

Objective of the LRFD

Develop a comprehensive and consistent LRFD specification that is calibrated to obtain uniform reliability (a measure of safety) at the strength limit state.

AASHTO addresses the following limit states in the design process:

- Service Limit State
- Fatigue and Fracture Limit State
- Strength Limit State
- Extreme Event Limit State

Definitions

Q = Load

γ = Load Factor

R = Resistance

φ = Resistance Factor

β = Target Reliability Index

For Safe Design

The basic requirement,
for each limit state:

$$\Sigma \gamma_i Q_i \leq \phi R$$

Load Resistance

Basis of LRFD Methodology

$$\Sigma \eta_i \gamma_i Q_i \leq \phi R_n$$ (Eq. 1.3.2.1-1, page 1-3)

For loads where max. value of γ_i is used:

$$\eta_i = \eta_D \eta_R \eta_I \geq 0.95$$

For loads where min. value of γ_i is used:

$$\eta_i = 1 / (\eta_D \eta_R \eta_I) \leq 1.00$$

→ η_i = load modifier

where:

γ_i = load factor: a statistically based multiplier applied to force effects

ϕ = resistance factor: a statistically based multiplier applied to nominal resistance, as specified in Sections 5, 6, 7, 8, 10, 11, and 12

η_i = load modifier: a factor relating to ductility, redundancy, and operational classification

η_D = a factor relating to ductility, as specified in Article 1.3.3

η_R = a factor relating to redundancy as specified in Article 1.3.4

η_I = a factor relating to operational classification as specified in Article 1.3.5

Q_i = force effect

R_n = nominal resistance

R_r = factored resistance: ϕR_n

Load Modifier, η_i

(1.3.3 – 1.3.5)

η_D = ductility factor, Strength Limit State
= 1.05 for non-ductile components
= 1.00 for conventional designs
= 0.95 for additional ductile components
= 1.0 for all other limit states

η_R = redundancy factor, Strength Limit State
= 1.05 for non-redundant members
= 1.0 for conventional design
= 0.95 exceptional levels of redundancy beyond continuity and closed sections
= 1.0 for all other limit states

Load Modifier, η_i

η_I = operational importance factor, Strength Limit State
= 1.05 for critical/essential bridges
= 1.00 for typical bridges
= 0.95 for less important bridges
= 1.0 for all other limit states

Ductility Factor, η_D

Resistance Factors, ϕ

5.5.4.2 (page 5-29)

Tension-controlled sections – RC	0.90
Tension-controlled sections – bonded strands, prestressed	1.00
Tension-controlled sections – unbonded strands, post-tensioned	0.9
Compression-controlled sections	0.75
Shear and torsion – normal weight concrete	0.90
Shear and torsion – lightweight concrete	0.80
Bearing	0.70
Compression - Strut & Tie Model	0.70

Unified Design Provisions for Reinforced and Prestressed Concrete

- ***Emphasize common features***
- ***Eliminate duplication***
- ***Unify design procedures***
- ***Promote the idea of "Structural Concrete"***

Example:

$$M_n = A_{ps} f_{ps}(d_p - a/2) + A_s f_s(d_s - a/2) - A'_s f'_s(d'_s - a/2) + 0.85 f'_c (b - b_w) h_f (a/2 - h_f/2)$$

R.C. case can be obtained by setting the P.S. term to zero!

AASHTO LRFD Bridge Design Specifications - Chapters

1. Introduction
2. General Design and Location Features
3. Loads and Load Factors
4. Structural Analysis and Evaluation
5. Concrete Structures
6. Steel Structures
7. Aluminum Structures

AASHTO LRFD Bridge Design Specifications - Chapters

8. Wood Structures
9. Decks and Deck Systems
10. Foundations
11. Abutments, Piers, and Walls
12. Buried Structures and Tunnel Liners
13. Railings
14. Joints and Bearings
15. Design of Sound Barriers

Questions?

Loads and Flexural Design of Bridges (AASHTO LRFD 2017)

Date: 27 February 2022
Session 40 – Bridges 02

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S-8A SEng PRP Training Program

02/28/2022 URP | RAJUK | S-8 COMPONENT

1

AASHTO LRFD Limit States

- The AASHTO LRFD Specifications require consideration of several load combinations corresponding to the following limit states (Chapter 3):
  - STRENGTH LIMIT STATES (strength and stability)
  - SERVICE LIMIT STATES (stress, deformation, and cracking)
  - FATIGUE & FRACTURE LIMIT STATES (stress range)
  - EXTREME EVENT LIMIT STATES (earthquakes, wind, vehicle and vessel collision, among others)

2

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Load and Load Designations (3.4.1)

STRENGTH I :	normal vehicular use without wind
STRENGTH II :	owner special design/evaluation, permit vehicles without wind
STRENGTH III :	bridge exposed to design wind speed at the location (50 year, 7% chance)
STRENGTH IV :	very high dead-to-live load ratios (heavy bridges)
STRENGTH V :	normal vehicular use with 129 kph (80 mph) wind

3

Load and Load Designations (3.4.1)

SERVICE I:	normal operational use of the bridge with 88 kph (55 mph) wind and nominal loads. Control cracking of reinforced concrete structures.
SERVICE II:	control yielding of steel structures and slip of connections due to live load.
SERVICE III:	control cracking in prestressed concrete superstructures
SERVICE IV:	control cracking in prestressed concrete substructures

SERVICE III & IV only apply to TENSION in PS members

4

Load and Load Designations (3.3.2, page 3-8)

DD =	down drag
DC =	dead load of structural components and nonstructural attachments
DW =	dead load of wearing surfaces and utilities
EH =	horizontal earth pressure
EL =	accumulated locked-in force effects resulting from the construction process, including the secondary forces from post-tensioning
ES =	earth surcharge load
EV =	earth fill vertical pressure
BR =	vehicular braking force
CE =	vehicular centrifugal force
CR =	creep
CT =	vehicular collision force
CV =	vessel collision force

EQ =	earthquake
BL =	blast load
FR =	friction
IC =	ice load
IM =	vehicular dynamic load allowance
LL =	vehicular live load
LS =	live load surcharge
PL =	pedestrian live load
SE =	settlement
SH =	shrinkage
TG =	temperature gradient
TU =	uniform temperature
WA =	water load and stream pressure
WL=	wind load on live load
WS=	wind load on structure

5

Other Loads?

6

Other Loads?

Identify possible loads from AASHTO LRFD

Bridge Location

7

Load Combinations and Load Factors (Table 3.4.1-1, page 3-14)

Has a range

Load Combination / Limit State	DC DD DW EH EV ES	LL IM CE BR PL LS	WA	WS	WL	FR	TU CR SH	TG	SE	Use One of These at a Time: EQ	IC	CT	CV
STRENGTH-I	γ_p	1.75	1.00	-	-	1.00	0.50/1.20	γ_{TG}	γ_{SE}	-	-	-	-
STRENGTH-II	γ_p	1.35	1.00	-	-	1.00	0.50/1.20	γ_{TG}	γ_{SE}	-	-	-	-
STRENGTH-III	γ_p	-	1.00	1.40	-	1.00	0.50/1.20	γ_{TG}	γ_{SE}	-	-	-	-
STRENGTH-IV EH, EV, ES, DW DC ONLY	γ_p 1.5	-	1.00	-	-	1.00	0.50/1.20	-	-	-	-	-	-
STRENGTH-V	γ_p	1.35	1.00	0.40	0.40	1.00	0.50/1.20	γ_{TG}	γ_{SE}	-	-	-	-
EXTREME-I	γ_p	γ_{EQ}	1.00	-	-	1.00	-	-	-	1.00	-	-	-
EXTREME-II	γ_p	0.50	1.00	-	-	1.00	-	-	-	-	1.00	1.00	1.00
SERVICE-I	1.00	1.00	1.00	0.30	0.30	1.00	1.00/1.20	γ_{TG}	γ_{SE}	-	-	-	-
SERVICE-II	1.00	1.30	1.00	-	-	1.00	1.00/1.20	-	-	-	-	-	-
SERVICE-III	1.00	0.80	1.00	-	-	1.00	1.00/1.20	γ_{TG}	γ_{SE}	-	-	-	-
FATIGUE-LL, IM & CE ONLY	-	0.75	-	-	-	-	-	-	-	-	-	-	-

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Load Combination Factors (Table 3.4.1-1)

Load Combination / Limit State	DC DD DW EH EV ES EL	LL IM CE BR PL LS	Use One of These at a Time: IC	CT	CV
STRENGTH-I	γ_p	1.75	–	–	–
SERVICE-I	1.00	1.00	1.00	1.00	1.00
SERVICE-III	1.00	0.80	–	–	–
FATIGUE-LL, IM & CE ONLY - I	–	[redacted]	–	–	–
FATIGUE-LL, IM & CE ONLY - II	–	0.75	–	–	–

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Load Factors for Permanent Loads, γ_p (Table 3.4.1-2)

Range of load factors for dead loads must be considered

Type of Load	Load Factor: Maximum	Load Factor: Minimum
DC: Component and Attachments	1.25	0.90
DD: Downdrag	1.80	0.45
DW: Wearing Surfaces and Utilities	1.50	0.65
• Rigid Buried Structure	1.30	0.90
• Rigid Frames	1.35	0.90
• Flexible Buried Structures other than Metal Box Culverts	1.95	0.90
• Flexible Metal Box Culverts	1.50	0.90
ES: Earth Surcharge	1.50	0.75

Only the two extremes needed

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Resistance Factors (5.5.4.2, p. 5-26)

	Standard Specs	LRFD 5.5.4.2
Flexure – RC	0.90	0.90
Flexure – PS	1.00	1.00
Shear – RC	0.85	0.90
Shear – PS	0.90	0.90
Compression	0.70 / 0.75	0.75
Bearing	0.70	0.70

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Load Factors, Service-I and Service-III Load Combinations

	Standardd Specs	LRFD Specs
DC	1.0	1.0
DW	1.0	1.0
LL (I)	1.0	1.0
LL (III)	1.0	0.8

where:

LL (I) = Live load → Service-I

LL (III) = Live load → Service-III for "tension in prestressed concrete structures with the objective of crack control"

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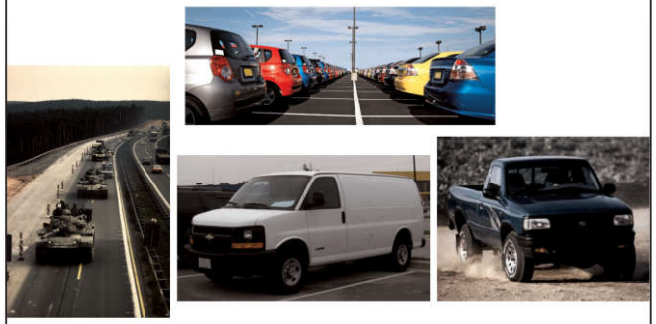
Bridge Load Types



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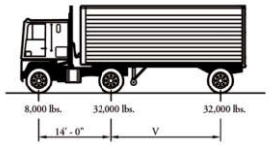
Design Vehicular Live Loads (3.6.1.2.1, p. 3-19)



Design Truck (6 ft. wide)

Design Tandem (6 ft. wide)

Design Lane Load (10 ft. wide)

Tandem: heavy truck with short wheels

Total Vehicular LL
(HL-93)

Design Truck
OR
Design Tandem
+
Design Lane Load

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Bridge Load Types

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Bridge Load Types

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Dynamic Load Allowance (3.6.2.1, p. 3-31)

The dynamic load allowance in Table 3.6.2.1-1 is an increment to be applied to the static wheel load to account for wheel load impact from moving vehicles.

Table 3.6.2.1-1 Dynamic Load Allowance, *IM*.

Component	IM
Deck Joints—All Limit States	75%
All Other Components	
• Fatigue and Fracture Limit State	15%
• All Other Limit States	33%

Sources of dynamic effects on bridges

- Hammering at surface discontinuities
- Dynamic response of bridge as a whole

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Dynamic Load Allowance (3.6.2.1) (Impact)

For design of most bridge components for all limit states except fatigue

- The LRFD Specifications simply require a constant magnification (IM) of 33% to be applied to the design truck or design tandem only
- The magnification (IM) is not applied to the design lane load
- This simple approach is based on a study that found the most influential factor affecting dynamic impact is roadway surface roughness
- Commentary has more background

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Live Load Distribution Factors to Girders

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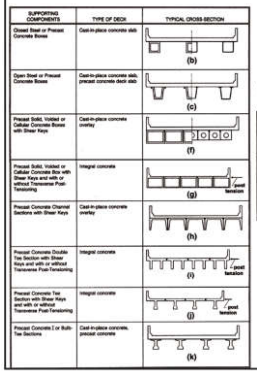
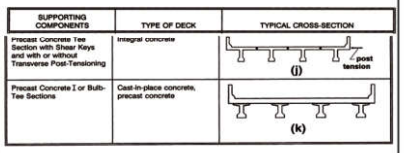
Common Superstructures (Table 4.6.2.2.1-1, p. 4-33)



SUPPORTING COMPONENTS	TYPE OF DECK	TYPICAL CROSS-SECTION
Closed Steel or Precast Concrete Boxes	Cast-in-place concrete slab	(b)
Open Steel or Precast Concrete Boxes	Cast-in-place concrete slab, precast concrete deck slab	(c)
Precast Solid, Voided or Cellular Concrete Boxes with Shear Keys	Cast-in-place concrete overlay	(f)
Precast Solid, Voided or Cellular Concrete Box with Shear Keys and with or without Transverse Post-Tensioning	Integral concrete	(g) post tension
Precast Concrete Channel Sections with Shear Keys	Cast-in-place concrete overlay	(h)
Precast Concrete Double Tee Section with Shear Keys and with or without Transverse Post-Tensioning	Integral concrete	(i) post tension
Precast Concrete Tee Section with Shear Keys and with or without Transverse Post-Tensioning	Integral concrete	(j) post tension
Precast Concrete I or Bulb-Tee Sections	Cast-in-place concrete, precast concrete	(k)



SUPPORTING COMPONENTS	TYPE OF DECK	TYPICAL CROSS-SECTION
Precast Concrete Tee Section with Shear Keys and with or without Transverse Post-Tensioning	Integral concrete	(j) post tension
Precast Concrete I or Bulb-Tee Sections	Cast-in-place concrete, precast concrete	(k)

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Simplified Distribution Factors (4.6.2.2.1, p. 4-29)

To use the simplified distribution factors, the following conditions must be met:

- Width of deck is constant
- Number of beams, $N_b \geq 4$
- Beams are parallel and of the same stiffness
- The roadway part of the overhang, $d_e \leq 3.0$ ft.
- Curvature is less than limit in 4.6.1.2.4
- Section appears in Table 4.6.2.2.1-1

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Design Lanes

- Width of Design Lane = 12 ft. (LRFD 3.6.1.1, p. 3-19)
- No. of Design Lanes = w (clear roadway)/12; take the integer part of this ratio
- If traffic lane width is less than 12 ft., number of design lanes = number of traffic lanes.

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Unit Weights

- In absence of more precise information, Table 3.5.1-1 (LRFD p. 3-18) may be used.

Table 3.5.1-1—Unit Weights

Material		Unit Weight (kcf)
Aluminum Alloys		0.175
Bituminous Wearing Surfaces		0.140
Cast Iron		0.450
Cinder Filling		0.060
Compacted Sand, Silt, or Clay		0.120
Concrete	Lightweight	0.110 to 0.135
	Normal Weight with $f'_c \leq 5.0$ ksi	0.145
	Normal Weight with $5.0 < f'_c \leq 15.0$ ksi	$0.140 + 0.001 f'_c$
Loose Sand, Silt, or Gravel		0.100
Soft Clay		0.100
Rolled Gravel, Macadam, or Ballast		0.140
Steel		0.490
Stone Masonry		0.170
Wood	Hard	0.060
	Soft	0.050
Water	Fresh	0.0624
	Salt	0.0640
Item		Weight per Unit Length (klf)
Transit Rails, Ties, and Fastening per Track		0.200

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Distribution of Live Loads Per Lane, Moment in Interior Beams (Table 4.6.2.2.2b-1, p. 4-37)

Type of Superstructure	Applicable Cross-Section from Table 4.6.2.2.1-1	Distribution Factors	Range of Applicability
Concrete Deck, Filled Grid, Partially Filled Grid, or Unfilled Grid Deck Composite with Reinforced Concrete Slab on Steel or Concrete Beams; Concrete T-Beams, T- and Double T-Sections	a, e, k and also i, j if sufficiently connected to act as a unit	One Design Lane Loaded: $0.06+\left(\frac{S}{14}\right)^{0.4}\left(\frac{S}{L}\right)^{0.3}\left(\frac{K_g}{12.0Lt_s^3}\right)^{0.1}$ Two or More Design Lanes Loaded: $0.075+\left(\frac{S}{9.5}\right)^{0.6}\left(\frac{S}{L}\right)^{0.2}\left(\frac{K_g}{12.0Lt_s^3}\right)^{0.1}$	$3.5 \leq S \leq 16.0$ $4.5 \leq t_s \leq 12.0$ $20 \leq L \leq 240$ $N_b \geq 4$ $10{,}000 \leq K_g \leq 7{,}000{,}000$
		use lesser of the values obtained from the equation above with $N_b = 3$ or the lever rule	$N_b = 3$

Notes:
1) Units are in LANES and not WHEELS!
2) No multiple presence factor applied (tabulated equations)
4) May be different for positive and negative flexure locations
5) Use greater of 1 or 2 lanes loaded

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Longitudinal Stiffness Parameter (p. 4-32)

$$\left(\frac{K_g}{12.0Lt_s^3}\right)^{0.1}$$

Gives an indication of the relative stiffness between the beam (longitudinal) and deck (transverse)

For preliminary design, this term may be taken as 1.10

K_g = longitudinal stiffness parameter, in.^{4}, = $n(I + Ae_g^2)$ [LRFD Eq. 4.6.2.2.1-1]

where

n = modular ratio between beam and deck materials

$= \frac{E_c(\text{beam})}{E_c(\text{slab})}$ > 1, the inverse of ratio (n) for section properties since it is transforming beam to deck

A = cross-section area of the beam (non-composite section), in.^{2}

I = moment of inertia of the beam (non-composite section), in.^{4}

e_g = distance between the centers of gravity of the beam and deck, in.

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Distribution Factors for I-Beams – Shear (Table 4.6.2.2.3a-1, p. 4-43)

The live load distribution factor for shear for interior beams with 2 or more lanes loaded:

$$g = 0.2 + \left(\frac{S}{12}\right) - \left(\frac{S}{35}\right)^{2.0}$$

26

Skew Bridges

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Distribution Factors for I-Beams: Moment with Skew (Table 4.6.2.2.2e-1, p. 4-41)

Bending moments in interior and exterior beams on skewed supports may be reduced using the following multiplier:

Use: $1 - c_1 (\tan\theta)^{1.5}$

where $c_1 = 0.25\left(\frac{K_g}{12.0Lt_s^3}\right)^{0.25}\left(\frac{S}{L}\right)^{0.5}$

Set $c_1 = 0$ when $\theta < 30°$

Set $\theta = 60°$ when $\theta > 60°$

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Distribution Factors for I-Beams – Shear with Skew (Table 4.6.2.2.3c-1)

Shear in exterior beams at the obtuse corner of the bridge may be reduced using the following multiplier:

$$\text{Use: } 1.0 + 0.20\left(\frac{12.0Lt_s^{\,3}}{K_g}\right)^{0.3}\tan\theta$$

This formula is valid for θ < 60°

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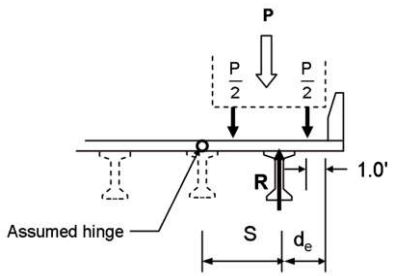
Lever Rule (4.6.2.2)



This approach is used for exterior girders only.

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Refined Methods of Analysis (4.6.3, p. 4-67)

4.6.3.1 General

Refined methods, listed in Article 4.4, may be used for the analysis of bridges. In such analyses, consideration shall be given to aspect ratios of elements, positioning and number of nodes, and other features of topology that may affect the accuracy of the analytical solution.

Nine methods are listed in Article 4.4 (p. 4-10), including

- Finite element method
- Finite difference method
- Grillage analogy method
- Yield line method

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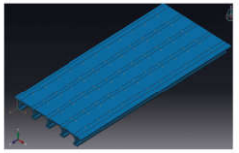
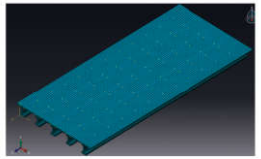
Finite Element Modeling and Evaluation, SH75 Bridge over Wilson Creek, McKinney, Texas





- Five girders with precast panels and CIP deck were modeled in ABAQUS CAE (2018) software.
- Using 3D deformable concrete elements.
- Prestressing and mild steel modeled as truss element.
- Thermal loading technique was used to apply prestress.

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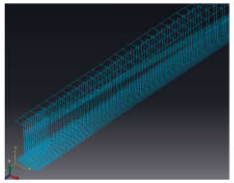
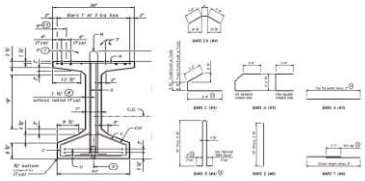
Finite Element Modeling (FEM)





- Each girder was modeled with 50 prestressing strands and 7 types of mild rebars.
- Material and section properties were defined accordingly.

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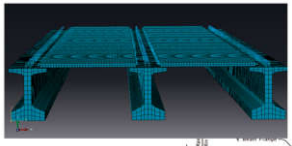
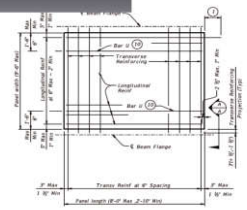
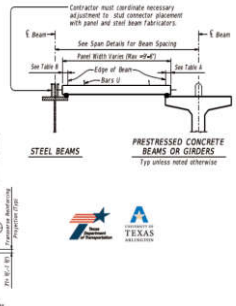
Finite Element Modeling (FEM)







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Design of Girders for Flexure

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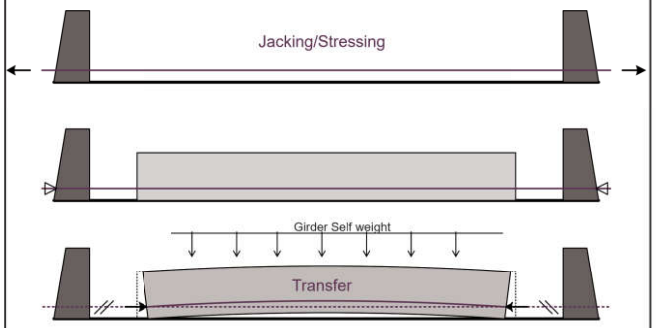
Construction Sequence



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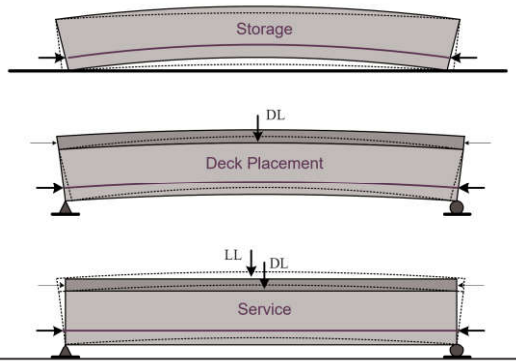
Loading Timeline



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Loading Timeline

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Loading Timeline

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Stress Limits: Prestressing Steel (Table 5.9.2.2.1, page 5-121)

YN1

Source: AASHTO (2017)

Table 5.9.2.2.1 **Stress Limits for Prestressing Tendons.** (Page 5-121)

Condition	Tendon Type: Stress-Relieved Strand and Plain High-Strength Bars	Low Relaxation Strand	Deformed High-Strength Bars
Pretensioning			
Immediately prior to transfer (f_{pbt})	$0.70 f_{pu}$	$0.75 f_{pu}$	—
At service limit state after all losses (f_{pe})	$0.80 f_{py}$	$0.80 f_{py}$	$0.80 f_{py}$
Post-Tensioning			
Prior to seating—short-term f_{pbt} may be allowed	$0.90 f_{py}$	$0.90 f_{py}$	$0.90 f_{py}$
At anchorages and couplers immediately after anchor set	$0.70 f_{pu}$	$0.70 f_{pu}$	$0.70 f_{pu}$
Elsewhere along length of member away from anchorages and couplers immediately after anchor set	$0.70 f_{pu}$	$0.74 f_{pu}$	$0.70 f_{pu}$
At service limit state after losses (f_{pe})	$0.80 f_{py}$	$0.80 f_{py}$	$0.80 f_{py}$

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Slide 40

YN1 Table 5.9.2.2.1, page 5-121
Yazdani, Nur, 9/26/2018

Stress Limits: Conc. Comp. (Transfer)

YN2

5.9.2.3- Stress Limits for Concrete

5.9.2.3.1 —For Temporary Stresses before Losses—Fully Prestressed Components

5.9.2.3.1a – Compressive Stresses

The compressive stress limit for pretensioned and post-tensioned concrete components, including segmentally constructed bridges, shall be 0.65 f'_{ci} (ksi).

5.9.2.3.1b – Tensile Stresses

The limits in Table 5.9.2.3.1b-1 shall apply for tensile stresses.

Source: AASHTO (2017)

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Stress Limits: Conc. Tension (Transfer)

YN3
YN4

Table 5.9.2.3.1b-1 Temporary Tensile Stress Limits in Prestressed Concrete Before Losses

Bridge Type	Location	Stress Limit
Other Than Segmentally Constructed Bridges	• In precompressed tensile zone without bonded reinforcement	N/A
	• In areas other than the precompressed tensile zone and without bonded reinforcement	0.0948λ
	• In areas with bonded reinforcement (reinforcing bars or prestressing steel) sufficient to resist the tensile force in the concrete computed assuming an uncracked section, where reinforcement is proportioned using a stress of 0.5 f_y, not to exceed 30 ksi.	$= 3\sqrt{f'_c}$ (psi) 0.24λ $\sqrt{f'_{ci}}$ (ksi)
	• For handling stresses in prestressed piles	0.158λ $\sqrt{f'_{ci}}$ (ksi)
Segmentally Constructed Bridges	Longitudinal Stresses Through Joints in the Precompressed Tensile Zone	

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YN2 0.65 f'ci
Yazdani, Nur, 9/26/2018

Slide 42

YN3 insert
Yazdani, Nur, 9/26/2018

YN4 lambda
Yazdani, Nur, 9/26/2018

Stress Limits: Conc. Comp (Service)

Table 5.9.2.3.2a-1 Compressive Stress Limits in Prestressed Concrete at Service Limit State After Losses, Fully Prestressed Components.

Location	Stress Limit
• In other than segmentally constructed bridges due to the sum of effective prestress and permanent loads	$0.45 f'_c$ (ksi)
• In segmentally constructed bridges due to the sum of effective prestress and permanent loads	$0.45 f'_c$ (ksi)
• Due to the sum of effective prestress, permanent loads, and transient loads as well as during shipping and handling	$0.60\ \phi_w f'_c$ (ksi)

Source: AASHTO (2017)

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Stress Limits: Conc. Tension (Service)

YN5

Table 5.9.2.3.2b-1 Tensile Stress Limits in Prestressed Concrete at Service Limit State After Losses

Bridge Type	Location	Stress Limit
Other Than Segmentally Constructed Bridges	Tension in the Precompressed Tensile Zone Bridges, Assuming Uncracked Sections	$= 6\sqrt{f'_c}\ (psi)$
	• For components with bonded prestressing tendons or reinforcement that are subjected to not worse than moderate corrosion conditions	$0.19\lambda\sqrt{f'_c} \leq 0.6$ (ksi)
	• For components with bonded prestressing tendons or reinforcement that are subjected to severe corrosive conditions	$0.0948\lambda\sqrt{f'_c}$ (ksi)
	• For components with unbonded prestressing tendons	No tension
Segmentally Constructed Bridges	Longitudinal Stresses Through Joints in the Precompressed Tensile Zone	

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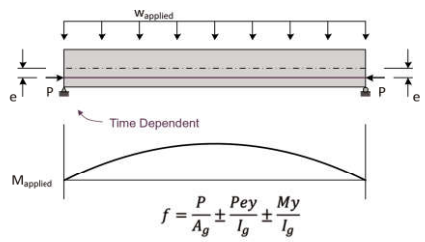
YN5 insert lambda
Yazdani, Nur, 9/26/2018

Stress Analysis



$$f = \frac{P}{A_g} \pm \frac{Pey}{I_g} \pm \frac{My}{I_g}$$

Gross vs. Transformed Section Properties

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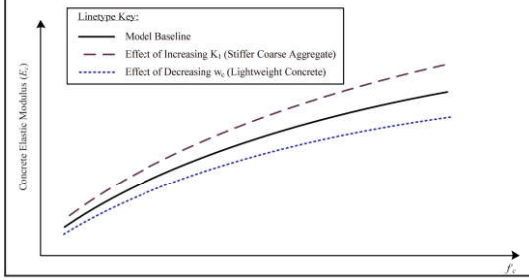
Concrete Properties: Elastic Modulus

AASHTO 5.4.2.4, p. 5-19

$$E_c = 33{,}000 K_1 w_c^{1.5} \sqrt{f_c'}$$



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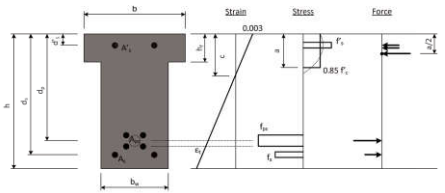
YN6

Nominal Moment Capacity



AASHTO 5.6.3.2, p. 5-39)

$$M_n = A_{ps}f_{ps}\left(d_p - \frac{a}{2}\right) + A_s f_s\left(d_s - \frac{a}{2}\right) - A'_s f'_s\left(d'_s - \frac{a}{2}\right) + \alpha_1 f'_c(b - b_w)h_f\left(\frac{a}{2} - \frac{h_f}{2}\right)$$

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YN7

Prestressing Steel Stress at Ultimate

AASHTO
5.6.3.1.1, p. 5-36

For bonded tendons:

$$f_{ps} = f_{pu}\left(1 - k\left(\frac{c}{d_p}\right)\right)$$

For low relaxation strands, k = 0.28

To locate the neutral axis:

$$c = \frac{A_{ps}f_{pu} + A_s f_s - A'_s f'_s - \alpha 1\, f'_c(b - b_w)h_f}{\alpha 1 f'_c \beta_1 b_w + kA_{ps}\left(\frac{f_{pu}}{d_p}\right)}$$

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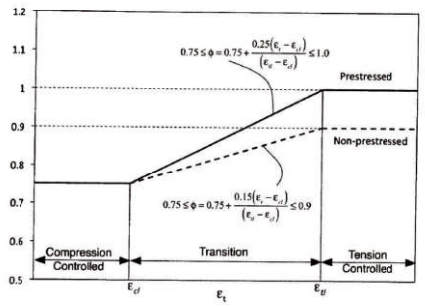
YN6 change 0.85 to alpha1
Yazdani, Nur, 9/26/2018

Slide 48

YN7 change 0.85
Yazdani, Nur, 9/26/2018

Resistance Factors

YN8



AASHTO 5.5.4.2, p. 5-30

Figure C5.5.4.2.1—Variation of ϕ with Net Tensile Strain ε_t for Nonprestressed Reinforcement and for Prestressing Steel

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Reinforcement Limits

- Maximum Reinforcement (5.6.3.3, p. 5-41)
  - No limit; has to be tension controlled
  - Effectively controlled by resistance factor, ensuring ductility
- Minimum Reinforcement

$$\phi M_n \ge Min \begin{cases} M_{cr} \\ 1.33 M_u \end{cases}$$

- Control of Cracking by Distribution of Reinforcement (5.6.7 p. 5-54)

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YN8 change fig. See page 5-30 AASHTO
Yazdani, Nur, 9/26/2018

Deflection

- Stiffer girders yield more durable decks
- Criteria for Deflection (AASHTO 2.5.2.6.2, p. 2-11) [optional]
  - All design lanes loaded
  - Consider stiffness of entire superstructure, including barriers, etc.
  - Use elastic methods; calculate I_e if section is cracked (AASHTO 5.6.3.5.2, p. 5-43)
  - Recommended limit: L/800

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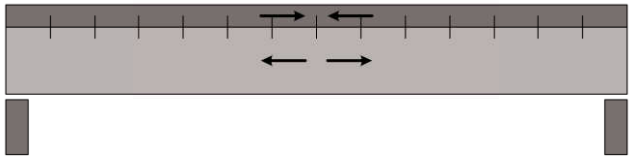
Composite Sections

AASHTO 5.7.4, p. 5-77

Design for interface shear



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Flexural Design Sequence

1. Estimate loads and span arrangement
2. Estimate cross section and material properties
3. Arrange prestressing to satisfy service stresses
   - If too much prestressing needed, use larger section, or increase number of girder lines
4. Consider draping/debonding to meet stress limits at transfer and end of span
5. Confirm ultimate capacity
6. Check reinforcing limits

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Points to Consider

- Service vs. Strength Limit States
- Sequence of construction/loading
- Stress analysis, including consideration for losses
- Prestress loss methods based on elastic analysis
- Girder design usually controlled by service stresses

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Example 1

A highway bridge uses steel girders.

Design Code:
AASHTO LRFD Bridge Design Specifications, 2017.

Design Data:
- Total service dead load moment (DC): 1,200 kip-ft
- Total service load moment including dynamic load allowance (LL + IM): 850 kip-ft
- Total service dead load moment due to wearing surface (DW): 100 kip-ft

Example 1

The design moment (kip-ft) using the Strength 1 loading combination with maximum load factors is most nearly:

a) 2,912
b) 3,138
c) 3,438
d) 3,684

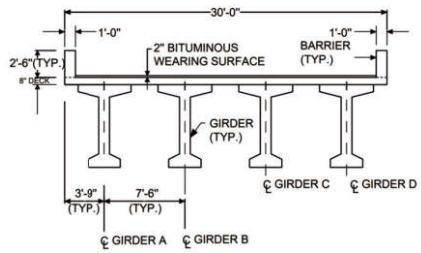
Solution:

$M_u = 1.25(1200) + 1.5(100) + 1.75(850)$ (Table 3.4.1-1, 3-15, 3.4.1-2, 3-16)

$= 3138\ kip - ft = 4255\ kN - m$

Example 2

Figure shows a bridge superstructure cross section. The bridge is a single-span structure with precast, prestressed concrete girders, a cast-in-place concrete deck, and a bituminous wearing surface. Barriers are cast in place.



Example 7 Continued

Design Code: *AASHTO LRFD Bridge Design Specifications, 2017*

Design Data

• Concrete unit weight	150 pcf (precast and cast-in-place concrete)
• Wearing surface unit weight	140 pcf kcf
• Bridge skew	0°
• Bridge length	120 ft (centerline of bearing to bearing)

Girder properties

$f'c = 7\ ksi$

Weight = 0.60 kip/ft

$I = 265{,}320\ in^4$

$A = 570\ in^2$

$e_{cb} = 28\ in$

Example 7 Continued

Cast-in-place properties:

$f'_c = 4\ ksi$

Assume 36 – 0.5 in. diameter low relaxation 270 ksi strands.

Eccentricity =

Assumptions:

- Bridge barrier and wearing surface are applied evenly to all girders
- Ignore design tandem loading
- Superstructure is a conventionally redundant system
- Bridge is considered operationally essential
- Bridge is conventional design

Example 2

Solution:

$W_{girder} = 0.60\ kip/ft\ (given)$

$W_{deck} = \left[\left(\frac{7'-6''}{2} * 2\right)\right]\left(\frac{8}{12}\right)(0.150) = 0.75\ kip/ft$

$W_{barrier} = \frac{2\ barriers}{4\ girders}[(2'\ 6'')(1)](0.15) = 0.19 kip/ft$

$W_{DC} = 0.6 + 0.75 + 0.19 = 1.54\ kip/ft$

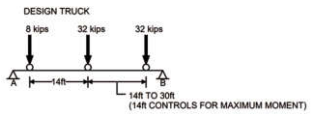
$M_{DC} = \frac{wL^2}{8} = \frac{1.54(120)^2}{8} = 2772\ kip - ft$

Determine the wearing surface moment for Girder A:

$W_{DW} = \frac{[30' - 2(2')]}{4\ girders}\left(\frac{2}{12}\right)(0.140) = 0.163\ kip/ft$ (Table 3.5-1-1, page 3-19)

$M_{DW} = \frac{wL^2}{8} = \frac{(0.163)(120)^2}{8} = 293\ kip - ft$

Example 2



Note: It is sufficiently accurate to place the center axle at midspan.

$\sum M_B = 0 = -(32)(60' - 14') - (32)(60') - (8)(60' + 14') + R_A(120')$

$R_A = \frac{3{,}984}{120} = 33.2\ kip$

$\sum M_{CL} = 0 = 33.2(60') - 8(14') - M_{truck} = 0$

$M_{truck/lane} = 1880\ kip - ft/lane$

Example 2

- Design lane

  $W_{lane} = 0.64\ kip/ft\ (over\ 3\ m\ width)$ (Art. 3.6.1.2.4, 3-23)

  $M_{lane/lane} = \frac{wL^2}{8} = \frac{0.64(120\ \ )^2}{8} = 1152\ kip - ft$

- Dynamic Load Allowance (IM)

  Strength 1 limit state for girder => IM =33% (Art 3.6.2, 3-31)

- $IM_{factor\ for\ truck} = \left(1 + \frac{33}{100}\right) = 1.33$

Example 2

- Application of design vehicular live loads (Art 3.6.1.3, 3-26)

Outside wheel of design truck placed 0.6 m from edge of design lane for girders.

$M_{LL+IM/lane} = 1880(1.33) + 1152 = 3652\ kip - ft/lane \approx 4951\ kN - m/lane$

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Example 2

$e = 0.77 + \frac{d_e}{9.1} = 0.77 + \frac{2.75}{9.1} = 1.072$

$f_c'(girder) = 7.0\ ksi$ $\quad W_{girder} = 0.150\ kcf$

$f_c'(C.I.P.) = 4.0\ ksi$ $\quad W_{C.I.P.} = 0.150\ kcf$

$E_{girder} = 33000K_1W_c^{1.5}\sqrt{f_c'} = 33{,}000(1.0)(0.150)^{1.5}\sqrt{7.0} = 5072\ ksi$ (Eq. C5.4.2.4-2, 5-20)

$E_{C.I.P.} = 33000(1.0)(0.150)^{1.5}\sqrt{4.0} = 3834\ ksi$

$n = \frac{E_B}{E_D} = \frac{5{,}072}{3{,}834} = 1.32$ (Eq.4.6.2.2.1-2, 4-32)

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Example 2

$K_g = n(I + Ae_g^2)$

$= 1.32(265{,}320 + 570(28)^2 = 940{,}104\ in^4$

One design lane loaded: $g_{int} = 0.06 + \left(\frac{S}{14}\right)^{0.4} + \left(\frac{S}{L}\right)^{0.3}\left(\frac{K_g}{12.0Lt_s^3}\right)^{0.1}$,

(Table 4.6.2.2.2b-1, 4-37)

Two or more design lanes loaded: $g_{int} = 0.075 + \left(\frac{S}{9.5}\right)^{0.6}\left(\frac{S}{L}\right)^{0.2}\left(\frac{K_g}{12.0Lt_s^3}\right)^{0.1}$

Applicability check

$3.5 \le S = 7.5' < 16.0$ OK $\quad N_b = 4$ OK

$4.5 \le t_s = 8" \le 12.0$ OK $\quad 10{,}000 \le K_g = 940{,}104\ in^4 \le 7{,}000{,}000$

$20 \le L = 120' < 240$ OK

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Example 2

$g_{int} = 0.06 + \left(\frac{7.5}{14}\right)^{0.4}\left(\frac{7.5}{120}\right)^{0.3}\left(\frac{(940{,}104)}{12.0\ (120)(8^3)}\right)^{0.1} = 0.408\ one\ lane$

$g_{int} = 0.075 + \left(\frac{7.5}{19.5}\right)^{0.4}\left(\frac{7.5}{120}\right)^{0.3}\left(\frac{(940{,}104)}{12.0\ (120)(8^3)}\right)^{0.1} = 0.586\ two\ lane$

$g_{int} = 0.586$

66

Example 2

For strength limit states:

$\gamma_{DC} = 1.25$ max, $\gamma_{DW} = 1.5$ max (Tables 3.4.1-1, 3-15, and 3.4.1-2, 3-16)

For Strength I Limit State:

$\sum \gamma_i Q_i = [\gamma_P(DC + DW) + 1.75(LL + IM)]$

$M_u = (1.25(2772) + 1.5(293) + 1.75(0.842)(3652)]$

$= 9285\ kip - ft \approx 12588\ kN - m, \quad at\ midspan\ of\ Girder\ A$

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Example 2

$A_{ps} = 36 * 0.153 = 5.508$ sq. in. (for 0.5 in, low lax),

$d_p = 75.7$ in.

$$c = \frac{5.508\ (270)}{0.85(4)(0.85)(108) + 0.28(5.508)\frac{270}{75.7}} = 5.54 \text{ in.}$$

a = depth of stress block = 0.85(4.68) = 3.98 in.< 8 in.

So, beam behaves as rectangular!

Av. stress in strands = $270\left(1 - 0.28\frac{4.68}{75.78}\right) = 265$ ksi

Moment capacity, M_u = $5.508(265)\left(75.78 - \frac{3.98}{2}\right) = 8986\ k - ft.$

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Example 2

Check tension-controlled: $\epsilon_t = \frac{78-4.68}{4.68} * 0.003 = 0.047 > 0.005, OK!$

$\emptyset = 1.0$

Design moment capacity, M_r = 8986 k-ft = 12183 kN-m > M_u = 12588 kN-m,

NG!

Change design to 40 strands.

Still needed: prestress losses; service limit state checks; minimum steel; end and transfer points; shear design, interface shear, anchorage zone

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Questions?

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Bridge Deck Design (AASHTO LRFD 2017)

Date: 2 March 2022
Session 42 – Bridges 03

Nur Yazdani, Ph.D. P.E., F.ASCE, F.ACI, F.SEI
Professor of Civil Engineering
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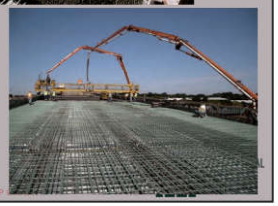
S-8A SEng PRP
Training Program

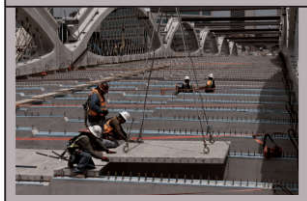
1

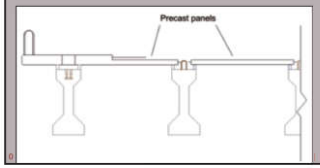
Design of Bridge Decks



2

Design of Bridge Decks





3

Design of Bridge Decks

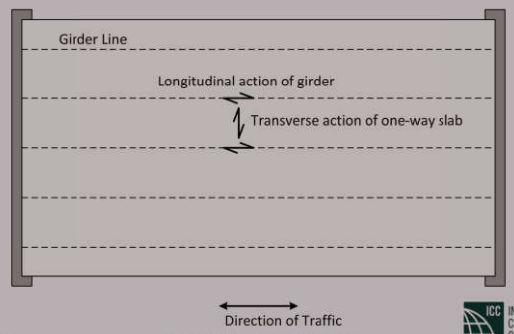
4

Design of Bridge Decks

- ✓ Deck replacement is impractical and almost impossible to do without closing the entire bridge. Therefore, it is always a good strategy to be conservative and allow for reserved capacity during design.
- ✓ Subjected to static and dynamic loads, thermal gradients, and creep and shrinkage effects. Proper consideration should be given to these effects to prevent cracking and deterioration.

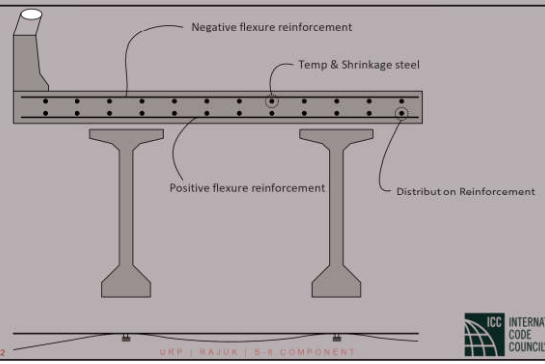
5

The Structural System



6

The Structural System



7

AASHTO LRFD Bridge Design Specifications; Chapter 9: Decks

- Minimum deck thickness = 7 in. (p. 9-7)
- All previous limit states apply.
- Methods of deck analysis (p. 9-6).
- Minimum covers in AASHTO Table 5.10.1-1 (p. 5-165).
- For decks with less than 25 deg. skew, place primary rebars in the skew direction. Otherwise, place it normal to the supporting beams.

Table 5.10.1-1—Cover for Unprotected Main Reinforcing Steel (in.)

Situation	Cover (in.)
Direct exposure to salt water	4.0
Cast against earth	3.0
Coastal	3.0
Exposure to deicing salts	2.5
Deck surfaces subject to tire stud or chain wear	2.5
Exterior other than above	2.0
Interior other than above • Up to No. 11 bar • No. 14 and No. 18 bars	 1.5 2.0
Bottom of cast-in-place slabs • Up to No. 11 bar • No. 14 and No. 18 bars	 1.0 2.0
Precast soffit form panels	0.8
Precast reinforced piles • Noncorrosive environments • Corrosive environments	 2.0 3.0
Precast prestressed piles	2.0
Cast-in-place piles • Noncorrosive environments • Corrosive environments o General o Protected • Shells • Auger-cast, tremie concrete, or slurry construction	 2.0 3.0 3.0 2.0 3.0
Precast concrete box culverts • Top slabs used as a driving surface • Top slabs with less than 2.0 ft of fill not used as a driving surface • All other members	 2.5 2.0 1.0

8

Concrete Covers (different than BNBC-2020 and ACI 318!)

- AASHTO 5.10.1 (p. 5-165).
- Minimum cover is 1.0 in. always.
- For unprotected steel, use minimum covers in Table 5.10.1-1, modified for w/c ratio (p. 5-164).
  - » For w/c ratio ≤ 0.4, modify by 0.8
  - » For w/c ratio ≥ 0.5, modify by 1.2
- For post-tensioned ducts, use
  - » Same as main reinforcing
  - » ½ of duct diameter
  - » Table 5.10.1-1
- For epoxy coated rebars, use interior exposure from Table 5.10.1-1 (p. 5-185).

5.10.1—Concrete Cover

Cover for unprotected prestressing and reinforcing steel shall not be less than that specified in Table 5.10.1-1 and modified for *W/CM* ratio, unless otherwise specified either herein or in Article 5.12.5.

Modification factors for *W/CM* ratio shall be the following:

- For *W/CM* ≤ 0.40 ..... 0.8
- For *W/CM* ≥ 0.50 ..... 1.2

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9

Deck Analysis Methods (AASHTO LRFD)

- Approximate Methods
  - » Strip Method (4.6.2.1.1, p. 4-22)
    - –Design aids allowed for decks with precast elements
  - » Empirical Method (9.7.2, p. 9-8)
- Refined Methods (4.6.3.2, p. 4-67)
- Overhang Design (9.7.1.5)

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10

Strip Method - Overview

- Model slab strip as continuous beam (p. 4-26)
- Analyze to find moment at critical sections
- Determine width of strip sharing load effect
- Design as one-way concrete slab

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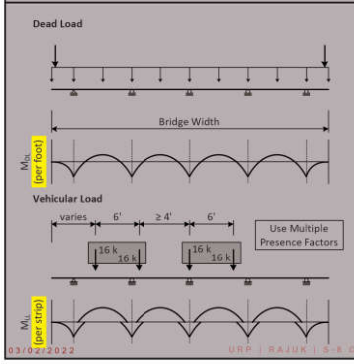
11

Strip Method – Analysis



LL moments can be found from AASHTO App. A4 (p. 4-98), Deck Slab Design Table

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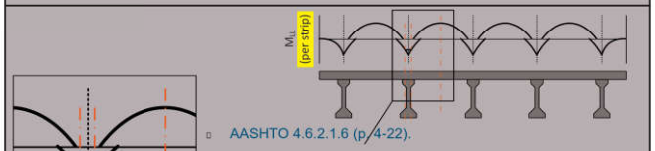
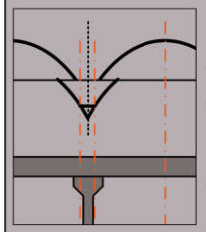
12

Strip Method – Critical Section





- AASHTO 4.6.2.1.6 (p. 4-22).
- For precast I-shaped girders, critical section for negative moment and shear is: $b_f/3 \leq 15$ in. from girder centerline (p. 4-27). For monolithic construction, precast boxes and stemmed beams, use section at the support face.
- Use beam theory for continuous beams and assume infinite rigidity for supporting elements.
- Apply maximum ±moments to all ± moment sections (p. 4-22).

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13

Location of Negative Moment

4.6.2.1.6—Calculation of Force Effects

The strips shall be treated as continuous beams or simply supported beams, as appropriate. Span length shall be taken as the center-to-center distance between the supporting components. For the purpose of determining force effects in the strip, the supporting components shall be assumed to be infinitely rigid.

The wheel loads may be modeled as concentrated loads or as patch loads whose length along the span shall be the length of the tire contact area, as specified in Article 3.6.1.2.5, plus the depth of the deck. The strips should be analyzed by classical beam theory.

The design section for negative moments and shear forces, where investigated, may be taken as follows:

- For monolithic construction, closed steel boxes, closed concrete boxes, open concrete boxes without top flanges, and stemmed precast beams, i.e., Cross-sections (b), (c), (d), (e), (f), (g), (h), (i), and (j) from Table 4.6.2.2.1-1, at the face of the supporting component,
- For steel I-beams and steel tub girders, i.e., Cross-sections (a) and (c) from Table 4.6.2.2.1-1, one-quarter the flange width from the centerline of support,

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Strip Method – Strip Width (p. 4-24)

Table 4.6.2.1.3-1—Equivalent Strips

Type of Deck	Direction of Primary Strip Relative to Traffic	Width of Primary Strip (in.)
Concrete:		
• Cast-in-place	Overhang	$45.0 + 10.0X$
	Either Parallel or Perpendicular	$+M$: $26.0 + 6.6S$ $-M$: $48.0 + 3.0S$
• Cast-in-place with stay-in-place concrete formwork	Either Parallel or Perpendicular	$+M$: $26.0 + 6.6S$ $-M$: $48.0 + 3.0S$
• Precast, post-tensioned	Either Parallel or Perpendicular	$+M$: $26.0 + 6.6S$ $-M$: $48.0 + 3.0S$

Source: AASHTO (2017)

X = distance of load from point of support
S = spacing of supports

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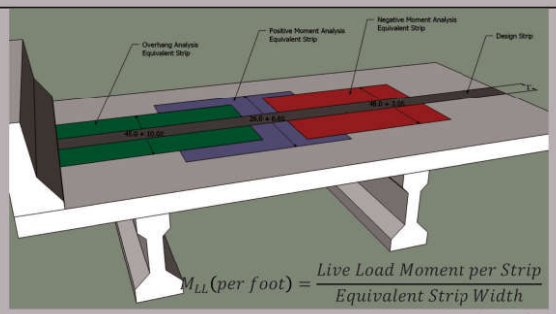
Strip Method – Strip Width



$$M_{LL}(per\ foot) = \frac{Live\ Load\ Moment\ per\ Strip}{Equivalent\ Strip\ Width}$$

LL Shortcut: AASHTO Table A4.1(p. 4-98)

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Strip Method – Strength Design

- Design Top and Bottom Transverse Reinforcement based on Factored Positive and Negative Moments.
  - » Include IM Factor.
  - » Include Multiple Presence Factors.

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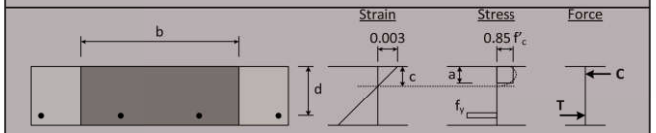
Strip Method – Strength Design



$$M_n = A_s f_y \left(d - \frac{a}{2}\right)$$

$$a = \beta_1 c$$

$$a = \frac{A_s f_y}{0.85 f_c' b}$$

$$A_s = \frac{A_{bar}}{s}$$

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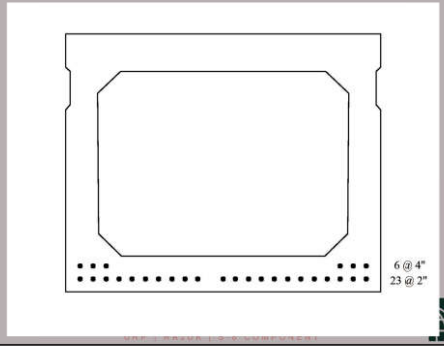
AASHTO Cover Provisions Satisfied?



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Rebar Spacing

5.10.3.2—Maximum Spacing of Reinforcing Bars

Unless otherwise specified, the spacing of the reinforcement in walls and slabs shall not be greater than the lesser of the following:

- 1.5 times the thickness of the member; or
- 18.0 in.

The maximum spacing of spirals, ties, and temperature shrinkage reinforcement shall be as specified in Articles 5.10.4, 5.10.5, and 5.10.6.

5.10.3—Spacing of Reinforcement

5.10.3.1 Minimum Spacing of Reinforcing Bars

5.10.3.1.1—Cast-in-Place Concrete

For cast-in-place concrete, the clear distance between parallel bars in a layer shall not be less than the largest of the following:

- 1.5 times the nominal diameter of the bars;
- 1.5 times the maximum size of the coarse aggregate; or
- 1.5 in.

5.10.3.1.2—Precast Concrete

For precast concrete manufactured under plant control conditions, the clear distance between parallel bars in a layer shall not be less than the largest of the following:

- the nominal diameter of the bars;
- 1.33 times the maximum size of the coarse aggregate; or
- 1.0 in.

5.10.3.1.3—Multilayers

Except in decks where parallel reinforcement is placed in two or more layers, with clear distance between layers not exceeding 6.0 in., the bars in the upper layers shall be placed directly above those in the bottom layer, and the clear distance between layers shall not be less than 1.0 in. or the nominal diameter of the bars.

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Minimum (Primary) Reinforcement

AASHTO 5.6.3.3

$$\phi M_n \geq least\ of \begin{cases} M_{cr} \\ 1.33 M_u \end{cases}$$

$$M_{cr} = \frac{f_r I_g}{y_t}$$

I_g – *gross moment of inertia*

y_t – *neutral axis distance to tension face*

f_r – *modulus of rupture* $= 0.37\sqrt{f'_c}$ *(ksi)*

Upper bound

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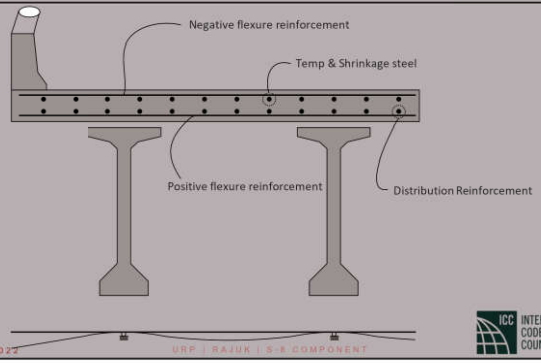
Distribution Reinforcement



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Distribution Reinforcement

AASHTO 9.7.3.2, p. 9-12

- Bottom of slab in secondary direction
  - » Engages additional primary positive moment reinforcement
- Based on a percentage of primary positive moment reinforcement
- When primary reinforcement is perpendicular to traffic:

$$\frac{A_{s-distribution}}{A_{s-primary\ positive}} = max \begin{cases} \frac{220}{\sqrt{S}}\% \\ 67\% \end{cases}$$

S – *effective slab span (~ girder spacing) see AASHTO 9.7.2.3*

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Temp. & Shrinkage Reinforcement

- Minimum reinforcement in both directions
- Usually only controls on top in secondary direction, distribution steel in bottom controls.

$$A_s\left(\frac{in^2}{ft}\right) \geq \frac{\overbrace{1.30bh}^{\text{Cross Sectional Area}}}{\underbrace{2(b+h)f_y}_{\text{Drying perimeter}}} \quad \text{where } 0.11 \leq A_s \leq 0.60$$

AASHTO 5.10.6, p. 5-170

- Bar spacing shall not exceed:

$$least \begin{cases} 3 * \text{slab thickness} \\ 18'' \end{cases}$$

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Temp. & Shrinkage Reinforcement

Epoxy coated rebars

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Empirical Method – AASHTO 9.7.2 (p. 9-8)

- Based on research and FEM verification
- Load resistance mechanism – internal arching action
- No analysis required
- Isotropic reinforcement – same both directions in each layer
- Does NOT apply to overhang design

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Empirical Method

- Many design conditions must be met: diaphragms, fully cast in place and water cured, uniform depth, and others (p. 9-10)……
- Steel spacing ≤ 18 in., at least Grade 60 steel.
- Bottom layer, each way: 0.27 in^2/ft.
  - » ~ #5 bars at 13.5 in.
- Top layer, each way: 0.18 in^2/ft.
  - » ~ #4 bars at 13 inches
- May not give as much "feel" for the structure

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Overhang Design – AASHTO A13.4

- 3 design cases (AASHTO A13.4.1, p. 13-25)
  - » Horizontal Vehicle Collision (Extreme Event II)
  - » Vertical Collision (Extreme Event II. p. 3-10)
  - » DL and LL on Overhang (Strength I)
- 3 design sections
  - » Inside of parapet (A)
  - » Critical section outside exterior girder (B)
  - » Critical section inside exterior girder (C)

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Overhang Design – AASHTO A13.4

[redacted]

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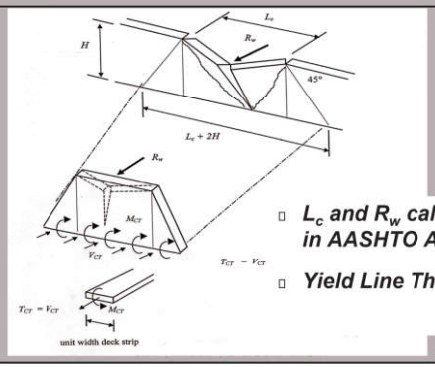
Barrier Capacity – Yield Line 1



- L_c and R_w calculated in AASHTO A13.3.1
- Yield Line Theory

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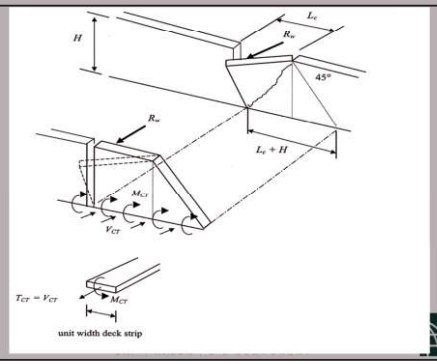
Barrier Capacity – Yield Line 2



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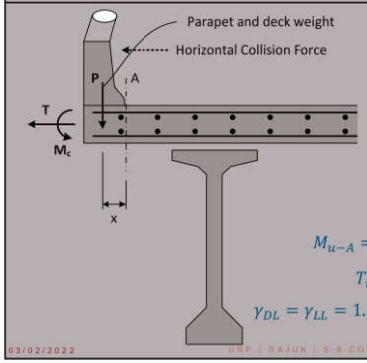
Case 1 – Section A



M_c: *Moment capacity of barrier about horizontal axis parallel to traffic*

$$T = \begin{cases} \text{Yield Line 1: } \dfrac{R_w}{L_c + 2H} \\ \text{Yield Line 2: } \dfrac{R_w}{L_c + H} \end{cases}$$

$$M_{u-A} = \gamma_{LL} M_c + \gamma_{DL} P x$$

$$T_{u-A} = \gamma_{LL} T$$

$$\gamma_{DL} = \gamma_{LL} = 1.0 \quad (\text{Extreme Event})$$

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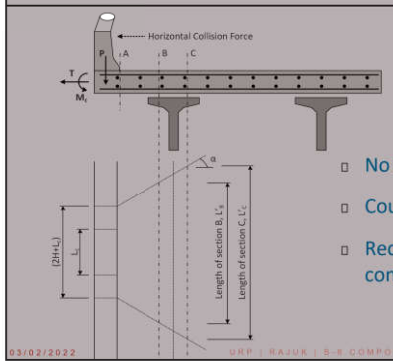
Case 1 – Section B & C



$$M'_B = M_c \frac{(L_c + 2H)}{L'_B}$$

$$T'_B = T \frac{(L_c + 2H)}{L'_B}$$

- No Spec guidance for α
- Could probably use 45 degrees
- Recommend 30 degrees conservatively

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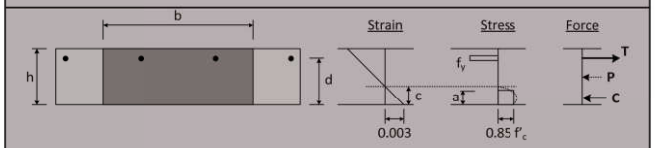
Overhang Capacity



$$T = sum \begin{cases} T_1 & (arises\ from\ moment, = C) \\ T_2 & (arises\ from\ axial\ force, = P) \end{cases}$$

$$a = \frac{A_s f_y - P}{0.85 f'_c b}$$

$$M_n = A_s f_y \left(d - \frac{a}{2}\right) - P\left(\frac{h}{2} - \frac{a}{2}\right)$$

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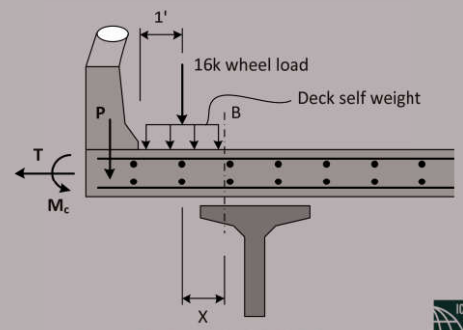
Case 3 – Section B



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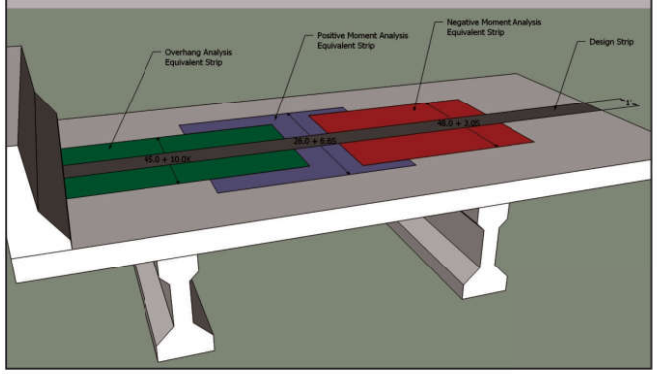
Overhang Equivalent Strip



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Simple Spans made Continuous

- Additional longitudinal reinforcement needed over interior piers to achieve ϕM_n in negative moment

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Key Points

- Decks are one-way slabs spanning between girders
- Four layers of reinforcement
  - » Primary negative reinforcement
  - » Primary positive reinforcement
  - » Distribution reinforcement
  - » Temp and Shrinkage reinforcement
- Equivalent strip width is the amount of concrete acting together to carry wheel load to girder
- Must consider horizontal collision force

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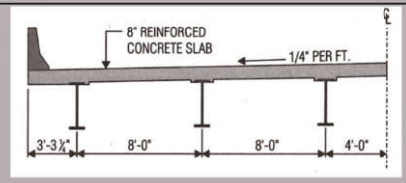
Example: Design of Reinforced Concrete Deck Slab



Given:
- Bridge to carry two traffic lanes.
- Concrete strength, f_c' = 4.5 ksi.
- Grade 60 reinforcement, f_y= 60 ksi.
- Account for 25 psf future wearing surface.
- Assume supporting steel girders are W36 × 150.
- Deck has a 0.5 in. integrated wearing surface.
- Design lane = 12 ft. width.
- Deck is composite with beams.

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Example

Determine applicability of AASHTO Deck Design Tables (A 4 -1, p. 4-98).

a) IM and multiple presence factors considered.
b) More than three girders.
c) Distance between exterior girder (40 ft) > 14 ft

Compute moments due to dead loads.

Dead loads from slab and future wearing surface:

DC = $(thickness\ of\ slab)(unit\ weight\ of\ concrete)$ = (8 in)/(12 in)(0.15 kcf)
= 0.1 $klft$

DW = $future\ wearing\ surface$ = 0.025 $klft$
= 0.025 $klft$

40

Example

Table 6.5.2—Approximate moments for nonprestressed continuous beams and one-way slabs

Moment	Location	Condition	M_u
Positive	End span	Discontinuous end integral with support	$w_u\ell_n^2/14$
		Discontinuous end unrestrained	$w_u\ell_n^2/11$
	Interior spans	All	$w_u\ell_n^2/16$
Negative[1]	Interior face of exterior support	Member built integrally with supporting spandrel beam	$w_u\ell_n^2/24$
		Member built integrally with supporting column	$w_u\ell_n^2/16$
	Exterior face of first interior support	Two spans	$w_u\ell_n^2/9$
		More than two spans	$w_u\ell_n^2/10$
	Face of other supports	All	$w_u\ell_n^2/11$
	Face of all supports satisfying (a) or (b)	(a) slabs with spans not exceeding 10 ft (b) beams where ratio of sum of column stiffnesses to beam stiffness exceeds 8 at each end of span	$w_u\ell_n^2/12$

[1]To calculate negative moments, ℓ_n shall be the average of the adjacent clear span lengths.

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Example

Positive bending moment/ft:

M_{DC}+ = (1/14)$DC \times S^2$ = (1/14)(0.1 k/ft)(8 ft)^2
= 0.45 $kip-ft$

M_{DW}+ = (1/14)$DW \times S^2$ = (1/14)(0.025 k/ft)(8 ft)^2
= 0.114 $kip-ft$

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Example

Equivalent strip widths (p. 4-24):

Positive moment: 26 + 6.6S = 26 + (6.6)(8) = 78.8 in.
Negative moment: 48 + 3.0S = 48 + (3.0)(8) = 72 in.

So, modified positive dead load moments/ft. (p. 4-23 commentary):

M_{DC}+ = 0.45(12*12/78.8) = 0.83 $kip-ft/ft$

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Example

M_{DW}+ = 0.114(12*12/78.8) = 0.208 $kip-ft/ft$
= 0.208 $kip-ft/ft$

Negative bending moment/ft:

M_{DC}− = (0.10$DC \times S^2$) = (0.10)(0.1 k/ft)(8 ft)^2
= 0.64 $kip-ft$

M_{DW}− = (0.10$DW \times S^2$)=(0.10)(0.025 k/ft)(8 ft)^2
= 0.16 $kip-ft$

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Example

So, modified negative dead load moments/ft. (p. 4-23 commentary):

$M_{DC}-$ = 0.64(12*12/72) =1.28 $kip-ft/ft$

$M_{DW}-$ = 0.161(12*12/72) = 0.32 $kip-ft/ft$

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Example

Compute live load bending moments:

AASHTO Table A4-1 (page 4.98) is used to obtain maximum approximate positive and negative HL-93 live load bending moments (Note: IM and multiple presence already factored in this table).

The deck span length is 8 ft. The top flange width for a W36 × 150 is 12 in, t_w = 0.625 in. (1/4 flange width = 3 in. is used for design negative moment location, p. 4-26).

M_L+ = 5.69 $kip-ft/ft$

M_L - = 5.65 $kip-ft/ft$

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

S	Positive Moment	Negative Moment						
		Distance from CL of Girder to Design Section for Negative Moment						
		0.0 in.	3 in.	6 in.	9 in.	12 in.	18 in.	24 in.
4 ft –0 in.	4.68	2.68	2.07	1.74	1.60	1.50	1.34	1.25
4 ft –3 in.	4.66	2.73	2.25	1.95	1.74	1.57	1.33	1.20
4 ft –6 in.	4.63	3.00	2.58	2.19	1.90	1.65	1.32	1.18
4 ft –9 in.	4.64	3.38	2.90	2.43	2.07	1.74	1.29	1.20
5 ft –0 in.	4.65	3.74	3.20	2.66	2.24	1.83	1.26	1.12
5 ft –3 in.	4.67	4.06	3.47	2.89	2.41	1.95	1.28	0.98
5 ft –6 in.	4.71	4.36	3.73	3.11	2.58	2.07	1.30	0.99
5 ft –9 in.	4.77	4.63	3.97	3.31	2.73	2.19	1.32	1.02
6 ft –0 in.	4.83	4.88	4.19	3.50	2.88	2.31	1.39	1.07
6 ft –3 in.	4.91	5.10	4.39	3.68	3.02	2.42	1.45	1.13
6 ft –6 in.	5.00	5.31	4.57	3.84	3.15	2.53	1.50	1.20
6 ft –9 in.	5.10	5.50	4.74	3.99	3.27	2.64	1.58	1.28
7 ft –0 in.	5.21	5.98	5.17	4.36	3.56	2.84	1.63	1.37
7 ft –3 in.	5.32	6.13	5.31	4.49	3.68	2.96	1.65	1.51
7 ft –6 in.	5.44	6.26	5.43	4.61	3.78	3.15	1.88	1.72
7 ft –9 in.	5.56	6.38	5.54	4.71	3.88	3.30	2.21	1.84
8 ft –0 in.	5.69	6.48	5.65	4.81	3.98	3.43	2.49	2.16
8 ft –3 in.	5.83	6.58	5.74	4.90	4.06	3.53	2.74	2.37
8 ft –6 in.	5.99	6.66	5.82	4.98	4.14	3.61	2.96	2.58
8 ft –9 in.	6.14	6.74	5.90	5.06	4.22	3.67	3.15	2.79
9 ft –0 in.	6.29	6.81	5.97	5.13	4.28	3.71	3.31	3.00
9 ft –3 in.	6.44	6.87	6.03	5.19	4.40	3.82	3.47	3.20
9 ft –6 in.	6.59	7.15	6.31	5.46	4.66	4.04	3.68	3.39
9 ft –9 in.	6.74	7.51	6.65	5.80	4.94	4.21	3.89	3.58
10 ft –0 in.	6.89	7.85	6.99	6.13	5.26	4.41	4.09	3.77
10 ft –3 in.	7.03	8.19	7.32	6.45	5.58	4.71	4.29	3.96
10 ft –6 in.	7.17	8.52	7.64	6.77	5.89	5.02	4.48	4.15
10 ft –9 in.	7.32	8.83	7.95	7.08	6.20	5.32	4.68	4.34
11 ft –0 in.	7.46	9.14	8.26	7.38	6.50	5.62	4.86	4.52
11 ft –3 in.	7.60	9.44	8.55	7.67	6.79	5.91	5.04	4.70
11 ft –6 in.	7.74	9.72	8.84	7.96	7.07	6.19	5.22	4.87
11 ft –9 in.	7.88	10.01	9.12	8.24	7.36	6.47	5.40	5.05
12 ft –0 in.	8.01	10.28	9.40	8.51	7.63	6.74	5.56	5.21
12 ft –3 in.	8.15	10.55	9.67	8.78	7.90	7.02	5.75	5.38
12 ft –6 in.	8.28	10.81	9.93	9.04	8.16	7.28	5.97	5.54
12 ft –9 in.	8.41	11.06	10.18	9.30	8.42	7.54	6.18	5.70
13 ft –0 in.	8.54	11.31	10.43	9.55	8.67	7.79	6.38	5.86
13 ft –3 in.	8.66	11.55	10.67	9.80	8.92	8.04	6.59	6.01
13 ft –6 in.	8.78	11.79	10.91	10.03	9.16	8.28	6.79	6.16
13 ft –9 in.	8.90	12.02	11.14	10.27	9.40	8.52	6.99	6.30
14 ft –0 in.	9.02	12.24	11.37	10.50	9.63	8.76	7.18	6.45
14 ft –3 in.	9.14	12.46	11.59	10.72	9.85	8.99	7.38	6.58
14 ft –6 in.	9.25	12.67	11.81	10.94	10.08	9.21	7.57	6.72
14 ft –9 in.	9.36	12.88	12.02	11.16	10.30	9.44	7.76	6.86
15 ft –0 in.	9.47	13.09	12.23	11.37	10.51	9.65	7.94	7.02

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Example

So, modified live load moments/ft. (p. 4-23 commentary):

M_L+ = 5.69(12*12/78.8) = 10.4 $kip-ft/ft$
M_L- = 5.65(12*12/72) = 11.3 $kip-ft/ft$

Compute total factored bending moments (Strength I).

M_f+ = 1.25$M_{DC}+$ + 1.5$M_{DW}+$ +1.75M_L+
= (1.25)(0.83 $kip-ft$) + (1.5)(0.208 $kipt-ft$) + (1.75)(10.4 $kip-ft$) = 19.55 $kip-ft/ft$

M_f- = 1.25$M_{DC}-$+1.5$M_{DW}-$+1.75M_L-
= (1.25)(1.28 $kip-ft$) + (1.5)(0.32 $kip-ft$) + (1.75)(11.3 $kip-ft$) = 21.85 $kip-ft$/ft

48

Example

Compute effective depth of slab:
· Slab depth = 8 in
· Sacrificial surface = 0.5 in
· Concrete cover = 2.5 in top and 1.0 in bottom (p. 5-165)
· #5 reinforcing steel bar diameter = 0.625 in

The effective depths of the slab:
d_+= 8.0 − 0.5 − 1.0− 0.625/2 = 6.19 in
d_-= 8.0 −2.5 − 0.625/2 = 5.19 in

49

Example

Positive bending moment design (use any method of preference):

R_n= $M_u/\emptyset bd^2$ = 19.55(12000)/1.0(12)6.192 = 510 psi

From design aid,
ρ = 0.0092, and A_s = 0.0092(12)(6.19) = 0.68 in²/ft
Now, using #6 rebars:
s+ = 0.44*12/0.68 = 7.76 in, use 7 in. o.c.

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Example

Negative bending moment design:
R_n = $M_u/\emptyset bd^2$ = 21.85(12000)/1.0(12)5.19² = 799 psi

From design aid,
ρ = 0.0149, and A_s = 0.0149(12)(5.19) = 0.93 in²/ft.

Now, using #6 rebars:
s_-= 0.44*12/0.93 = 5.67 in, use 5 in. o.c.

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Example

Check maximum spacing for tension steel (p. 5-168):
s_{max}= 1.5∗8 or 18 in, OK

Distribution reinforcement in bottom of slab (p. 9-12):

The effective span length:

S = 8.0 ft − 0.05ft = 7.95 ft
D = 220/√S = 2 20/√7.95 = 78% > 67%, so use 67%

52

Example

Distribution reinforcement = $(A_s)(67\%)$ = $(0.68\ in^2/ft)(0.67)$
= $0.46\ in^2/ft$

So, use no. 6 bars at 11 in. o. c.

T&S reinforcement at the top of slab (p. 5-170):

$A_s \geq 1.30(12)(8)/2(12 + 8)60 = 0.052 < 0.11$ minimum

So, use $A_s = 0.11\ in^2$

Use #3 @ 12 in. o.c., satisfies 3h or 18 in. max spacing.

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Segmental Deck Design

- Design should be checked for construction load combinations, such as segment lifting, construction equipment, and segment stacking.
- It is standard practice to select a minimum top deck thickness of 8 in, although AASHTO-PCI-ASBI Standard Sections Committee recommends a minimum thickness of 9 in.
- It is recommended that for all posttensioned box girders the top deck be transversely posttensioned, even for short overhangs since it improves long-term durability and results in low life cycle cost.
- When a static concentrated load is applied on a deck, the deck will deflect transversely as well as longitudinally, similar to two-way slabs.

54

Transverse Analysis

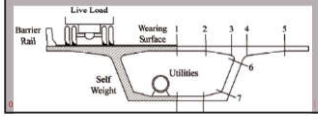
- ✓To evaluate transverse bending moments from both permanent and live loads.
- ✓Results are used to design the reinforcing and post-tensioning of the cantilever wings, top slab, webs, and bottom slab.
- ✓An accepted approach for the transverse analysis of a concrete box girder superstructure is a simplified 2D approach with sufficient consideration for longitudinal load distribution.

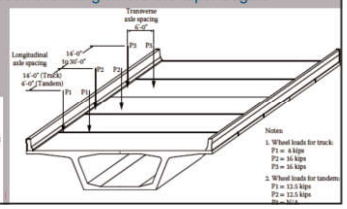
One Foot Section Segmental Bridge
for Transverse Analysis

55

Transverse Analysis

- ➢Live load is strategically placed to produce worst condition. Some common points where stresses are checked are:
  - ✓Maximum negative bending moment at the root of deck overhang
  - ✓Maximum positive and negative bending moments at the center line between two webs
  - ✓Maximum negative bending moment in the top deck at the interior face of the webs
  - ✓Maximum negative and positive bending moments in the webs and bottom slab
  - ✓Maximum negative moment in the deck overhang where the taper begins





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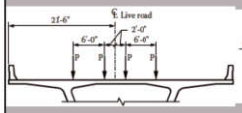
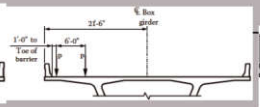
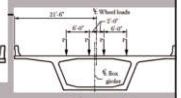
LRFD Live Load Transverse Configurations



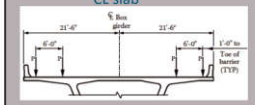
Live load configuration for maximum positive moment at CL slab



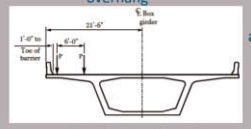
Live load configuration for maximum negative moment at overhang



Live load configuration for maximum negative moment in top slab at inside face of web



Live load configuration for maximum negative moment at CL slab



Live load configuration for maximum negative moment in bottom slab at inside face of web

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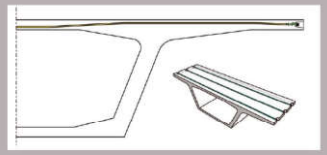
Transverse Post-tensioning

- The cantilever wings and the top slab are typically prestressed transversely with post-tensioning to offset tensile stresses resulting from permanent and live loads.
- Narrow precast box girders with widths of 16 ft. or less, often used for single track transit systems, may not greatly benefit from transverse post-tensioning.



Typical Transverse Tendon Layout

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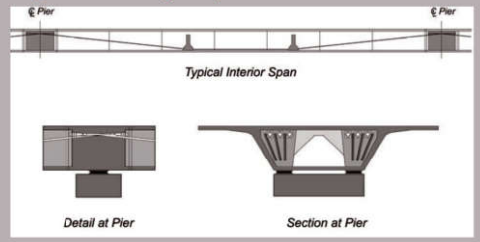
Longitudinal Design

- The longitudinal design depends on the erection method.



Example Tendon layout for Span-By-Span Segmental Bridge

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Video Clips and Software

- Segmental Bridge Construction
- Precast yard
- Software

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Questions?

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Bridge Substructure Design and Service Life Extension

Date: 7 March 2022
Session 45 – Bridges 04

Nur Yazdani, Ph.D. P.E., F.ASCE, F.ACI, F.SEI
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S-8A SEng PRP Training Program

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1

Types of Piers

- Pier reference: generally include bent caps and piers both.
- Single pier, multiple pier
- Solid wall, group of piles.
- Integral pier.
- Pile foundation: footings, piles or a combination.
- Don't forget riprap if needed for scour control.

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2

Pier Behavior

- Pier rigidly connected to foundation: cantilever action.
- Rigid bearings supported: wider foundation needed to resist overturning.
- Expansion bearings: narrower foundations.
- Any bearing friction should be considered.
- To consider: braking force, wind loads, water loads, seismic loads, debris loads, centrifugal loads, impact…
- Consider load combinations.
- Consider slenderness and end conditions.

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3

Design Considerations

- Overturning
- Sliding
- Soil bearing
- Differential settlement
- Structural failure
- Scour effect
- Software options
- Longitudinal vs. transverse load
- Critical loading for bent cap, pier and foundation
- Aesthetics

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4

Sample Piers

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Sample Piers

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6

Sample Piers

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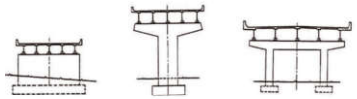
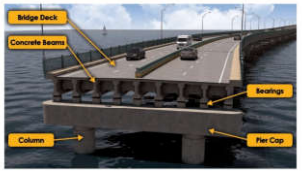
Bent Cap Design





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8

Bent Cap Design

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9

Sources of Bridge Design Examples and References

- PCA (www.cement.org)
  LRFD Design of Cast-in-Place Concrete Bridges
  AASHTO LRFD Strut & Tie Model Design Examples
- PCI (www.pci.org)
  Bridge Design Manual
- AASHTO (bridges.transportation.org)
- FHWA National Highway Institute (www.fhwa.dot.gov/bridge)
  Comprehensive LRFD Bridge Design Examples (2)

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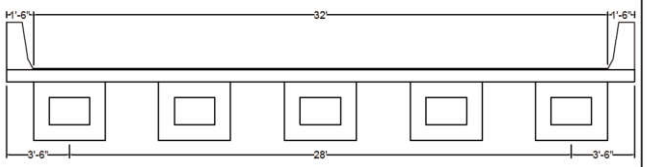
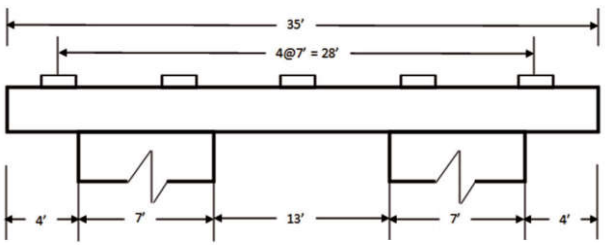
Bent Cap Design Example

For the bridge superstructure shown, design the Bent Cap. You may use CAP18 software or other similar avenues.



11

Bent Profile



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Example

Cap Dimensions

Cap Width - 3 ft. 6 in.; Cap Depth - 3 ft. 6 in., Cover - 2.5 in.

Cap Concrete

f'_c = 6 ksi; w_c = 0.150 kcf

Column Width - 3 ft. 6 in.

Girder span = 89 ft. with two spans

Girder section = 812.5 in^2

Deck thickness = 8 in.; w_c = 0.145 kcf

Barrier = 0.3 kip/ft

Wearing surface = 0.5 in. with 0.145 pcf

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Example

Dead Load Effects
(point reaction loads from girders)

Exterior and Interior Girders:

$$DC = \underbrace{\frac{812.5*0.150*89}{144}}_{\text{Girder DL}} + \underbrace{\frac{8*35*0.145*89}{12*5}}_{\text{Deck DL}} + \underbrace{\frac{2*0.3*89}{5}}_{\text{Barrier DL}} = 146 \text{ kip/girder}$$

$$DW = \frac{0.5*35*0.145*89}{12*5} = 3.76 \text{ kip/girder}$$

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Example

Live Load Effects:

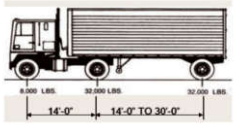
Limit States

Strength I	Service I
LL+IM: 1.75	LL+IM: 1
DC: 1.25	DC & DW: 1
DW: 1.5	

•Multiple presence factor

Multiple Presence Factors	
Number of Lanes Loaded	Multiple Presence Factor, m
1	1.20
2	1.00
3	0.85
>3	0.65

AASHTO 3.6.1, Table 3.6.1.1.2-1, page 321



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Example

Application of Live Loads

Design Lanes –10 ft. wide

Lane loading

-10 ft. wide

-Located within design lane

Truck loading

-6 ft. transverse wheel spacing

-Located within design lane

-Wheel located not less than 2 ft. from design lane edge

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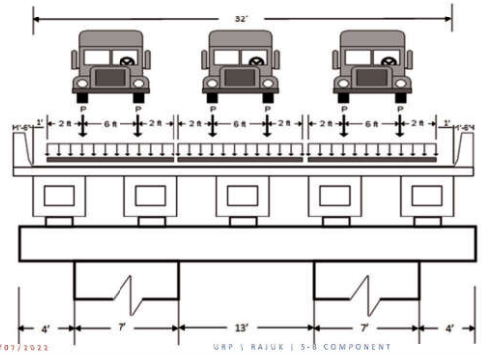
Example



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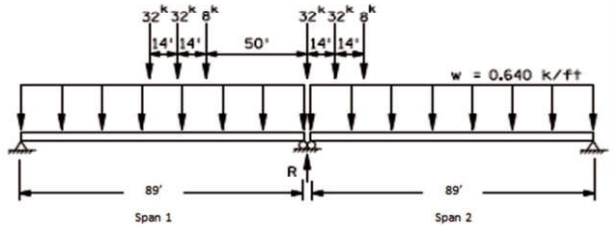
- For negative moment between points of contraflexure under a uniform load on all spans, and reaction at interior piers only, 90 percent of the effect of two design trucks spaced a minimum of 50.0 ft between the lead axle of one truck and the rear axle of the other truck, combined with 90 percent of the effect of the design lane load. The distance between the 32.0-kip axles of each truck shall be taken as 14.0 ft. The two design trucks shall be placed in adjacent spans to produce maximum force effects.

AASHTO 3.6.1.3.1



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Example

Girder Reactions from superstructure analysis

$R_{truck} = \left(\frac{32*11+32*25+8*39+32*89+32*75+8*61}{89}\right) * 1.2 = 97.1$ k

$R_{lane} = \frac{2*0.64*89}{2} * 1.2 = 68.35$ kip

Wheel Loads:

$R_{truck} = 97.1$ kip

$P_{wheel} = R_{truck}/2 \times (1+IM) \times 0.90 = 58$ kip

IM = 0.33

0.9 reduction for interior piers (3.6.1.3.1)

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Example

Equivalent uniform lane loads:

$R_{lane} = 68.35$ kip

$W_{lane} = R_{lane}/10 \times 0.9 = 6.15$ kip/ft

Weight of bent cap = $\frac{42*42*0.15}{144*2} = 0.92$ kip/pier

Braking Load Effects – BR: Greater of (3.6.4)

25% of axle weights (truck or tandem)

0.25 * (32 + 32 + 8) = 18 kip (truck) [Controls]

0.25 * (25 + 25) = 12.5 kip(tandem)

3.6.4—Braking Force: *BR*

The braking force shall be taken as the greater of:

- 25 percent of the axle weights of the design truck or design tandem, or
- Five percent of the design truck plus lane load or five percent of the design tandem plus lane load

This braking force shall be placed in all design lanes which are considered to be loaded in accordance with Article 3.6.1.1.1 and which are carrying traffic headed in the same direction. These forces shall be assumed to act horizontally at a distance of 6.0 ft above the roadway surface in either longitudinal direction to cause extreme force effects. All design lanes shall be simultaneously loaded for bridges likely to become one-directional in the future.

The multiple presence factors specified in Article 3.6.1.1.2 shall apply.

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Example

5% of (axle weights of truck + lane loading) =

0.05 * [(32 + 32 + 8) + (0.64 * 178)] = 9.3 kip

5% of (axle weights of tandem + lane loading)

0.05 * [(25 + 25) + (0.64 * 178)] = 8.2 kip

Braking Force per Bearing

BR = 18/ 5 = 3.6 kip/bearing

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Example

Wind Load Effects – WS

Calculate Horizontal Wind Pressure (3.8.1, page 3-40):

Use base wind velocity [Assuming bridge height < 30 ft.]

Assumed $V_b = 115$ mph

Determine base wind pressure (ignoring various factors)

$-P_b = 0.00256\ (115)^2 = 0.034$ ksf

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Example

Calculate Wind Pressure on Superstructure:

Superstructure Depth

H_{super}= Barrier Height + Deck thickness + Haunch thickness + Girder Height

= 3.5 + 0.667 + 0.25 [Assumed] + 3.25 =7.67 ft.

WS = 0.034 ksf x 7.67 ft. = 0.26 kip/ft. < 0.3 kip/ft

Use WS = 0.3 k/ft. or 0.039 ksf

Example

Calculate Wind Load Force Effects – WS

Use coefficients from Table 3.8.1.2.3a -1 (page 3-49)

Example for 60^0 wind angle:

$-WS_{trans} = A_{trans}$ x 0.34 x 0.039 = 682 x 0.34 x 0.039 = 9.08 kip

$-WS_{long} = A_{long}$ x 0.38 x 0.039 = 1365 x 0.38 x 0.039 = 20.23 kip

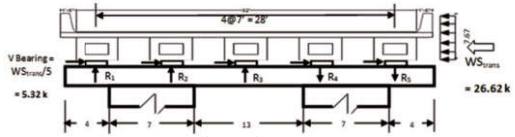
Table 3.8.1.2.3a-1—Skew Coefficients for Various Skew Angles of Attack

Skew Angle (degree)	Trusses, Columns, and Arches		Girders	
	Transverse Skew Coefficient	Longitudinal Skew Coefficient	Transverse Skew Coefficient	Longitudinal Skew Coefficient
0	1.000	0.000	1.000	0.000
15	0.933	0.160	0.880	0.120
30	0.867	0.373	0.820	0.240
45	0.627	0.547	0.660	0.320
60	0.320	0.667	0.340	0.380

S. K. Ghosh Associates LLC
International Code Council (ICC)

Example

Pier Design WS Force Effects		
Wind Attack Angle	Bridge Transverse Axis	Bridge Longitudinal Axis
Degree	kip	kip
0	26.62	0
15	23.43	6.39
30	21.83	12.78
45	17.57	17.04
60	9.08	20.23



Example

Calculate Bearing Reactions for Wind Force Effects -WS

0^0 wind attack angle

$M_{trans} = WS_{trans}$ x (H_{super}/2) = 26.62 k x 7.67/2 = 102 kip-ft.

$I_{box\ girders} = I_g + Ad^2 = \Sigma d^2 = 2(14')^2 + 2(7')^2 = 490\ ft^2$

$R_1 = R_5$ = (102 k-ft x 14')/490 ft^2 = 2.92 kip

$R_2 = R_4$ = (102 k-ft x 7')/490 ft^2 = 1.46 kip

Example

Vertical Wind Load Effects

Calculate Bearing Reactions for Vertical Wind Effects -WS

Resultant wind loading at Pier

From 3.8.2,

A vertical upward wind force of

0.02 ksf times the width of the deck

= 0.02 ksf * 35 ft. = 0.7 kip/ft.

Eccentricity: ¼ x width = 8.75 ft.

3.8.2—Vertical Wind Load

The effect of forces tending to overturn structures, unless otherwise determined in Article 3.8.3, shall be calculated as a vertical upward wind load equal to:

- 0.020 ksf for Strength III load combination and
- 0.010 ksf for Service IV load combination

times the width of the deck, including parapets and sidewalks, shall be applied as a line load. This force shall be applied only when the direction of horizontal wind is taken to be perpendicular to the longitudinal axis of the bridge. This line load shall be applied at the windward quarter-point of the deck width in conjunction with the horizontal wind loads specified in Article 3.8.1.

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Example

-Vertical force = 0.7 kip/ft x tributary length = 0.7 x 89 = 62.3 kip

-Moment = Vert. force x eccentricity = 62.3 x 8.75' = 545 k-ft.

-Reaction = P/A ± (M x C)/I

R1 = -62.3 k / 5 + (545 k-ft x 14')/ 490 ft.^{2} = 3.1 kip

R2 = -62.3 k / 5 + (545 k-ft x 7')/490 ft.^{2} = -4.67 kip

R3 = -62.3 k / 5 = -12.46 k

R4 = -62.3 k / 5 - (545 k-ft x 7')/490 ft.^{2} = -20.2 kip

R5 = -62.3 k / 5 - (545 k-ft x 14')/490 ft.^{2} = -28.0 kip

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Example

Calculate Horizontal Wind Force on cap and pier (3.8.1.2.3)

Base Wind Pressure P_b= 0.05 ksf

Calculate Substructure Tributary Area

Pier Cap Tributary Area

–Cap Front Area = 3.5' x 35' = 122 ft^2

–Cap End Area = 3.5' x 3.5' = 12.2 ft^2

Pier Column Tributary Area [Assuming pier height of 15 ft.]

–Column Front Area = 15' x 7' = 105 ft.^{2}

–Column End Area = 15' x 3.5' = 52.5 ft.^{2}

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Example

Attack Angle, deg.	AP_{cap}, ft^2	Ap_{col}, ft^2	Total Wind Load, kip	Ws_{tran}, kip	Ws_{long}, kip
0	12.25	60	3.61	3.6	0
15	43.54	85.1	6.43	6.2	1.67
30	71.86	104.5	8.82	7.6	4.4
45	95.3	116.7	10.6	7.5	7.5

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Example

Wind on Live Load (3.8.1.3):

Transverse Tributary Length = ½ (Span 1 + Span 2) = 89'

Longitudinal Tributary Length = Span 1 + Span 2 = 178'

Calculate Wind Load Force Effects –WL

Table 3.8.1.3-1—Wind Load Components on Live Load

Skew Angle (degrees)	Transverse Component (kif)	Longitudinal Component (kif)
0	0.100	0.000
15	0.088	0.012
30	0.082	0.024
45	0.066	0.032
60	0.034	0.038

30-degree wind attack angle

$WL_{trans} = L_{trans}$ x 0.082 k/ft. = 89 ft x 0.082 = 7.3 k

$WL_{long} = L_{long}$ x 0.024 k/ft. = 178 ft x 0.024 = 4.3 k

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Example

Calculate Bearing Reactions for Wind on LL –WL

30 deg. wind attack angle

$M_{trans} = WL_{trans}$ x Height = 7.3 x (7.67'+ 2.5') = 74 kip-ft

$R_1 = R_5$ = (74.24 x 14')/490 = 2.12 kip

$R_2 = R_4$ = (74.24 x 7')/490 = 1.06 kip

Temperature Force Effects -TU

Assume a 20-kip total force: TU = 20k

TU per bearing = TU/5 bearings = 4.0 kip/bearing (3.12.2)

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CAP 18 Output and Load Combinations

	Max + M	Max - M	Max + Shear	Max - Shear
Dead Load	546 kip-ft	-1251 kip-ft	313 kip	-467 kip
Service	2281 kip-ft	-1912 kip-ft	712 kip	-478 kip
Ultimate	3380 kip-ft	-2727 kip-ft.	1016 kip	-682 kip

	Load Factors							
	Strength I		Strength III		Strength V		Service I	
Load	γ_{max}	γ_{min}	γ_{max}	γ_{min}	γ_{max}	γ_{min}	γ_{max}	γ_{min}
DC	1.25	0.90	1.25	0.90	1.25	0.90	1.00	1.00
DW	1.50	0.65	1.50	0.65	1.50	0.65	1.00	1.00
LL	1.75	1.75	---	---	1.35	1.35	1.00	1.00
BR	1.75	1.75	---	---	1.35	1.35	1.00	1.00
TU	1.20	0.50	1.20	0.50	1.20	0.50	1.20	1.00
WS	---	---	1.40	1.40	0.40	0.40	0.30	0.30
WL	---	---	---	---	1.00	1.00	1.00	1.00
EV	1.35	1.00	1.35	1.00	1.35	1.00	1.00	1.00

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Example

Governing Load Combination:

Strength I

DC * 1.25 + DW * 1.5 + (LL+IM) * 1.75 + BR * 1.75+ TU * 1.2 + EV * 1.35

Flexural Reinforcement:

Assuming Maximum Moments:

M_{dl} = 1251 kip-ft.

M_u = 3380 kip-ft.

34

Example

Try 14-#11 bars Top and Bottom of bent cap:

A_s = 21.84 in^2 $d_{stirrup}$ = 0.625 in

d_s = cap depth – cover – $d_{stirrup}$ - d_{bar} /2 = 38.4 in

b = cap width = 42 in; c = 4.11 in; a = c * β_1 = 3.5 in

M_n = 3977 kip-ft;

M_r = ϕ *M_n= 0.9*3977 = 3579 kip-ft > M_u [OK]

Example

Skin Reinforcement (5.7.3.4.2)

No. of # 5 rebars = 5; Bar Area = 0.31 in^2

A_{sk} = 1.55 in^2

h_{skin} = 34.97 in = 2.91 ft

$A_{sk(reqd)}$ = 0.286 in^2 < A_{sk} [OK]

Actual Spacing of Skin Reinforcement

s_{sk} = 5.83 in

Required Spacing of Skin Reinforcement

S_{sk_req} = 6.36 in > s_{sk} [OK]

Use 5 #5 on each side of bent

Example: Bridge Pier Design

The columns are fixed at the top and bottom in both directions.

Design Data

Concrete strength $f_c' = 3.5\ ksi$

Yield strength of reinforcement $f_y = 60\ ksi$

Factored loads for the Extreme Event I load combination for each column are shown below.

$$M_u = 4050\ kip - ft$$

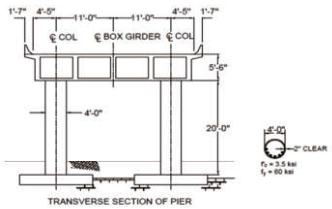
$$V_u = 600\ kip$$

$$P_u = 2700\ kip\ (compression)$$

Example: Bridge Pier Design

(a) For the Extreme Event I load combination, determine the vertical reinforcing required for the column. Neglect any slenderness effect.

(b) For the Extreme Event I load combination, design the required spiral reinforcing.



Example: Bridge Pier Design

Design for moment and axial force

- Column requirement
  (5.11.4.1, 5-193)

$h/D_c =$
$ratio\ of\ clear\ height\ to\ maximum\ plan\ dimension\ of\ column$
$= 20/4 = 5 > 2.5$

The column qualifies to be designed as a column and not as a pier.

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Example: Bridge Pier Design

Vertical reinforcement, flexural strength

Design for $M_u = 4050\ kip - ft$ and $P_{u\,max} = 2700\ kip$

$$\phi = 0.9$$

$$M_n = nominal\ moment = \frac{M_u}{\phi} = \frac{4050}{0.9} = 4500\ kip - ft$$

5.11.4.1.2—Flexural Resistance

The biaxial strength of columns shall not be less than that required for flexure, as specified in Article 3.10.9.4. The column shall be investigated for both directional combinations of force effects specified in Article 3.10.8, at the extreme event limit state. The resistance factors of Article 5.5.4.2 shall be replaced for columns with either spiral or tie reinforcement by the value of 0.9.

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$$P_n = nominal\ axial\ force = \frac{P_u}{\phi} = \frac{2700}{0.9} = 3000\ kip$$

From the interaction diagram for $M_n = 4500\ kip - ft$ and $P_u = 3000\ kip$,

$\rho = 0.025$

Limits of vertical reinforcement

5.11.4.1.1—Longitudinal Reinforcement

The area of longitudinal reinforcement shall not be less than 0.01 or more than 0.04 times the gross cross-section area, A_g.

$0.01 < \rho = 0.025 < 0.04\ OK$

$A_s = 0.025 \times \pi(48)^2/4 = 45.24\ in^2$

Using 29 #36 bars with $A_s = 29 \times 1.56 = 45.24\ in^2 \approx$
$29187\ mm^2\ will\ staisfy\ the\ requirement$

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Example: Bridge Pier Design

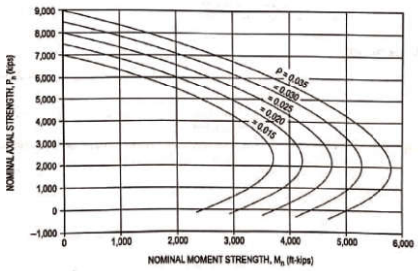

FIGURE 901B

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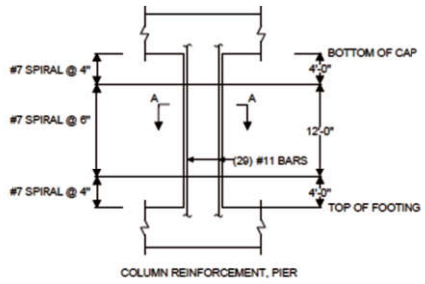
Example: Bridge Pier Design



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Can We Achieve 100+ year Service Life?

Extend Service Life Through:

- Structural Strategies
- Material Strategies

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Curing

- Continuous Wet Curing Preferred
  - Immediately After Finishing; 7 Days or More
- Types of Wet Curing
  - Ponding
  - Fogging (Initial Curing)
  - Sprinkler/Soaker Hoses
- Often Used With Burlap/Mats or Plastic
- Enhances Performance
  - Concrete Gains Strength Before Drying
  - Reduction in Shrinkage Cracking

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Curing

Fogging

Sprinkler Saturating Burlap

Photos: *Design and Control of Concrete Mixtures*, by Kosmatka, Kerkhoff, and Panarese, 14th Edition, CD100, Portland Cement Association, 2009.

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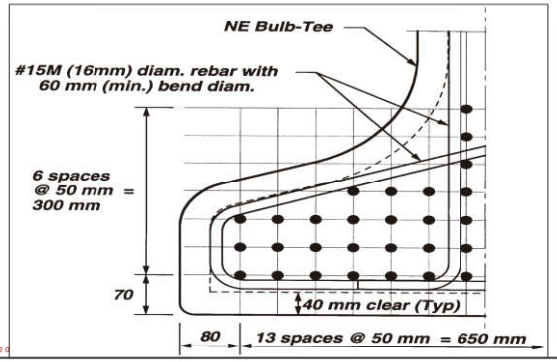
Cover



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Minimum Concrete Cover

AASHTO LRFD Article 5.10.1

SITUATION	COVER (IN)
Direct exposure to salt water	4.0
Cast against earth	3.0
Coastal	3.0
Exposure to deicing salts	2.5
Deck surfaces subject to tire stud or chain wear	2.5
Exterior other than above	2.0
Interior other than above • Up to No. 11 bar • No. 14 and No. 18 bars	 1.5 2.0
Bottom of cast-in-place slabs • Up to No. 11 bar • No. 14 and No. 18 bars	 1.0 2.0
Precast soffit form panels	0.8
Precast reinforced piles • Noncorrosive environments • Corrosive environments	 2.0 3.0
Precast prestressed piles	2.0
Cast-in-place piles • Noncorrosive environments • Corrosive environments - General - Protected • Shells • Auger-cast, tremie concrete, or slurry construction	 2.0 3.0 3.0 2.0 3.0

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Minimum Concrete Cover Issues

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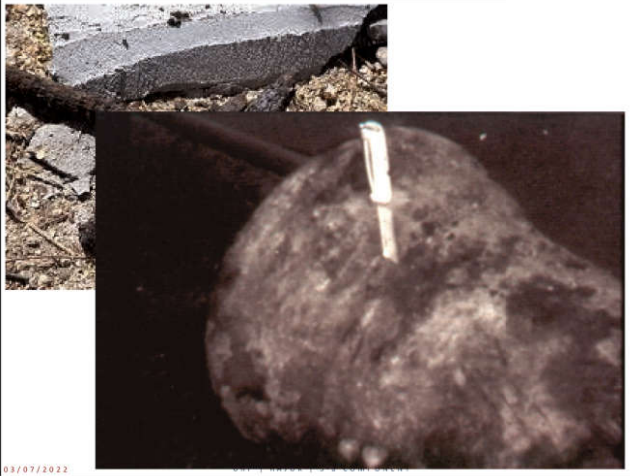

50

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Compressive Strength

Min. f'_c = 2.4 ksi for Structural Applications

Min. f'_c = 4.0 ksi for Decks

Min. f'_c = 4.0 ksi for Prestressed Concrete Members

5.4.2—Normal Weight and Lightweight Concrete

5.4.2.1—Compressive Strength

For each component, the compressive strength of concrete for use in design, f'_c, or the class of concrete shall be shown in the contract documents.

Design concrete compressive strengths above 10.0 ksi for normal weight concrete shall be used only when allowed by specific Articles or when physical tests are made to establish the relationships between the concrete strength and other properties. Concrete with compressive strengths used in design below 2.4 ksi should not be used in structural applications.

The design concrete compressive strength for prestressed concrete and decks shall not be less than 4.0 ksi.

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Compressive Strength

AASHTO LRFD, 5.4.2.1:

Concrete strength > 10 ksi used only when physical tests are made to establish relationships between concrete strength and other properties, or as allowed by other articles in AASHTO LRFD.

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Compressive Strength

AASHTO LRFD:

Provisions valid for $f'_c \leq 15.0$ ksi for calculating:

- Shrinkage and Creep (5.4.2.3.1, p. 5-17)
- Modulus of Elasticity of Concrete (5.4.2.4, p. 5-19)
- Modulus of Rupture of Concrete (5.4.2.6, p. 5-20)
- Prestress Losses (5.9.3.1, p. 5-127)

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Control of Cracking by Distribution of Reinforcement

- Provide for maximum rebar spacings

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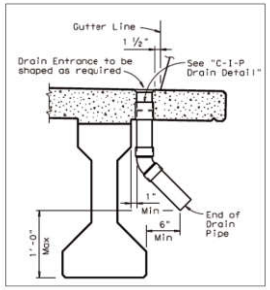
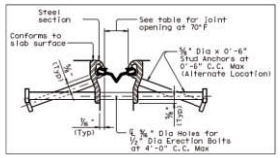
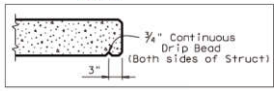
Detailing



Drain Detail



Sealed Joint Detail



Drip Bead Detail

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High Performance Concrete

Concrete That Attains Mechanical, Durability, or Constructability Properties Exceeding Those of Normal Concrete

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High Performance Concrete (HPC)

Increased Durability and Normal Strength

or

Increased Durability and High Strength

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Designing HPC Mix

- Strategic Selection of Quantities
- Low Water–to–Cementitious Materials Ratio
- Key Additions to Basic Concrete Mix
  - Supplementary Cementitious Materials
  - Chemical Admixtures

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Performance Classification for HPC for Bridges

Introduced By FHWA In February, 1996 and Updated in August, 2006

Performance Characteristic Grades 1 – 3

+ Grade 1 = Good Performance
+ Grade 3 = Best Performance

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Performance Characteristic Grades for HPC

Performance Characteristic	Performance Characteristic Guide		
	1	2	3
Freeze/Thaw Durability	70% ≤ F/T < 80%	80% ≤ F/T < 90%	90% ≤ F/T
Scaling Resistance	3.0 ≥ SR > 2.0	2.0 ≥ SR > 1.0	1.0 ≥ SR > 0.0
Abrasion Resistance	2.0 ≥ AR ≥ 1.0 mm	1.0 ≥ AR ≥ 0.5 mm	0.5 mm > AR
Chloride Penetration (coulombs)	2500 ≥ CP > 1500	1500 ≥ CP > 500	500 ≥ CP

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Performance Characteristic Grades for HPC

Performance Characteristic	Performance Characteristic Guide		
	1	2	3
Sulfate Resistance	SR ≤ 0.10 @ 6 mo.	SR ≤ 0.10 @ 1 yr.	SR ≤ 0.10 @ 1.5 yr.
Alkali-Silica Reactivity	0.20 ≥ ASR > 0.15	0.15 ≥ ASR > 0.10	0.10 ≥ ASR
Workability—Slump	SL > 7½ in		
Workability—Slump Flow (SCC)	SF < 20 in	20 ≤ SF ≤ 24 in	24 in < SF

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Performance Characteristic Grades for HPC

Performance Characteristic	Performance Characteristic Guide		
	1	2	3
Strength (ksi)	$8 \le f'_c < 10$	$10 \le f'_c < 14$	$14 \le f'_c$
Elasticity (x 10^6 psi)	$5 \le E_c < 6$	$6 \le E_c < 7$	$7 \le E_c$
Shrinkage (microstrain)	$800 > S \ge 600$	$600 > S \ge 400$	$400 > S$
Creep (microstrain/psi)	$0.52 \ge C > 0.38$	$0.38 \ge C > 0.21$	$0.21 \ge C$

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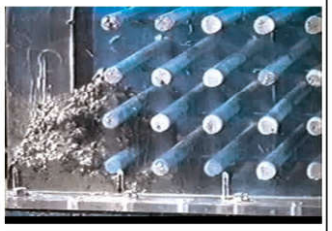
Self Consolidating Concrete



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I-35W St. Anthony Falls Bridge

Photo Courtesy of Minnesota DOT

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Ultra-High Performance Concrete

- Very High Strength Concrete ≥ 20 ksi
- Research by FHWA—Turner Fairbanks
- Increased Durability

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Corrosion-Resistant Reinforcement

- Epoxy-Coated
- Hot-Dip Galvanized
- Dual-Coated
- Stainless Steel
- Low-Carbon, Chromium
- Stainless-Clad
- Fiber-Reinforced Polymer (FRP) Composite

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Corrosion-Resistant Reinforcement

68

Epoxy-Coated Reinforcing Steel

Photo: *Design and Control of Concrete Mixtures*, by Kosmatka, Kerkhoff, and Panarese, 14th Edition, CD100, Portland Cement Association, 2009.

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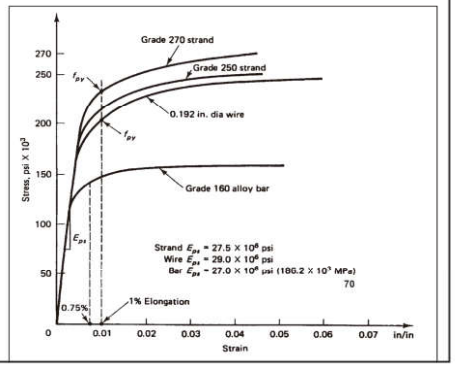
Typical Stress-Strain Curves for Prestressing Steel

Grade 270 Typical



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Plastic Duct

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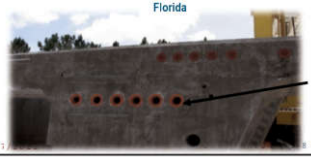
GTI Segmental Duct Coupler

I-95 / I-295 Interchange, Jacksonville, Florida

Match Cast Coupler



Bulkhead Coupler

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VSL Multi-Strand Anchorage System

System Pressure Testing

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Permanent Grout Cap

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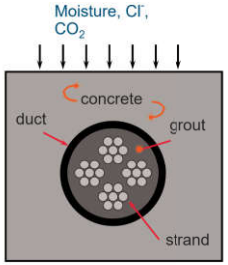
Grout for Post-Tensioning

- Extremely Durable When Properly Constructed
  - Crack Control
  - Multiple Levels of Protection
- *The Grout is the Third Level of Protection for the Strand*



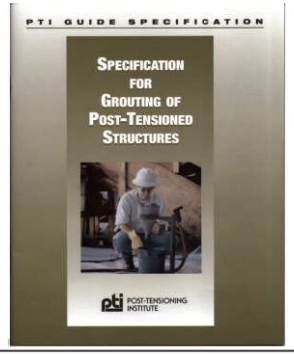
Keys to a Good Grout

- Completely Fills Duct
- Low Permeability
- Appropriate Bleed Resistance
- Careful Use of Admixtures
  - May Enhance One Property While Degrading Another

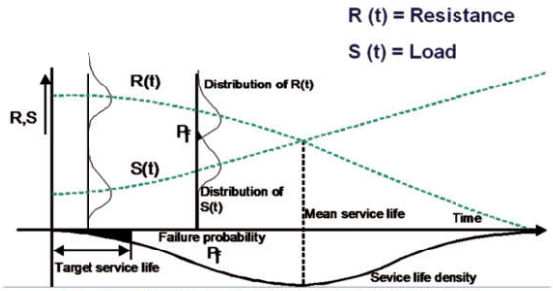
Specifications for Grouting Materials

Post-Tensioning Institute

- PTI Guide Specifications
  - Section 2: Materials

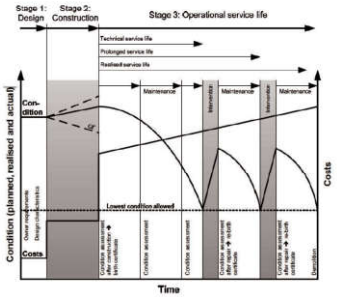


Failure Probability and Service Life



Failure Probability and Service Life, *fib* Bulletin 34, "Model Code for Service Life Design"

Complete Service Life



Complete Service Life, "Condition Control of Existing Structures by Performance Testing" by Gehlen, C., Dauberschmidt, C., and Nurnberger, U.

Design for Service Life

- Strategic Highway Research Program (SHRP 2)
  - SHRP 2 R19(A), "Bridges for Service Life Beyond 100 Years: Innovative Systems, Subsystems, and Components"
    - Status: Active
  - SHRP 2 R19(B), "Bridges for Service Life Beyond 100 Years: Service Limit State Design"
    - Status: Active

For More Information, Visit www.trb.org

Conclusions

Design for 100+ Year Life Today

Structural Strategies for Durability

+

Material Strategies for Durability

=

Durable Concrete Bridges

Points to Remember

- AASHTO LRFD Bridge Design Specifications Provides for 75 Year Design Life
- Design Life Can be Extended by Selecting Appropriate Materials, Design and Construction approaches for the Service Environment
- Full Probabilistic Design is Limited at this time due to Inaccuracies in Deterioration Modeling
- Achieving a 100+ Year Bridge Life is Possible Using Service Life Design Principles Along with Timely Intervention Strategies

QUESTIONS?

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THE WORLD BANK

STRUCTURAL DESIGN OF RAILWAY BRIDGES

Date: 06 February 2022

Session 44 : Bridge 6-Railway Bridge

SMEC ACE Consultants Ltd.

Engr. ASM Anisuzzaman, MEng. (Structural)
Sr. Structural & Bridge Design Engineer
AGM Design, ACE Consultants Ltd. Bangladesh

S-8A SEng PRP Training Program

INTERNATIONAL CODE COUNCIL

1

Rail Lines Owner in Bangladesh

DHAKA MASS TRANSIT COMPANY LIMITED

- Bangladesh Railway (BR), Ministry of Railway
- Dhaka Mass Transit Company Limited (DMTCL) for MRT Line Ministry of Road Transport and Bridges

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2

Course Contents on Structural Design of Railway Bridges

- Railway Bridge under Bangladesh Railway, Ministry of Railway, GoB.
- Design Criteria as per TOR / Clients Requirement
- Types of Railway Bridge used in Bangladesh
- Loads on bridge Superstructures (IRS Bridge Rules)
- Seismic Load (IRS Seismic Code)
- Loads of bridge Substructures (IRS Substructures Rules)
- IRS Steel Bridge Code
- Fatigue Requirement for Steel Bridges
- Analysis and Design for Steel Stringer of STPG

3

Standards Requirement for BR

❑ General Standards for Alignment, Embankment, Track, Bridge etc. (From TOR)

- ❑ Bangladesh Railway (BR)
- ❑ Indian Railway Standards (IRS)
- ❑ Indian Standards (IS)
- ❑ American Society for Testing Materials (ASTM)
- ❑ International Union for Railway (UIC)
- ❑ European (EN)
- ❑ American Railway Engineering and Maintenance Association (AREMA)

4

Standards Requirement for BR

❑ General Standards for Alignment, Embankment, Track, Bridge etc. (From TOR)

Design parameters and criteria adopted for the design of rail alignment and track structure are based on:

- BRWWM (Bangladesh Railway Way and Works Manual, 2013);
- BR SOD BG (Bangladesh Railway Schedule of Dimensions for BG);
- BR SOD MG (Bangladesh Railway Schedule of Dimensions for MG);
- BRLWRM (Bangladesh Railway Manual of Instructions on Long Welded Rails, 2013);
- Instructions and policy circulars issued by Bangladesh Railway;
- Practices adopted by Indian and other Railways; and
- Modifications to suit DG track.

5

Standards Requirement for BR

❑ Standards for Bridges and Culverts Design (From TOR)

a) IRS Standard-Rules Specifying the Loads for Design of Super-structure and Sub-structure of Bridges (Bridge Rules);
b) IRS Standard-Code of Practice for the Design of Steel or Wrought Iron Bridges Carrying Rail, Road or Pedestrian Traffic (Steel Bridge Code);
c) IRS Standard-Code of Practice for Metal Arc Welding in Structural Steel Bridges Carrying Rail, Rail-cum-Road or Pedestrian Traffic (Welded Bridge Code);
d) IRS Standard-Code of Practice for Plain, Reinforced and Pre-stressed Concrete For General Bridge Construction (Concrete Bridge Code);
e) IRS Standard-Code of Practice for The Design of Sub-Structures and Foundations of Bridges (Bridge Sub-structures and Foundation Code);
f) Indian Railways Bridge Manual-1998; and
g) Indian Roads Congress (IRC) Standard: 89-1997 Guidelines for Design and Construction of River Training and Control Works for Road Bridges.

6

Standards Requirement for BR

❑ Standards for Bridges and Culverts

Railway Master Plan. All bridges, culverts, and substructures have to be designed in accordance with modified BGML loading (IRS 2008, 25-ton axle load) considering double stacking of containers, future electric traction lines, and oversized consignments

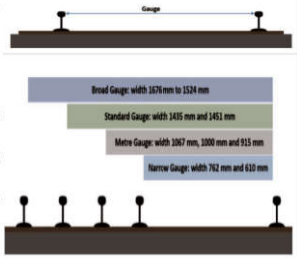
1.3.1 Design Life

In accordance with the design codes, all structures shall be designed for a design life of 100 years.

7

Track Gauge for BR Different Types

- Broad Gauge : width 1676 mm to 1524 mm or 5'6" to 5'0" (BR)
- Standard Gauge: width 1435 mm and 1451 mm or 4'-8½" (DMRT)
- Metre Gauge: width 1067 mm, 1000 mm and 915 mm or 3'-6", 3'-33/8" and 3'-0" (BR)
- Narrow Gauge: width 762 mm and 610 mm or 2'-6" and 2'-0".
- Dual Gauge: BG+MG combined (3 Rails with 1 common rail) (BR)



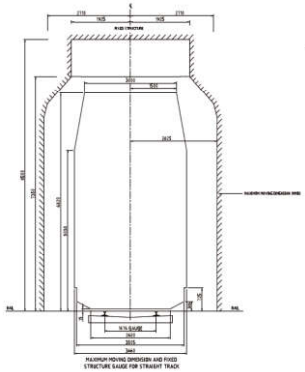
Source Internet

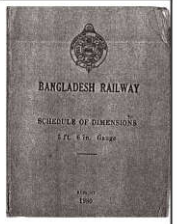
8

Schedule of Dimension for BR BG Track Bridge

- Horizontal clearance 2825*2= 5650mm
- Vertical Clearance 7350mm
- Viaduct 8500mm

(BR Circular)





9

Types of Railway Bridge

❑ Types are:
- Minor Bridge
- Major Bridge
- Important Bridge
- Box Culvert
- DTPG
- STPG
- TT (Open Web)

2.12 MAJOR BRIDGES are those which have either a total waterway of 18m or more or which have a clear opening of 12m or more in any one span.

2.11 IMPORTANT BRIDGES are those having a lineal waterway of 300m or a total waterway of 1000 Sq.m or more and those classified as important by the Chief Engineer/ Chief Bridge Engineer, depending on considerations such as depth of waterway, extent of river training works and maintenance problems.

Source: IRS Substructure Rules

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Types of Railway Bridge
Box Culverts

- Span Length 2m to 6m and Length up to 15m multi cell box culvert
- Used for Shallow depth, no navigation required
- Less cost
- Smooth riding as like
- Embankment

Source: Internet

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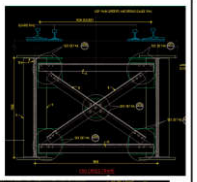
Types of Railway Bridge
Deck Type Plate Girder (DTPG)

- Rail line is above the steel Girder
- Span length from 10m to 20m
- Not suitable where navigation is required
- Less cost than that of STPG
- It can be open deck or Ballasted Deck



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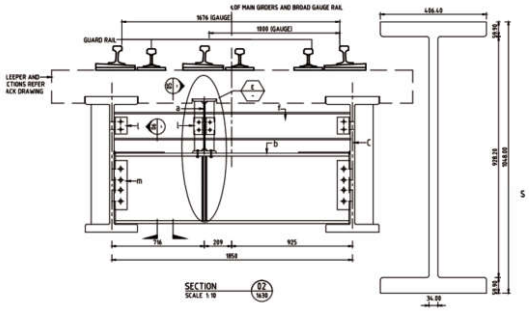
Types of Railway Bridge
Deck Type Plate Girder (DTPG)



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Types of Railway Bridge
Deck Type Plate Girder (DTPG)

Fabrication of DTPG in India for KGTP and MDP

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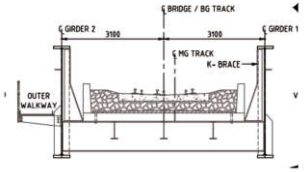
Types of Railway Bridge
Semi-Through Plate Girder (STPG)

- Rail Line passes inside the girder depth
- Span kept normally from 15m to 35m
- Suitable for navigable stream
- Need to follow the schedule of dimension
- Can be open deck or ballasted deck
- No Extra sound and Riding smooth for ballasted deck



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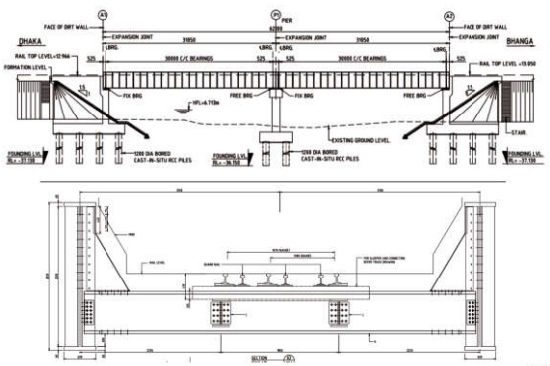
Types of Railway Bridge
Semi-Through Plate Girder (STPG)



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Types of Railway Bridge
Semi-Through Plate Girder (STPG)

Semi Through Plate Girder
RBCP Parbatipur- Panchagarh
Under Construction Line (Opened now)

Fabricated at Macdonalds, Gazipur 35m span STPG Under Fabrication

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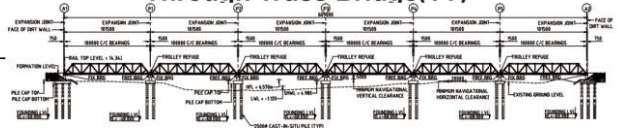
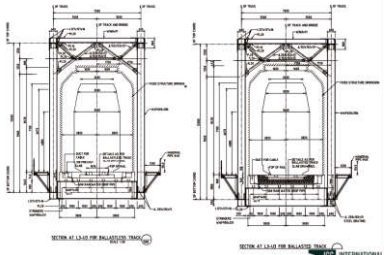
Types of Railway Bridge
Through Truss Bridge(TT)



- Span 35m to 100m or more,
- Long span needs for Navigation Clearance,
- It can be ballasted, Ballast less, Open deck.



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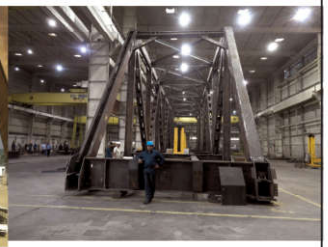
Types of Railway Bridge
Through Truss Bridge(TT)

TT Bridge Fabricated in China
Weathered Steel on Feni River



TT Bridge Fabricated in India
For KGTP

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Types of Railway Bridge
Through Truss Bridge(TT)

TT Bridge Padma Rail Link Project

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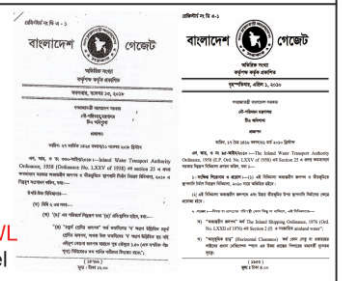
Navigation Clearance
BIWTA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- Vertical Clearance from SHWL (not HFL) to Girder soffit level
- NC Needs for span arrangement

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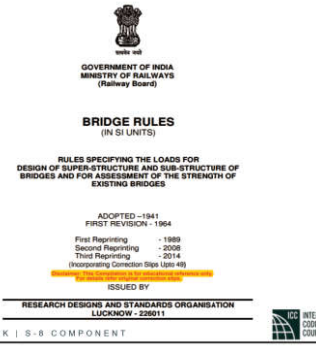
IRS Bridge Rules

- Used for different types of Loads
- Corrected up to Slips 49 in 2017


GOVERNMENT OF INDIA
MINISTRY OF RAILWAYS
(Railway Board)

BRIDGE RULES
(IN SI UNITS)

RULES SPECIFYING THE LOADS FOR DESIGN OF SUPER-STRUCTURE AND SUB-STRUCTURE OF BRIDGES AND FOR ASSESSMENT OF THE STRENGTH OF EXISTING BRIDGES

ADOPTED – 1941
FIRST REVISION - 1964

First Reprinting - 1989
Second Reprinting - 2008
Third Reprinting - 2014
(Incorporating Correction Slips Upto 49)

ISSUED BY

RESEARCH DESIGNS AND STANDARDS ORGANISATION
LUCKNOW - 226011

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IRS Bridge Rules
Loads 2.1

- Dead load
- Live load
- Dynamic effects
- Forces due to curvature or eccentricity of track
- Temperature effect
- Frictional resistance of expansion bearings
- Longitudinal force
- Racking force
- Forces on parapets
- Wind pressure effect (BNBC)
- Forces and effects due to earthquake (BNBC)
- Erection forces and effects
- Derailment loads
- Load due to Plasser's Quick Relay System (PQRS)
- Forces due to accidental impact from any vehicles such as road vehicles, ships or derailed train vehicles using the bridge (ACS 48 dtd. 22.06.17)

IRS Bridge Rules
Dead Load 2.2

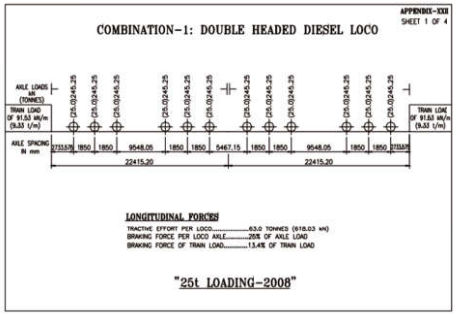
- 2.2.1 Dead load is the weight of the structure itself together with the permanent loads carried thereon.
- Super Imposed Dead Load: Dead Load over the structure to be designed

- 2.2.2 For design of ballasted deck bridges, a ballast cushion of 400mm for BG and 300mm for MG shall be considered. However, ballasted deck bridges shall also be checked for a ballast cushion of 300mm on BG and 250mm on MG.

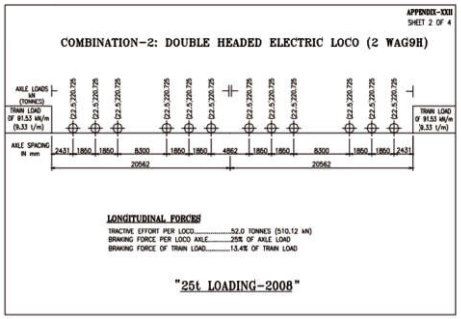
IRS Bridge Rules
2.3 Live Load

- (a) For Broad Gauge - 1676mm – "25t Loading-2008" with a maximum axle load of 245.2 kN (25.0t) for the locomotives and a train load of 91.53 kN/m (9.33t/m) on both sides of the locomotives (Appendix-XXII)
- (2) Diagrams of Standard loading and Equivalent Uniformly Distributed Loads (EUDL) on each track for calculating Bending Moment and Shear Force are shown in the accompanying Appendices XXII, XXIII & XXIII (a) respectively
- (4) EUDLs shall be used for simply supported spans. In case of continuous super-structures over supports, the Bending Moments and Shear Forces for design purposes at various sections shall be computed for loadings shown in Appendix-XXII.

IRS Bridge Rules
Live Load 2.3



IRS Bridge Rules
Live Load 2.3



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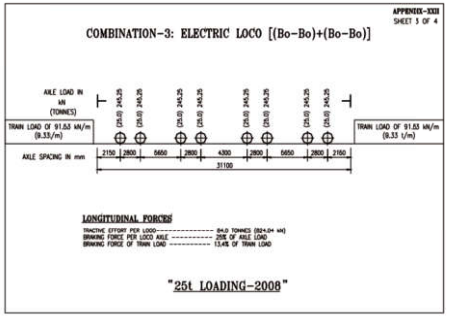
IRS Bridge Rules
Live Load 2.3



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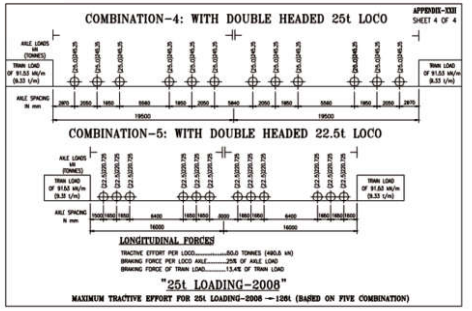
IRS Bridge Rules
Live Load 2.3



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IRS Bridge Rules, Live Load 2.3

APPENDIX-XXIII

"25t Loading-2008"

BROAD GAUGE-1676 mm

Equivalent Uniformly Distributed Loads (EUDL) in kão Newtons (tonnes) on each track, and Coefficient of Dynamic Augment (CDA).

For Bending Moment, L is equal to the effective span in metres. For Shear Force, L is the loaded length in metres to give the maximum Shear Force in the member under consideration.

The Equivalent Uniformly Distributed Load (EUDL) for Bending Moment (BM), for spans upto 10m, is that uniformly distributed load which produces the BM at the centre of the span equal to the absolute maximum BM developed under the standard loads. For spans above 10m, the EUDL for BM is that uniformly distributed load which produces the BM at one-sixth of the span equal to the BM developed at that section under the standard loads.

EUDL for Shear Force (SF) is that uniformly distributed load which produces SF at the end of the span equal to the maximum SF developed under the standard loads at that section.

NOTE:

(1) *Cross girders – The live load on a cross girder will be equal to half the total load for bending in a length L, equal to twice the distance over centres of cross girders.*

(2) *L for Coefficient of Dynamic Augment (CDA) shall be as laid down in clause 2.4.1.*

(3) *When loaded length lies between the values given in the table, the EUDL for Bending Moment and Shear Force can be interpolated.*

L (m)	Total load for Bending Moment		Total load for Shear Force		Impact Factor CDAe
	kN	t	kN	t	0.15+6/(6+L)
1	2	3	4	5	6
1.0	490.50	50.00	490.50	50.00	1.000
1.5	490.50	50.00	490.50	50.00	1.000
2.0	490.50	50.00	527.00	53.75	1.000
2.5	490.50	50.00	618.03	63.00	1.000
3.0	490.50	50.00	678.58	69.17	1.000
3.5	531.02	54.13	721.72	73.57	0.992
4.0	595.96	60.75	790.98	80.63	0.950
4.5	676.89	69.00	866.52	88.33	0.912
5.0	745.58	76.00	927.05	94.50	0.877
5.5	811.58	82.73	976.59	99.55	0.846
6.0	866.52	88.33	1017.79	103.75	0.817
6.5	913.11	93.08	1052.81	107.32	0.790
7.0	952.94	97.14	1088.75	110.78	0.765
7.5	987.57	100.67	1119.42	114.11	0.743
8.0	1017.79	103.75	1148.37	119.10	0.721
8.5	1044.47	106.47	1214.67	123.82	0.702
9.0	1068.21	108.89	1265.49	129.00	0.683
9.5	1089.40	111.05	1315.03	134.05	0.666
10.0	1108.53	113.00	1377.32	140.40	0.650
11.0	1262.66	130.75	1492.89	152.18	0.621
12.0	1377.32	140.40	1589.22	162.00	0.594
13.0	1475.13	150.37	1670.74	170.31	0.571
14.0	1558.91	158.91	1740.59	177.43	0.550
15.0	1631.60	166.32	1813.48	184.86	0.531
16.0	1706.41	174.15	1905.46	194.23	0.514
17.0	1819.56	185.48	1997.12	203.58	0.498
18.0	1889.60	192.64	2088.94	212.94	0.483
19.0	1978.28	201.66	2180.86	222.29	0.470
20.0	2066.50	210.55	2272.42	231.64	0.458
21.0	2151.63	219.33	2364.21	241.00	0.446
22.0	2236.88	228.02	2455.84	250.34	0.436
23.0	2321.34	236.63	2547.46	259.68	0.426
24.0	2405.22	245.18	2639.18	269.03	0.417
25.0	2489.40	253.68	2730.81	278.37	0.408
26.0	2571.00	262.08	2822.44	287.71	0.400
27.0	2653.21	270.46	2914.06	297.05	0.392
28.0	2735.03	278.80	3005.69	306.39	0.385
29.0	2816.32	287.09	3097.31	315.73	0.379
30.0	2897.36	295.35	3188.94	325.07	0.372
32.0	3058.66	311.79	3372.19	343.75	0.361
34.0	3058.66	328.12	3555.34	362.42	0.350
36.0	3278.17	344.36	3738.59	381.10	0.340
38.0	3218.86	360.53	3921.74	399.77	0.332
40.0	3694.74	376.63	4104.90	418.44	0.324
42.0	3852.29	392.69	4288.05	437.11	0.317
44.0	4027.20	410.52	4471.20	455.78	0.310
46.0	4210.26	429.18	4654.26	474.44	0.304
48.0	4393.31	447.84	4837.41	493.11	0.298
50.0	4576.37	466.50	5020.56	511.78	0.293
55.0	5034.00	513.15	5479.30	558.44	0.281
60.0	5491.64	559.80	5936.03	605.10	0.271
65.0	5949.27	606.45	6393.77	651.76	0.263
70.0	6406.91	653.10	6851.50	698.42	0.255
75.0	6864.55	699.75	7309.23	745.08	0.249
80.0	7322.18	746.40	7766.97	791.74	0.243
85.0	7779.82	793.05	8224.61	838.39	0.238
90.0	8237.46	839.70	8682.34	885.05	0.233
95.0	8695.09	886.35	9139.98	931.70	0.229

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IRS Bridge Rules
Live Load 2.3

L (m)	Total load for Bending Moment		Total load for Shear Force		Impact Factor CDA=
	kN	t	kN	t	0.15+8/(6+L)
100.0	9152.73	933.00	9597.61	978.35	0.225
105.0	9610.37	979.65	10055.35	1025.01	0.222
110.0	10068.00	1026.30	10512.98	1071.66	0.219
115.0	10525.64	1072.95	10970.62	1118.31	0.216
120.0	10983.28	1119.60	11428.36	1164.97	0.213
125.0	11440.91	1166.25	11885.99	1211.62	0.211
130.0	11898.55	1212.90	12343.63	1258.27	0.209

EUDL for BM and Shear Force given in this Appendix are not applicable for ballasted deck for spans upto and including 8.0m for which Appendix XXIII(a) is to be referred.

IRS Bridge Rules, Live Load 2.3

APPENDIX XXIII (a)

"25t LOADING-2008"

BROAD GAUGE 1676mm

1. **Equivalent Uniformly Distributed Load (EUDL) for Bending Moment in Kilo-Newton (tonnes) for cushions of various depths and spans upto and including 8m.**

For Bending Moment L is equal to the effective span in metres.

NOTE:

(1) *For intermediate values of L and cushions, the EUDL shall be arrived at by linear interpolation.*

(2) *The figures given below do not include dynamic effects.*

L	EUDL for Bending Moment							
	Cushion (mm)							
	200		300		400		600	
Metres	KN	t	kN	t	KN	t	KN	t
0.5	267.5	27.3	221.1	22.6	186.2	19.2	143.5	14.6
1.0	378.8	38.7	354.3	36.2	329.8	33.7	280.8	28.7
1.5	415.8	42.4	389.5	40.8	383.2	39.1	350.5	35.8
2.0	434.4	44.3	422.1	43.1	409.9	41.8	385.4	39.3
2.5	445.5	45.5	435.7	44.5	425.9	43.5	406.3	41.5
3.0	452.9	46.2	444.8	45.4	436.6	44.6	420.2	42.9
3.5	498.6	50.9	491.7	50.2	484.6	49.5	471.1	48.1
4.0	570.9	58.2	564.8	57.6	559.2	57.1	549.0	56.0
4.5	654.0	66.7	649.1	66.2	644.2	65.7	634.4	64.7
5.0	722.6	73.7	717.7	73.2	712.8	72.7	703.0	71.7
5.5	790.5	80.7	786.1	80.2	781.6	79.8	772.7	78.9
6.0	847.1	86.4	843.0	86.0	839.0	85.6	830.8	84.8
7.0	936.1	95.5	932.7	95.2	929.1	94.8	922.1	94.1
8.0	1002.8	102.3	999.7	102.0	996.8	101.7	990.6	101.1

APPENDIX XXIII (a) (Contd....)

2. **Equivalent Uniformly Distributed Load (EUDL) for Shear Force in Kilo-Newton (tones) for cushions of various depths and spans upto and including 8m.**

For Shear Force, L is the loaded length in metres to give the maximum Shear Force in the member.

NOTE:

(1) *For intermediate values of L and cushions, the EUDL shall be arrived at by linear interpolation.*

(2) *The figures given below do not include dynamic effects.*

L	EUDL for Shear Force							
	Cushion (mm)							
	200		300		400		600	
Metres	KN	t	KN	t	KN	t	KN	t
0.5	276.1	27.5	221.3	22.6	187.5	19.1	143.6	14.6
1	379.3	38.7	354.6	36.2	330.1	33.7	281.1	28.7
1.5	416.3	42.4	400.0	40.8	383.6	39.1	350.9	35.8
2	434.9	44.3	422.6	43.1	410.4	41.8	385.7	39.3
2.5	528.0	53.3	509.3	51.9	488.7	49.9	455.3	46.4
3	604.3	61.6	588.0	59.9	571.5	58.3	538.0	54.9
3.5	658.1	67.1	644.1	65.7	630.0	64.3	601.9	61.4
4	710.4	72.4	694.1	71.0	682.4	69.6	658.9	66.9
4.5	762.3	80.8	776.0	79.1	759.7	77.4	727.1	74.1
5	860.2	87.7	845.5	86.2	831.2	84.7	801.4	81.7
5.5	915.8	93.4	902.4	92.0	889.0	90.6	862.3	87.9
6	962.2	98.1	949.8	96.8	937.8	95.6	912.9	93.1
7	1034.8	105.5	1024.4	104.4	013.9	103.4	992.9	101.2
8	1089.5	111.1	1080.3	110.1	071.1	109.2	1052.6	107.3

IRS Bridge Rules
Live Load 2.3

APPENDIX-XXV

DERAILMENT LOADS FOR BALLASTED DECK BRIDGES
(25t Loading-2008)

S.N.	Condition and approach	Bridges with guard rails	Bridges without guard rails
1.	**Ultimate –** The bad at which a derailed vehicle shall not cause collapse of any major element.	a) Two vertical line loads of 75 kN/m each 1.6m* apart parallel to the track in the most unfavorable position inside an area of 1.3m on either side of track centre line. b) A single load of 200 kN acting on an area of 1.3m on either side of track centre line in the most unfavorable position.	a) Two vertical line loads of 75 kN/m each 1.6m* apart parallel to the track in the most unfavorable position inside an area of 2.25m on either side of track centre line. b) A single line load of 200 kN acting on an area of 2.25m on either side of track centre line in the most unfavorable position.
2.	**Stability –** The structure shall not overturn.	A vertical line load of 94 kN/m with a total length of 20m acting on the edge of the structure under consideration.	A vertical line load of 94 kN/m with a total length of 20m acting on the edge of the structure under consideration

** The distance 1.6m is based on Broad Gauge distance 1.676m as adopted for derailment loads for MBG-1987 loading and HM loading.*

IRS Bridge Rules
2.3.4.2 Dispersion of railway live loads

(a) Distribution through sleepers and ballast:
The sleeper may be assumed to distribute the live load uniformly on top of the ballast over the area of contact given below:

Type I	Type II
	Under each rail seat
BG 2745mm x 254mm	760mm x 330mm
MG 1830mm x 203mm	610mm x 270mm

- The load under the sleeper shall be assumed to be dispersed by the fill including ballast at a slope not greater than half horizontal to one vertical and all deck slabs shall be designed for both types of sleepers.

IRS Bridge Rules
2.3.4.2 Dispersion of railway live loads

(b) Distribution through R.C. Slab: When there is effective lateral transmission of Shear Force, the load may be further distributed in a direction at right angles to the span of the slab equal to the following:

(i) ¼ span on each side of the loaded area in the case of simply supported, fixed and continuous spans.

(ii) ¼ of loaded length on each side of the loaded area in the case of cantilever slabs.

- No distribution through the slab may be assumed in the direction of the span of the slab.

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IRS Bridge Rules
2.4 DYNAMIC EFFECT,
2.4.1 Railway Bridges (Steel)

- The CDA should be obtained as follows and shall be applicable up to 160 km/h on BG and 100 km/h on MG Line

(a) For single track spans:

$$CDA = 0.15 + \frac{8}{(6+L)}$$

- CDA<=1.0, Where L is:
- (1) the loaded length of span in metres for the position of the train giving the maximum stress in the member under consideration.
- (2) 1.5 times the cross-girder spacing in the case of stringers (rail bearers) and
- (3) 2.5 times the cross girder spacing in the case of cross girders.

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IRS Bridge Rules
2.4 DYNAMIC EFFECT,
2.4.1 Railway Bridges (Steel)

CDA for Stringer, L=2.5m c/c of Cross Girder x1.5

Coefficient of Dynamic Augment, CDA	=	0.15	+	8 / (6 + L)					≤ 1.0
	=	0.15	+	8 / (6 + 2.5 × 1.5)					≤ 1.0
	=	0.9705							

CDA for Cross Girder, L=2.5m c/c of Cross Girder x2.5

Coefficient of Dynamic Augment, CDA	=	0.15	+	8 / (6 + L)					≤ 1.0
	=	0.15	+	8 / (6 + 2.5 × 2.5)					≤ 1.0
	=	0.8031							

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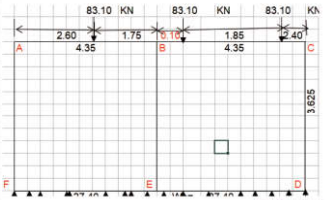
IRS Bridge Rules
2.4 DYNAMIC EFFECT,
2.4.1 Railway Bridges

- Wheel Load Dispersion on Box Culvert

Case - 1 : Live load and Impact				
"25 T Loading - 2008" as per RDSO				
Effective L=	4.35	m		
where, I	=	0.15+8/(L+6) ≤ 1.0		Impact factor
	=	0.923	≯ 1.00	
	=	0.923		
When the depth of fill <		0.9	m	
Depth of Fill=		0.30	m	
The co-efficient of Dynamic Augment shall be				
[2-(d/0.9)]*1/2*CDA		=	0.77	
Effect of Dispersion				
Sleeper width	=	2.745m	m	
Depth of Fill	=	0.30	m	
Span	=	4.35	m	
Effect of Dispersion= Sleeper width+(Depth of fill/2+span/4)x2				
E	=	5.22	m	

For the axel load	=	245.2	Kn/m²
Live Load including Dynamic Augment, P	=	433.8	Kn/m²
Load on top slab= P/E	=	83.1	KN/m
Uplift Pressure on bottom slab	=	27.396	Kn/m²
Assume 1.0 m width			



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IRS Bridge Rules
2.4 DYNAMIC EFFECT,
2.4.1 Railway Bridges (Steel)

(b) For main girders of double track spans with 2 girders, CDA as calculated above may be multiplied by a factor of 0.72 and shall be subject to a maximum of 0.72. (c) For intermediate main girders of multiple track spans, the CDA as calculated in Clause 2.4.1.1(a) may be multiplied by a factor of 0.6 and shall be subject to a maximum of 0.6. (d) For the outside main girders of multiple track spans with intermediate girders, CDA shall be that specified in Clause 2.4.1.1(a) or (b) whichever applies. (e) For cross girders carrying two or more tracks, CDA as calculated in Clause 2.4.1.1(a) may be multiplied by a factor of 0.72 and shall be subject to a maximum of 0.72

IRS Bridge Rules
2.4 DYNAMIC EFFECT,
2.4.1 Railway Bridges (Steel)

(f) Where rails, with ordinary fish-plated joints, are supported directly on transverse steel troughing or steel sleepers (H Beam) the dynamic augment for calculating stresses in such troughing or sleepers shall be taken as:

$\frac{7.32}{B+5.49}$ for BG $\qquad$ $\frac{5.49}{B+4.27}$ for MG

Where B = the spacing of main girders in metres.
The same Coefficient of dynamic augment (CDA) may be used for calculating the stresses in main girders up to 7.5m effective span, stringers with spans up to 7.5m and also chords of triangulated girders supporting the steel troughing or steel sleepers.

IRS Bridge Rules
2.4 DYNAMIC EFFECT,
2.4.2 Railway pipe culverts, arch bridges, concrete slabs and concrete girders

2.4.2.1 For all gauges
(a) If the depth of fill is less than 900mm, the Coefficient of Dynamic Augment shall be equal to- $[2-(d/0.9)] \times \frac{1}{2} \times CDA$

as obtained from Clause 2.4.1.1(a)
Where, **d** = depth of fill in 'm'.

(b) If the depth of fill is 900mm, the Coefficient of Dynamic Augment shall be half of that specified in clause 2.4.1.1(a) subject to a maximum of 0.5. Where depth of fill exceed 900mm, the Coefficient of Dynamic Augment shall be uniformly decreased to zero within the next 3 metres.
(c) In case of concrete girders of span of 25m and larger, the CDA shall be as specified in Clause 2.4.1.1. (a)

IRS Bridge Rules
2.5 FORCES DUE TO CURVATURE AND ECCENTRICITY OF TRACK

- 2.5.1 For ballasted deck bridges, even on straight alignment, an eccentricity of centre line of track from design alignment up to 100mm shall be considered for the purpose of designs.

Refer appendix XXIII of IRS bridge rules
For 35.00 m span,
EUDL = 3298.51 kN / 35.00 m × 2 m = 47.12 kN/m
CDA = 0.350
Total EUDL = (1 + CDA) × Total EUDL
= (1 + 0.350)× 47.1216 = 63.61 kN/m
On one Main Girder, EUDL = 1 × Total EUDL
= 1 × 63.6141 = 63.614 kN/m
onsidering effect of eccentricity of track as per Clause 2.5.1 of IRS Bridge Rules:
dditional EUDL on main girder = 63.6141 × 0.1 / 6.2 * (dist between centre lines of main girder)
= 1.026 kN/m
So, Total EUDL on main girder = 63.6141 + 1.02603 = 64.640 kN/m

C/C of Girder 6.2m

IRS Bridge Rules
2.5 FORCES DUE TO CURVATURE AND ECCENTRICITY OF TRACK

- 2.5.3 For railway bridges the following loads must be considered: (a) The extra loads on one girder due to the additional reaction on one rail and to the lateral displacement of the track calculated under the following two conditions: (i) Live load running at the maximum speed. (ii) Live load standing with half normal dynamic augment
- (b) The horizontal load due to centrifugal force which may be assumed to act at a height of 1830mm for "25t Loading2008" for BG, 3000mm for "DFC loading (32.5t axle load)" for BG and 1450mm for MG above rail level is:

$$C = \frac{WV^2}{12.95R} \text{ OR } \left(\frac{WV^2}{127R} \text{ in MKS Units}\right)$$

Where, C= Horizontal effect in kN/m run (t/m run) of span.
W= Equivalent Distributed live load in t/m run.
V= Maximum speed in km per hour, and R= Radius of the curve in m.

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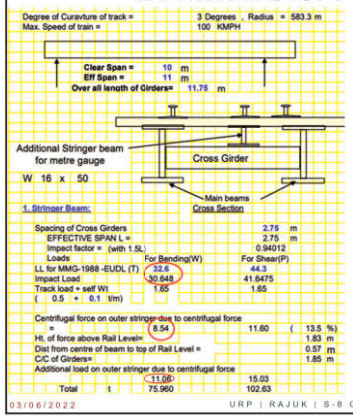
2.5 FORCES DUE TO CURVATURE AND ECCENTRICITY OF TRACK



Degree of Curvature 3^0
R=1750/D=1750/3=583.3m
Let Speed, V=100 km/hr
W=LL+I=32.6+30.648t
=63.248t
C=WV^2/127R
= 63.248x100^2/(127x583.3)
=8.54t/m
Distance from center of beam to C force=
(1.83+0.57)m=2.4m
C/C of Girder= 1.85m
Add. Vertical Load
=8.54x2.4/1.85= 11.06t/m

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IRS Bridge Rules
2.6 TEMPERATURE EFFECT

- 2.6.1 Where any portion of the structure is not free to expand or contract under variation of temperature, allowance shall be made for the stresses resulting from this condition.
- 2.6.2 The coefficient of expansion shall be taken as below: for steel and reinforced concrete 11.7 x 10^{-6} per 1^0 C for plain concrete 10.8 x 10^{-6} per 1^0 C
- Let Length of Girder L=35m

Average Temperature variation t=30^0 C
Coefficient of Expansion of steel =11.7x10^{-6} per 1^0 C
Expansion Length= 35x1000x11.7x10^{-6}x30=12.285mm

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IRS Bridge Rules
2.7 FRICTIONAL RESISTANCE OF EXPANSION BEARINGS

Roller bearing	0.03
Sliding bearings of steel on cast iron or steel bearing	0.25
Sliding bearing of steel on ferro bestos	0.20
Sliding bearings of steel on hard copper alloy bearings	0.15
For sliding bearings of PTFE/Elastomeric type	0.10
Concrete over concrete with bitumen layer in between	0.50
Concrete over concrete not intentionally roughened	0.60

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IRS Bridge Rules

2.8 LONGITUDINAL FORCES

- Tractive effort of the driving wheels of locomotives;
- Braking force resulting from the application of the brakes to all braked wheels;
- Resistance to the movement of the bearings due to change of temperature and deformation of the bridge girder. Roller, PTFE or elastomeric bearings may preferably be provided to minimize the longitudinal force arising on this account.
- Forces due to continuation of LWR/CWR over the bridges.

IRS Bridge Rules, Live Load 2.8 Tractive/ Braking

APPENDIX-XXIV

"25t Loading-2008"
BROAD GAUGE-1676 mm
Longitudinal Loads (Without Deduction for Dispersion)

NOTE: Where loaded length lies between the values given in the Table, the tractive effort or braking force can, with safety, be assumed as that for the longer loaded length.

L (Loaded length in metres)	Tractive effort		Braking force		Maximum LF	
	kN	t	kN	t	kN	t
1	2	3	4	5	6	7
1.0	103.01	10.50	61.80	6.30	103.01	10.50
1.5	103.01	10.50	61.80	6.30	103.01	10.50
2.0	206.01	21.00	122.63	12.50	206.01	21.00
2.5	206.01	21.00	122.63	12.50	206.01	21.00
3.0	206.01	21.00	122.63	12.50	206.01	21.00
3.5	245.25	25.00	165.79	16.90	245.25	25.00
4.0	309.02	31.50	184.43	18.80	309.02	31.50
4.5	309.02	31.50	184.43	18.80	309.02	31.50
5.0	309.02	31.50	184.43	18.80	309.02	31.50
5.5	309.02	31.50	184.43	18.80	309.02	31.50
6.0	309.02	31.50	184.43	18.80	309.02	31.50
6.5	326.87	33.32	220.73	22.50	326.87	33.32
7.0	326.87	33.32	220.73	22.50	326.87	33.32
7.5	326.87	33.32	220.73	22.50	326.87	33.32
8.0	408.59	41.65	275.66	28.10	408.59	41.65
8.5	408.59	41.65	275.66	28.10	408.59	41.65
9.0	408.59	41.65	275.66	28.10	408.59	41.65
9.5	412.02	42.00	275.66	28.10	412.02	42.00
10.0	490.30	49.98	331.58	33.80	490.30	49.98
11.0	490.30	49.98	331.58	33.80	490.30	49.98
12.0	515.03	52.50	331.58	33.80	515.03	52.50
13.0	618.03	63.00	331.58	33.80	618.03	63.00
14.0	618.03	63.00	367.88	37.50	618.03	63.00
15.0	618.03	63.00	367.88	37.50	618.03	63.00
16.0	618.03	63.00	386.51	39.40	618.03	63.00
17.0	618.03	63.00	386.51	39.40	618.03	63.00
18.0	653.74	66.64	441.45	45.00	653.74	66.64
19.0	653.74	66.64	441.45	45.00	653.74	66.64
20.0	735.46	74.97	496.39	50.60	735.46	74.97
21.0	735.46	74.97	496.33	50.60	735.46	74.97
22.0	735.46	74.97	511.10	52.10	735.46	74.97
23.0	735.46	74.97	523.12	53.38	735.46	74.97
24.0	735.46	74.97	551.32	56.20	735.46	74.97
25.0	824.04	84.00	551.32	56.20	824.04	84.00
26.0	824.04	84.0	560.15	57.10	824.04	84.0

L (Loaded length in metres)	Tractive effort		Braking force		Maximum LF	
	kN	t	kN	t	kN	t
27.0	927.05	94.50	572.90	58.40	927.05	94.50
28.0	927.05	94.50	607.05	61.88	927.05	94.50
29.0	980.61	99.96	662.18	67.50	980.61	99.96
30.0	980.61	99.96	662.18	67.50	980.61	99.96
32.0	980.61	99.96	680.61	69.40	980.61	99.96
34.0	980.61	99.96	735.75	75.00	980.61	99.96
36.0	1030.05	105.00	735.75	75.00	1030.05	105.00
38.0	1133.06	115.50	760.28	77.50	1133.06	115.50
40.0	1236.06	126.00	784.80	80.00	1236.06	126.00
42.0	1236.06	126.00	809.33	82.50	1236.06	126.00
44.0	1236.06	126.00	833.85	85.00	1236.06	126.00
46.0	1236.06	126.00	858.38	87.50	1236.06	126.00
48.0	1236.06	126.00	882.90	90.00	1236.06	126.00
50.0	1236.06	126.00	907.43	92.50	1236.06	126.00
55.0	1236.06	126.00	968.25	98.70	1236.06	126.00
60.0	1236.06	126.00	1030.05	105.00	1236.06	126.00
65.0	1236.06	126.00	1090.87	111.20	1236.06	126.00
70.0	1236.06	126.00	1152.68	117.50	1236.06	126.00
75.0	1236.06	126.00	1213.50	123.70	1236.06	126.00
80.0	1236.06	126.00	1275.30	130.00	1275.30	130.00
85.0	1296.06	136.00	1336.12	136.20	1336.12	136.20
90.0	1236.06	126.00	1397.93	142.50	1397.93	142.50
95.0	1236.06	126.00	1458.75	148.70	1458.75	148.70
100.0	1236.06	126.00	1520.55	155.00	1520.55	155.00
105.0	1236.06	126.00	1581.37	161.20	1581.37	161.20
110.0	1236.06	126.00	1643.18	167.50	1643.18	167.50
115.0	1236.06	126.00	1704.00	173.70	1704.00	173.70
120.0	1236.06	126.00	1765.80	180.00	1765.80	180.00
125.0	1236.06	126.00	1826.62	186.20	1826.62	186.20
130.0	1236.06	126.00	1888.43	192.50	1888.43	192.50

- ***In between given Length Longer loaded length***

IRS Bridge Rules

2.8 LONGITUDINAL FORCES LWR/CWR

- A length of greater than 250 meter on Broad Gauge and 500 m on Meter Gauge will normally 4 function as LWR (Fig. 1.1). The maximum length of LWR under Indian conditions shall normally be restricted to one block section.
- 2) Continuous Welded Rail (CWR) is a LWR which would continue through station yards including points and crossings. (internet Source)
- IRS Refers to UIC 774-3 R-2001

UIC Code 774-3 R

Track/bridge Interaction
Recommendations for calculations

Union Internationale des Chemins de fer
Internationale Eisenbahnverband
International Union of Railways
UIC

IRS Bridge Rules

2.8 LONGITUDINAL FORCES LWR/CWR

- Ballasted deck bridges without bearings (slabs, box culverts and arches) need not be checked for forces/effects due to continuation of LWR/CWR.
- (g) If rail-free fastenings, SEJ (Switch Expansion Joint)are provided as per provisions of Manual Of Instructions On Long Welded Rails, such that there is no interaction between the rail and the bridge, then there is no need for checking for forces/effects due to continuation of LWR/CWR.
- It shall be ensured that the additional stresses in rail as per computations done using provisions of UIC 774-3R do not exceed the values given in table below:

IRS Bridge Rules

2.8 LONGITUDINAL FORCES LWR/CWR

- Span and sub structure arrangement shall be such that the various checks on rotation/ deflection specified in UIC 774-3R are satisfied.

Rail Section	Maximum additional Stresses in Compression	Maximum additional Stressess in Tension
60 Kg 90 UTS Rail	60 N/mm^2	75 N/mm^2
52 Kg 90 UTS Rail	50 N/mm^2	60 N/mm^2

- Track Resistance in Ballasted Deck Bridges : For track structure minimum 52 kg 90 UTS rails and PRC sleepers at sleeper density 1540 nos/KM with elastic fastenings, the value of track resistance for computations as per UIC 774-3R shall be taken as 25kN per meter of track in unloaded condition and 50 kN per meter of track in loaded condition.

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IRS Bridge Rules

2.8 LONGITUDINAL FORCES LWR/CWR

- 2.8.3.4 For the design of new bridges and in case of rebuilding of existing bridges, dispersion of longitudinal force shall not be allowed.
- Multiplication Factor of LF for More than 1 track

1	1.00
2	1.00
3	0.90*
4 or more	0.75*

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IRS Bridge Rules

2.8 LONGITUDINAL FORCES

2.8.5 When considering seismic forces, only 50% of gross tractive effort/braking force, to be reduced by taking dispersion and distribution of longitudinal forces, shall be considered along with horizontal seismic forces along/across the direction of the traffic.

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IRS Bridge Rules

2.9 RACKING FORCES

2.9.1 Lateral bracings of the loaded deck of railway spans shall be designed to resist, in addition to the wind and centrifugal loads specified above, a lateral load due to racking forces of 5.88 kN/m (600 kg/m) treated as moving load.

This lateral load need not be taken into account when calculating stresses in chords or flanges of main girders.

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IRS Bridge Rules

2.10 FORCES ON PARAPETS

- shall be designed to resist a lateral horizontal force and a vertical force of 1.47 kN/m(150 kg/m) applied simultaneously at the top of the railing or parapet.

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IRS Bridge Rules

2.11 WIND PRESSURE EFFECT

- 2.11.1.1 Wind pressures are expressed in terms of a basic wind pressure 'P' which is an equivalent static pressure in the windward direction. (From BNBC wind speed)
- 2.11.2 The wind pressure specified above shall apply to all loaded or unloaded bridges provided that a bridge shall not be considered to be carrying any live load when the wind pressure at deck level exceeds the following limits:

Broad Gauge bridges	1.47 kN/m^2 (150 kg/m^2)
Metre and Narrow Gauge Bridges	0.98 kN/m^2 (100kg/m^2)
Foot-bridges	0.74 kN/m^2 (75 kg/m^2)

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IRS Bridge Rules

2.11 WIND PRESSURE EFFECT

- (a) For unloaded spans and trestles net exposed area shall be considered as one and half times the horizontal projected area of the span or the trestle, except for plate girders for which the area of the leeward girder shall be multiplied by the factors shown below and added to the area of the windward girder: -

When the spacing of the leeward girder does not exceed half its depth	0.00
For spacing exceeding half depth and upto full depth	0.25
For spacing exceeding full depth and upto one and half times depth	0.50
For spacing exceeding one and a half times depth and upto twice its depth or more	1.00

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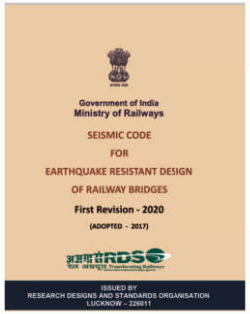
IRS SEISMIC CODE

2.12 FORCES AND EFFECTS DUE TO EARTHQUAKE

Refers to "Seismic Code for Earthquake resistant design of Railway Bridges"

3.7 Effects of Earthquake - The effects of earthquake motion that shall be considered in seismic design of bridge include inertial force, displacements, earth pressure, hydrodynamic pressure and liquefaction of soil.



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IRS Seismic Code
5 DESIGN PHILOSOPHY

5.1 Serviceability Limit State The design of bridge should meet the serviceability limit state under design basis earthquake (DBE). The parts of the bridge intended to contribute to energy dissipation shall undergo minor damage without giving rise to need for reduction of traffic or immediate repair.

5.2. Ultimate Limit State The design of bridge should meet non-collapse requirement that is, ultimate limit state under maximum considered earthquake (MCE). While designing as per IRS concrete bridge code, DBE may be considered. The bridge shall be retain its structural integrity and adequate residual resistance, although considerable damage may occur in some portions of the bridge. The structure should be able to sustain emergency traffic, inspections and repair could be performed easily after the earthquake. The bridge superstructure however shall in general be protected from the formation of plastic hinges and from unseating due to extreme seismic displacements under MCE.

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IRS Seismic Code
5 DESIGN PHILOSOPHY

5.3 Ductile Behavior The reinforced and pre-stressed concrete components shall be designed as under-reinforced so as to cause a tensile failure. Further, they should be suitably designed to ensure that premature failure due to shear or bond does not occur. Stresses induced in the superstructure due to earthquake ground motion are usually quite nominal. Therefore, ductility demand under seismic shaking has not been a major concern in the bridge superstructures during past earthquakes. However, the seismic response of bridges is critically dependent on the ductile characteristics of the sub-structures. Provisions for appropriate ductile detailing of reinforced concrete members given in Annex B shall be applicable to sub-structures. Bridges shall be designed such that under severe seismic shaking plastic hinges form in the sub-structure, rather than in the deck or foundation. Ductile detailing is mandatory for piers/portals of bridges located in seismic III, IV & V.

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IRS Seismic Code
2.12 Seismic Zoning Map India

9.4.6 Seismic Zone Map

For the purpose of determining design seismic forces, the country is classified into four seismic zones. A seismic zone map of India is shown in Fig. 1A. The peak ground acceleration (PGA) (or zero period acceleration, ZPA), associated with each zone, is called zone factor, Z. The zone factor is given in Table 1A. Zone factors for some important towns are given in Appendix E.

Table 1A - Zone Factor Z for Horizontal Motion

Seismic Zone	I	III	IV	V
Z	0.10	0.16	0.24	0.36

Note: Near Source Effect

For bridges which are within a distance of 11 km from a known active fault, seismic hazard shall be specified after detailed geological study of the fault and the site condition. In absence of such detailed investigation, the near-source modification in the form of 25% increase in zone factor may be used.

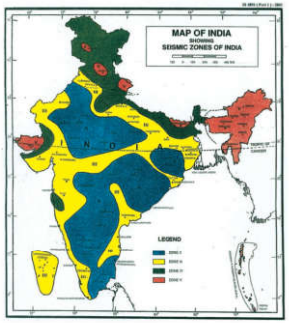

Fig. 1A Seismic zone map of India

BNBC-2020

I	II	III	IV
0.12	0.20	0.28,	0.36

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2.12 Seismic Zoning Map BNBC 2020

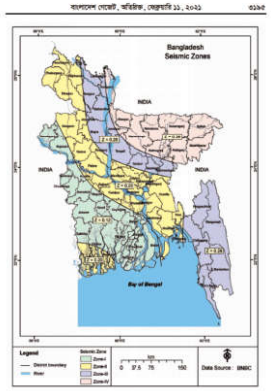

Figure 6.2.24 Seismic zoning map of Bangladesh

৫১৯৬ বাংলাদেশ গেজেট, অতিরিক্ত, ফেব্রুয়ারি ১১, ২০২১

Table 6.2.14: Description of Seismic Zones

Seismic Zone	Location	Seismic Intensity	Seismic Zone Coefficient, Z
1	Southwestern part including Barisal, Khulna, Jessore, Rajshahi	Low	0.12
2	Lower Central and Northwestern part including Noakhali, Dhaka, Pabna, Dinajpur, as well as Southwestern corner including Sundarbans	Moderate	0.20
3	Upper Central and Northwestern part including Brahmanbaria, Sirajganj, Rangpur	Severe	0.28
4	Northeastern part including Sylhet, Mymensingh, Kurigram	Very Severe	0.36

Both Coefficient nearly same

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IRS Seismic Code
7.4.2 CALCULATION OF NATURAL PERIOD OF BRIDGE

7.4.2 The seismic zone factor for vertical ground motions, when required may be taken as two-thirds of that for horizontal motions given in Table 2 of IS 1893 (Part 1).

However, the time period for the superstructure has to be worked out separately using the characteristic of the superstructure for vertical motion, in order to estimate S /g for vertical acceleration. The natural time period of superstructure can be a estimated using appropriate modelling and free vibration analysis using computer.

However, for simply supported superstructure with uniform flexural rigidity, the fundamental time period T_v , for vertical motion T_v can be estimated using the expression:

$$T_v = \frac{2}{\pi} l^2 \sqrt{\frac{m}{EI}}$$

where l = span; m = mass per unit length; and EI= flexural rigidity of the superstructure. For Elastic Analysis R=1.0

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IRS Seismic Code

- 8. CALCULATION OF NATURAL PERIOD OF BRIDGE
- 8.1 Simply Supported Bridges
- Where the vibration unit of sub-structure can be idealized as a single cantilever pier carrying the superstructure mass, resting on well, pile or open foundation, the fundamental period shall be calculated from the following equation:

$$T = 2\pi\sqrt{\frac{\delta}{g}}$$

Where δ = horizontal displacement at the top of pier due to horizontal force (= *mg*)

where m = lumped mass at the top of pier.

δ=Fh^2/(3EI) for single pier, F is the total Lumped Weight on pier

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IRS Seismic Code

- 8. CALCULATION OF NATURAL PERIOD OF BRIDGE

IRC:6-2014

Annex D
*(Clause **219.5**)*

The fundamental natural period T (in seconds) of pier/abutment of the bridge along a horizontal direction may be estimated by the following expression:

$$T = 2.0\sqrt{\frac{D}{1000F}}$$

where,

D = Appropriate dead load of the superstructure and live load in kN

F = Horizontal force in kN required to be applied at the centre of mass of superstructure for one mm horizontal deflection at the top of the pier/ abutment for the earthquake in the transverse direction; and the force to be applied at the top of the bearings for the earthquake in the longitudinal direction.

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IRS Seismic Code
2.12 LONGITUDINAL FORCES

- In general pier shall be considered fixed at the foundation level. However, in case of soft soil or deep foundations, soil flexibility may be considered in the calculation of natural period as per 6.4 of IS 1893 (Part 1)
- Moment of Inertia may be considered for calculation of time period. In case of RCC bridge piers,75% of gross moment of Inertia may be considered as cracked moment of inertia, in absence of detailed calculation.

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IRS Seismic Code

2.12 LONGITUDINAL FORCES

- **9 METHOD OF CALCULATING SEISMIC FORCES**
- 9.1 The following methods of seismic analysis may be employed for calculation of seismic forces in bridges:
- a) Seismic coefficient method (SCM);
- b) Response spectrum method (RSM);
- c) Time history method (THM); and
- d) Nonlinear pushover analysis (NPA).
- The recommended method of analysis for different category of bridges and earthquake level is given in Table 1.
- The linear analysis considering elastic behaviour is required for DBE (Design Basis Earthquake).

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IRS Seismic Code

2.12 LONGITUDINAL FORCES

Table 1 Method of Seismic Analysis of Bridges
(Clause 9.1)

Earthquake level	Category of Bridge Type		
	Regular	Special Regular	Special Irregular
(1)	(2)	(3)	(4)
DBE	SCM	RSM	RSM THM NPA

NOTE : In case of MCE, non-linear analysis and Tie History Method shall be adopted for regular, special regular and special irregular bridges.

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IRS Seismic Code

2.12 LONGITUDINAL FORCES

3.12 Regular Bridge - A regular bridge has no abrupt or unusual changes in mass, stiffness or geometry along its span and has no large differences in these parameters between adjacent supports (abutments excluded). A bridge shall be considered regular for the purpose of this standard, if

(a) it is straight or it describes a sector of an arch which subtends an angle less than 90° at the center of the arch;

(b) the adjacent piers do not differ in stiffness by more than 25 percent (Percentage difference shall be calculated based on the lesser of the two stiffness); and

(c) girder bridges, T-beam bridges, truss bridges, hammer head bridges, bridges having single or multiple simply supported spans with each span less than 120 m and pier height above foundation level less than 30 m.

3.9 Special Regular Bridge - The bridges specified under regular bridges but single span more than 120 m or pier height measured from founding level to the top of pier cap to be more than 30 m. In case of pile foundation pier height shall be considered from the point of fixity.

3.16 Special and Irregular Types of Bridges - The bridges with innovative designs and bridges such as suspension bridge, cable stayed bridge, arch bridge, bascule bridge and irregular bridges such as skew bridge of angle 30° and above with span more than 60 m shall be categorized under these types.

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IRS Seismic Code

2.12 LONGITUDINAL FORCES

9.2 Seismic Co-efficient Method

The seismic force to be resisted by bridge components shall be computed as follows:

$$F = A_h W$$

where

F = horizontal seismic force to be resisted;

W = weight of mass under consideration ignoring reduction due to buoyancy or uplift; and

A_h = design horizontal seismic coefficient as determined from 9.4.1.

9.3 For embedded portion of foundation at depths exceeding 30 m below scour level, the seismic force due to foundation mass may be computed using design seismic coefficient equal to $0.5A_h$.

For portion of foundation between the scour level and up to 30 m depth, the seismic force due to that portion of foundation mass may be computed using seismic coefficient obtained by linearly interpolating between A_h at scour level and $0.5A_h$ at a depth 30 m below scour level.

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IRS Seismic Code

2.12 LONGITUDINAL FORCES

9.4.1 Horizontal Seismic Coefficient, A_h

The design horizontal seismic coefficient, A_h shall be determined from following expression of 6.4.2 of IS1893 (Part1)

$$A_h = \frac{Z}{2} \cdot \frac{I}{R} \cdot \frac{S_a}{g}$$

Provided that for any structure with T <0.1 s, the value of A_h shall not be taken less than $Z/2$, whatever be the value of I/R.

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IRS Seismic Code

2.12 LONGITUDINAL FORCES

where

Z = *zone factor;*

I = importance factor (*see* Table 2);

R = response reduction factor (*see* Table 3); and

S_a/g = average acceleration coefficient for rock or soil sites as given in Fig. 1.

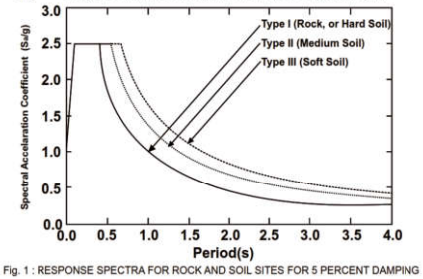

Fig. 1 : RESPONSE SPECTRA FOR ROCK AND SOIL SITES FOR 5 PERCENT DAMPING

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2.12 LONGITUDINAL FORCES

Table 2 Importance Factor
(Clause 9.4.4)

S.No. (1)	Seismic Class (2)	Illustrative Examples of Bridges (3)	Importance Factor 'I' (4)
i)	Normal bridges	All bridges except those mentioned in other class	1.0
ii)	Important bridges	a) River bridges and flyovers inside cities	1.2
		b) Bridges on national and state highways	1.2
		c) Bridges serving traffic near ports and other centres of economic activities.	1.2
		d) Bridges crossing railway lines.	1.2
iii)	Large critical bridges in all Seismic Zones	a) Long bridges more than 1 m length across perennial rivers and creeks	1.5
		b) Bridges for which alternative routes are not available	1.5
iv)	Railway bridges	a) All important bridges irrespective of route.	1.5
		b) Major bridges on group A, B and C routes (Route classification as per IRP way manual)	1.5
		c) Major bridges on all other routes.	1.25
		d) All other bridges on group A,B, and C routes	1.25
		e) All other bridges	1.0

NOTE : While checking for seismic effect during construction, the importance factor of I shall be considered for all bridges in all zones.

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IRS Seismic Code

2.12 LONGITUDINAL FORCES

Table 3 Response Reduction Factor R for Bridge Components
(Clause 9.4.5)

Sl. No. (1)	Structure, Component or Connection (2)	R (3)
i)	Superstructure	2
ii)	Substructure:	
	a) Reinforced concrete piers with ductile detailing cantilever type, wall type	3.0
	b) Reinforced concrete piers without ductile detailing*, cantilever type, wall type	2.5
	c) Masonry piers (un reinforced) cantilever type, wall type	1.5
	d) Reinforced concrete, framed construction in piers, with ductile detailing, columns of RCC bents, RCC single column piers	4.0
	e) Steel framed construction	2.5
	f) Steel cantilever piers	1.0
	g) Steel trussed arch	1.5
	h) Reinforced concrete arch	3.5
	i) Abutments of mass concrete and masonry	1.0
	j) R.C.C. abutment	2.5
	k) Integral frame with ductile detailing, and	4.0
	l) Integral frame without ductile detailing	3.3
iii)	Bearings (Elastomeric, pot, knuckle, roller-rocker)	2.0
iv)	Expansion joints and connections within a span of structure, hinge	1.0
v)	Stoppers in bearings	1.0
vi)	Foundations (well, piles or open).	2.0

NOTE - Response reduction factor, R should be taken as 1.0 for calculating displacements.

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2.12 LONGITUDINAL FORCES

7.2 Combination of Seismic Design Forces with Other Forces

The design seismic force resultant at a cross-section of a bridge component shall be appropriately combined with those due to other forces as per Table 12 of IRS Concrete Bridge Code (reprint 2014). However, in lieu of combination 2 of Clause 11.0 of IRS Concrete Bridge Code, following load combinations shall be used:

(A) Ultimate limit state design

1) $1.25DL + 1.5DL(S) + 1.5EQ + 1.4PS + 1.7EP$

2) $1.25DL + 1.5DL(S) + 0.5(LL + LL(F)) + 1.2EQ + 1.7EP + 1.4PS + 1.4HY + 1.4BO$

3) $0.9DL + 0.8DL(S) + 1.5EQ + 1.4PS + 1.7EP$

(B) Serviceability Limit State

1) $1.0DL + 1.2DL(S) + 1.0EQ + 1.0EP + 1.0PS + 1.0HY + 1.0BO$

2) $1.0DL + 1.2DL(S) + 0.5(LL + LL(F)) + 1.0EQ + 1.0EP + 1.0PS + 1.0HY + 1.0BY$

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IRS Seismic Code

2.12 LONGITUDINAL FORCES

C) During the construction stage, following load combination shall be used:

$1.0DL + 1.2DL(S) + 0.8EQ + 1.0ER + 1.3EP + 1.0PS + 1.0HY + 1.0BO$

The symbols used in above equation are explained as below

DL = dead load,

DL(S) = superimposed dead load,

LL = Full live load (without any reduction)

The live load (LL) includes impact effect, longitudinal forces (tractive and braking), and centrifugal force.

LL (F) = live load on footpath,

EQ = earthquake load,

EP = earth pressure,

ER = erection load such as cranes, machines etc

PS = prestressing load,

HY = hydrodynamic load,

BO = buoyancy load,

SH = shrinkage load,

CR = creep load,

TE = temperature load.

a) $\pm EL_x \pm 0.3EL_y$

b) $\pm 0.3EL_x \pm EL_y$

1) $\pm EL_x \pm 0.3EL_y \pm 0.3EL_z$

2) $\pm 0.3EL_x \pm EL_y \pm 0.3EL_z$

3) $\pm 0.3EL_x \pm 0.3EL_y \pm EL_z$

$\sqrt{EL_x^2 + EL_y^2}$ or $\sqrt{EL_x^2 + EL_y^2 + EL_z^2}$

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IRS Seismic Code

10 HYDRODYNAMIC FORCE ON SUB-STRUCTURE

10.1 The hydrodynamic force on submerged portion of pier and foundation up to mean scour level shall be assumed to act in a horizontal direction corresponding to that of earthquake motion. The total horizontal force is given by the following formula:

$F = C_e A_h W_e$

where

C_e = Coefficient(see table 4);

A_h = design horizontal seismic coefficient;

W_e = weight of the water in the enveloping cylinder,

$= \rho_w \pi a^2 H$, See **10.3**

ρ_w = unit weight of water;

H = height of submerged portion of pier; and

a = radius of enveloping cylinder.

Table 4 Values of C_e (Clause 10.1)

S.No. (1)	H/a (2)	Ce (3)
i)	1.0	0.390
ii)	2.0	0.575
iii)	3.0	0.675
iv)	4.0	0.730

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IRS Seismic Code

Hydrodynamic Forces

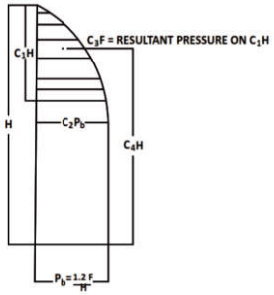

FIG. 2 DIAGRAM SHOWING HYDRODYNAMIC PRESSURE

Table 5 Coefficients C_1, C_2, C_3, and C_4 (Clause 10.1)

Sl.No. (1)	C_1 (2)	C_2 (3)	C_3 (4)	C_4 (5)
i)	0.1	0.410	0.026	0.934
ii)	0.2	0.673	0.093	0.871
iii)	0.3	0.832	0.184	0.810
iv)	0.4	0.922	0.289	0.751
v)	0.5	0.970	0.403	0.694
vi)	0.6	0.990	0.521	0.639
vii)	0.8	0.999	0.760	0.532
viii)	1.0	1.000	1.000	0.428

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IRS Seismic Code

2.12 LONGITUDINAL FORCES

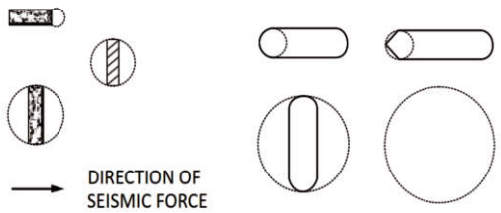

FIG. 3 CASES OF ENVELOPING CYLINDER

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IRS Seismic Code

2.12 LONGITUDINAL FORCES

11 SUPERSTRUCTURE

11.1 The superstructure shall be designed for the design seismic forces as specified in para 9 plus other loads required in design load combinations.

11.2 Under simultaneous action of horizontal and vertical accelerations, the superstructure shall have a factor of safety of at least 1.5 against overturning under DBE condition.

11.3 The superstructure shall be secured, when necessary to the sub-structure in all zones through bearings possessing adequate vertical holding down devices and/or unseating prevention system for superstructure. These devices should be used for suspended spans also with the restrained portion of the superstructure. However, frictional forces in the devices should not be relied upon for preventing dislodging and jumping of superstructure.

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IRS Seismic Code

2.12 LONGITUDINAL FORCES

14 SEATING WIDTH

The bearing seat width S_E in mm, between the end of girder and edge of sub-structure, Fig. 4 and minimum S_E between the ends of girder at suspended joint should be not less that the following values:

S_E = 203 + 1.67 *L* + 6.66 *H* for seismic Zones II and III

S_E = 305+2.50 *L* + 10.0 *H* for seismic Zones IV and V

where

L = length of the superstructure to the adjacent expansion joints or to the end of superstructure. In case of bearings under suspended spans, it is the sum of the lengths of two adjacent portions of the superstructure. In case of single span bridges, it is equal to the length of the superstructure, in m.

H = average height of all columns or piers supporting the superstructure to the next expansion joint, for bearings at abutments, in m. It is equal to zero for single span bridges. For bearings at column or piers, it is the height of column or pier. For bearings under suspended spans, it is the average height of two adjacent columns or piers.

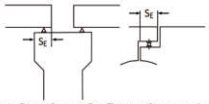

FIG. 4 BRIDGE SEATS ON PIER TOP OR AT SUSPENDED JOINT

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IRS Seismic Code

22.1.1 Dynamic Active Earth Pressure

$$C_a = \frac{(I \pm A_v)\cos^2(\phi - \lambda - \alpha)}{\cos\lambda \cos^2\alpha \cos(\delta + \alpha + \lambda)} \times \left[\frac{1}{1 + \sqrt{\frac{\sin(\phi + \delta)\sin(\phi - I - \lambda)}{\cos(\alpha - I)\cos(\delta + \alpha + \lambda)}}} \right]^2$$

22.1.1 Dynamic Active Earth Pressure Due to Backfill

Figure 5 shows a wall of height *h*, inclined with an angle α with vertical, retaining dry/moist cohesionless earth fill. The dynamic active earth pressure exerted against the wall shall be:

$$\left(P_{AH}\right)_{dyn} = \frac{1}{2}\gamma h^2 C_d$$

where

$(P_{AH})_{dyn}$ = dynamic total active earth pressure, in kN/m length of wall;

γ = unit weight of soil, in kN/m^3; and

h = height of wall, in m.

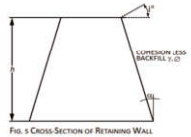

FIG. 5 CROSS-SECTION OF RETAINING WALL

rtical seismic coefficient - its direction being taken consistently throughout the stability analysis of wall and equal to $\frac{2}{3} A_h$;

ϕ = angle of internal friction of soil;

λ = $\tan^{-1}\left[\frac{A_h}{I \pm A_v}\right]$;

α = angle with earth face of the wall makes with the vertical;

I = slope of earth fill;

δ = angle of friction between the wall and earth fill; and

Ah = horizontal seismic coefficient.

The expression of $(C_a)_{dyn}$ gives two values depending on the sign of *Av*. For design purpose higher of the two values shall be taken.

Like Mononobe Okabe Equitation

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IRS Seismic Code

2.12 Dynamic Active Earth Pressure

Height of Soil Face, h	=	8.00m
Unit weight of Soil, γ	=	17.80Kn/m^3
Angle of Internal Friction of Soil	=	30.00deg.
δ = Angle of Friction between Soil and Abutment (backfill subjected to vibration)	=	10.00deg.
i = Backfill Slope Angle	=	0.00deg.
α = Wall Sloping Angle	=	0.00deg.
Seismic Zone Co-efficient, Z, α_a	=	0.120
Vertical Acceleration Coefficient, α_v , α_a /2	=	0.060
ϕ = Angle of Friction of Soil	=	30.00deg.
Horizontal Ground Acceleration, $A_{max} = Z * g$	=	1.18
λ = Arc Tan (α_a / (1 - α_v)) * 57.30	=	7.28deg.
$\sin(\phi + \delta)^* \sin(\phi - \lambda - i)$	=	0.2483
$\cos(\delta + \alpha + \lambda) \cos(i - \alpha)$	=	0.9549
$\cos^2(\phi - \lambda - \alpha)$	=	0.8508
$\cos\lambda \cos^2\alpha \cos(\delta + \alpha + \lambda)$	=	0.9472
Ψ = (1 + ($\sin(\phi + \delta)^* \sin(\phi - \lambda - i)$ / $\cos(\delta + \alpha + \lambda) \cos(i - \alpha)$)^05)^2	=	2.2799
$C_{AD} = \dfrac{(1 \pm \alpha_v)(\cos^2(\phi - \lambda - \alpha))}{\cos\lambda \cos^2\alpha \cos(\delta + \alpha + \lambda)} * \Psi$	=	0.418

03/06/2022 URP | RAJUK | S-8 COMPONENT

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IRS Seismic Code

DUCTILE DETAILING

- B-1.1 Minimum grade of concrete should be M25 (f_{ck} = 25 MPa).
- Longitudinal Bar 1% to 8% but not more than 6%
- Lap is not allowed in 2xD from the plastic hinge region

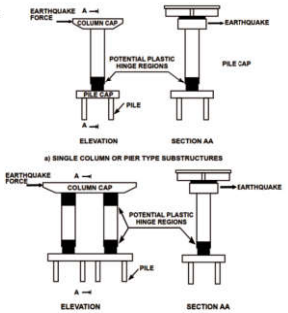

Fig. 13 Potential Location of Plastic Hinges in Substructures

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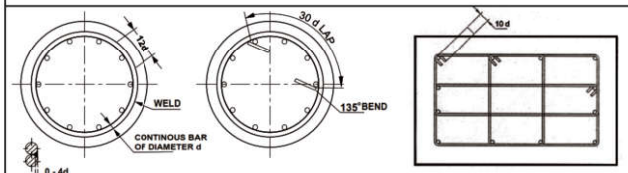
DUCTILE DETAILING



a) WELDING IN CIRCULAR HOOPS b) LAPPING IN CIRCULAR HOOPS c) RECTANGULAR HOOPS

Fig. 14 Transverse Reinforcement in Column

03/06/2022 URP | RAJUK | S-8 COMPONENT

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IRS Seismic Code

DUCTILE DETAILING

B-5.5 Amount of Transverse Steel to be Provided

B-5.5.1 The area of cross-section, A_{sh}, of the bar forming circular hoops or spiral, to be used as special confining reinforcement, shall not be less than

$$A_{sw} = 0.09SD_k\left[\frac{A_g}{A_c} - 1.0\right]\frac{f_{ck}}{f_y}$$

OR,

$$A_{sw} = 0.024SD_k\frac{f_{ck}}{f_y}$$

Whichever is the greater

where

A_{sh}	=	area of cross-section of circular hoop;
S	=	pitch of spiral or spacing of hoops, in mm;
D_k	=	Diameter of core measured to the outside of the spiral or hoops, in mm;
f_{ck}	=	characteristic compressive strength of concrete;
f_y	=	yield stress of steel (of circular hoops or - spiral);
A_g	=	gross area of the column cross-section; and
A_c	=	Area of the concrete core $\frac{\pi}{4}D_c^2$

B-5.5.2 The total area of cross section of the bar forming rectangular hoop and cross ties, A_{sh}, to be used as special confining reinforcement shall not be less than

$$A_{sw} = 0.24Sh\left[\frac{A_g}{A_k} - 1.0\right]\frac{f_{ck}}{f_y}$$

Or

$$A_{sw} = 0.096Sh\frac{f_{ck}}{f_y}$$

Where

h = longer dimension of the rectangular confining

A_k = hoop measured to its outer face; and area of confined core concrete in the rectangular hoop measured to its outer side dimensions

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IRS Concrete Bridge Code Load Combination

LOADS TO BE TAKEN IN EACH COMBINATION WITH APPROPRIATE V_R
(Clauses 11.2 and 11.3)

LOAD		LIMIT STATE	V_R TO BE CONSIDERED IN COMBINATION				
			1	2	3	4	5
Dead weight of concrete		ULS	1.25	1.25	1.25	1.25	1.25
		SLS	1.00	1.00	1.00	1.00	1.00
Superimposed dead load		ULS	2.00	2.00	2.00	2.00	2.00
		SLS	1.20	1.20	1.20	1.20	1.00
Wind	During erection	ULS	-	1.25	-	-	-
		SLS	-	1.00	-	-	-
	with dead and superimposed dead loads only and for members primarily resisting wind loads	ULS	-	1.60	-	-	-
		SLS	-	1.00	-	-	-
	With dead plus superimposed dead plus other appropriate combination 2 loads	ULS	-	1.25	-	-	-
		SLS	-	1.00	-	-	-
	Relieving effect of wind	ULS	-	1.00	-	-	-
		SLS	-	1.00	-	-	-
Earth quake	During erection	ULS	-	1.25	-	-	-
		SLS	-	1.00	-	-	-
	With dead and superimposed dead loads only	ULS	-	1.60	-	-	-
		SLS	-	1.00	-	-	-
	With dead plus superimposed dead plus other appropriate combination 2 loads	ULS	-	1.25	-	-	-
		SLS	-	1.00	-	-	-
Temperature	Restraint against movement except frictional	ULS	-	-	1.50	-	-
		SLS	-	-	1.00	-	-
	frictional restraint	ULS	-	-	-	1.50	-
		SLS	-	-	-	1.00	-
	Differential temperature effect	ULS	-	-	1.15	-	-
		SLS	-	-	0.80	-	-
Differential settlement		ULS	As specified by engineer				
		SLS					
Earth Pressure	Fill retained and or live load surcharge	ULS	1.70	1.70	1.70	1.70	-
		SLS	1.00	1.00	1.00	1.00	-
	relieving effect	ULS	1.00	1.00	1.00	1.00	-
Erection temporary loads (when being considered)		ULS	-	1.30	1.30	-	-
Live load on foot path		ULS	1.50	1.25	1.25	-	-
		SLS	1.00	1.00	1.00	-	-
Live load		ULS	1.75	1.40	1.40	-	-
		SLS	1.10	1.00	1.00	-	-
Derailment loads		(As specified by bridge rules for combination 5 only)					

NOTE 1-ULS : Ultimate limit state SLS : serviceability limit state
NOTE 2- Superimposed dead load shall include dead load of ballast, track, ballast retainer, precast footpath, wearing course, hand rails, utility services,kerbs etc
NOTE 3- Wind and earth quake loads shall not be assumed to be acting simultaneously.
NOTE 4- Live load shall also include dynamic effect, forces due to curvature exerted on track, longitudinal forces, braking forces and forces on parapets.

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Part 2

IRS Substructure Rules

GOVERNMENT OF INDIA
MINISTRY OF RAILWAYS
(Railway Board)

INDIAN RAILWAY STANDARD

CODE OF PRACTICE FOR THE DESIGN OF SUB-STRUCTURES AND FOUNDATIONS OF BRIDGES

(BRIDGE SUB-STRUCTURES & FOUNDATION CODE)

ADOPTED – 1936
FIRST REVISION -1985
SECOND REVISION -2013

(Incorporating Correction Slip Upto ACS 9 dt 06.07.2020)

Disclaimer: This Compilation is for educational reference only. For details refer original correction slips.

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IRS Substructure & Foundation Rules

4.8 Clearance, 4.9 Free Board

4.8 CLEARANCE (C)

4.8.1 The minimum clearance for bridges excluding arch bridges, syphons, pipe culverts and box culverts from the water level of design d is c h a r ge (Q) including afflux s h a l l b e i n accordance with Table below **(ACS 1 dt 17.4.14)**

Discharge in cumecs	Vertical clearance (mm)
0-30	600
31-300	600-1200(Pro-rata)
301-3000	1500
Above 3000	1800

4.9 FREE BOARD (F)

- 4.9.1. The free-board from the water level of the design discharge (Q) to the formation level of the Railway embankment or the top of guide bund shall not be less than 1m. In cases where heavy wave action is expected, the free-board shall be increased suitably.

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IRS Substructure & Foundation Rules

5.7 EARTH PRESSURE (EP)

5.7.1 All earth retaining structures shall be designed for the active pressure due to earth fill behind the structure. The general condition encountered is illustrated in (Fig.2)

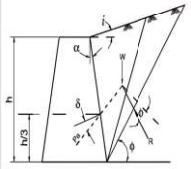

Fig -2
The active pressure due to earth fill shall

$P_a = ½ Wh^2K_a$ where :-

P_a = Active earth pressure per unit length of wall.

W = Unit weight of soil.

h = height of wall.

ϕ = angle of internal friction of back fill soil.

δ = angle of friction between wall and earth fill where value of δ is not determined by actual tests, the following values may be assumed.

(i) δ = 1/3 ϕ for concrete structures.

(ii) δ = 2/3 ϕ for masonry structures.

i = angle which the earth sirface makes with horizontal behind the earth retaining structure.

α = angle which the back surface of earth retaining structure makes with vertical.

K_a = Coefficient of static active earth pressure condition.

$$K_a = \frac{\cos^2(\phi-\alpha)}{\cos^2\alpha\cos(\alpha+\delta)\left[1+\sqrt{\frac{\sin(\phi+\delta)\sin(\phi-i)}{\cos(\alpha+\delta)\cos(\alpha-i)}}\right]^2}$$

K_a= Coefficient of Active Earth Pressure

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IRS Substructure & Foundation Rules

5.7 EARTH PRESSURE (EP)

5.7.1.6 The passive pressure P_p due to the soil shall be calculated in accordance with the formula :

$$P_p = \frac{1}{2} W h^2 K_p$$

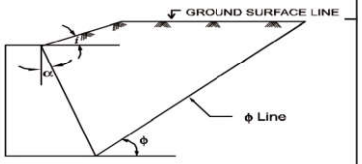
Where,

P_p = Passive earth pressure per unit length of wall

W = Unit weight of soil

h = height from the base of the wall to the top surface of the soil.

K_p = Coefficient of static passive earth pressure.



$$K_p = \frac{\cos^2(\phi+\alpha)}{\cos^2\alpha\cos(\alpha-\delta)\left[1-\sqrt{\frac{\sin(\phi+\delta)\sin(\phi+i)}{\cos(\alpha-\delta)\cos(\alpha-i)}}\right]^2}$$

If at-rest earth pressure coefficient can be estimated by the following equation for normally consolidated soils:

Ko = 1 – sin φ

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IRS Substructure & Foundation Rules

5.7 EARTH PRESSURE (EP)

5.7.1.8 Where such tests are not done, values of ϕ for granular soil may be assumed as given in Table -1.

TABLE 1

Material	Loose state	Dense State
(a)Sand Coarse	33 Degrees	45 Degrees
(b) Sandy gravel	35 "	45 "
(c) Silty and fine sand	30 "	35 "

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IRS Substructure & Foundation Rules
5.8 EARTH PRESSURE DUE TO SURCHARGE

5.8.1 Earth pressure due to surcharge on account of live load and dead loads (i.e. track, ballast etc.) shall be considered as equivalent to loads placed at formation level and extending up to the front face of ballast wall.

The surcharge due to live loads for the different standards of loading is indicated in Table-3.

TABLE-3

Standard of loading	Surcharge,S (Kg/m)	Width of uniform distribution at formation Level, B (m)
DFC Loading (32.5t axle load)	16,300	3.0
25t Loading-2008	13,700	3.0
HM Loading - 1995	15,800	3.0
Modified BG-1987	13,700	3.0
Modified MG-1988	9,800	2.1
MGML	9,800	2.1
NG'A' Class	8,300	1.8

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IRS Substructure & Foundation Rules
5.8.2 Earth Pressure Due To Surcharge On Abutments

- The horizontal active earth pressure P due to surcharge, dead and live loads per unit length on abutment will be worked out for the following two cases.

Case-1 : When depth of the section h is less than (L-B).

Case-2 : When depth of the section h is more than (L-B) .

Where : L= Length of the abutment; B= Width of uniform distribution of surcharge load at formation level; and h= Depth of the section below formation level.

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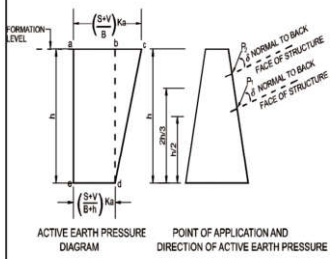
IRS Substructure & Foundation Rules
5.8.2 Earth Pressure Due To Surcharge On Abutments

Case-1 : h ≤ (L-B)

The active earth pressure diagrams are as under :



Whereas -

S = Live load surcharge per unit length

V = Dead load surcharge per unit length

P_1 =Force due to active earth pressure on 'abde'

P_2 = Force due to active earth pressure on 'bcd'.

$P_1 = \frac{(S+V)}{(B+h)} h.k_a$, acting at $\frac{h}{2}$ from section under consideration

$P_2 = \frac{(S+V)h^2 K_a}{2B(B+h)}$, acting at $\frac{2h}{3}$ from section under consideration.

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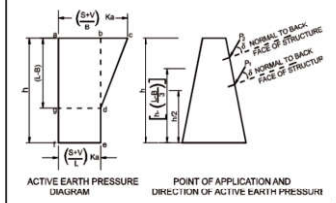
IRS Substructure & Foundation Rules
5.8.2 Earth Pressure Due To Surcharge On Abutments

Case-2 : h >(L-B)

The active earth pressure diagrams are a- under :



P_1 = Force due to active earth pressure on "abdefg"

P_2 = Force due to active earth pressure on "bcd"

$P_1 = \frac{(S+V)}{L} K_a h$

acting at $\frac{h}{2}$ from section under consideration

$P_2 = \frac{(S+V)(L-B)^2 K_a}{2BL}$ acting at

$\left[h - \left(\frac{L-B}{3}\right)\right]$ from section under consideration.

Where,

S = Live load surcharge for unit length

V = Dead load surcharge for unit length.

h = Height of fill.

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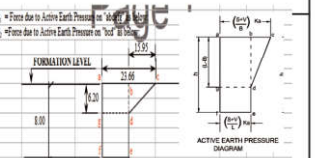
IRS Substructure & Foundation Rules
5.8.2 Earth Pressure Due To Surcharge On Abutments

Example for Case-2: h≥ (L-B)

Earth Pressure Due to Surcharge on Abutments

(Static Condition)

h = Depth of section below formation level						=	8.00	m
L = Length of Abutment						=	9.20	m
Angle of Internal Friction of Backfill Materials, φ						=	30.00	deg.
Live Load Surcharge, S						=	13,700	kg/m
B = Width of uniform distribution of surcharge load at formation level						=	3.00	m
h ≥ (L - B) : CASE-2								
Calculation of Dead Load Surcharge, V								
Weight of Rails:		=	60	Kg/m		=	7.06	Kn
Weight of Sleepers: 2 Nos.								
Size (mm)	2754	x	220	x	250	=	14.30	Kn
Weight of Ballasts:								
Size (mm)	4800	x	1000	x	400	=	60.29	Kn
					Total.	w=	81.65	Kn
Total Ballast width, Z=						=	4.80	m
Longitudinal Distribution								
Angle of internal friction						=	30.00	deg
90 - 45 + φ/2		=	30.0	deg				
b= Width of the Sliding Wedge						=	4.62	m
So, Dead Load Surcharge per metre Run,						=	78.57	Kn/m
Live Load Surcharge per Unit Length, S from table						=	134.40	Kn/m
Co-eff., Active Earth Press. (Ca)						=	0.333	



Earth Pressure due to Surcharge only

Taken due to Dead and Live Load Surcharge		
Active Earth Pressure, Rectangular Part, = (S+V)/L/*Ca	= 7.72	kn/m²
Active Earth Pressure at top, = (S+V)/B*Ca	= 23.66	kn/m²
Force due to Active Earth Pressure, Rectangular Part, P_1 = (S+V)/L*Ca*h	= 61.73	Kn/m
Acting at h/2 from section under consideration at base	= 4.00	m
Active Earth Pressure, Triangular Part	= 15.95	kn/m²
Force due to Active Earth Pressure, Triangular Part, P_2 = ((S+V)*(L-B)Ka)/2BL**	= 49.43	Kn/m
Acting at (h-((L-B)/3)) from section under consideration	= 5.95	m

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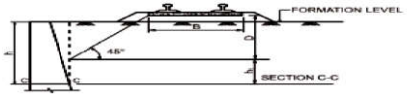
IRS Substructure & Foundation Rules
5.8.3 Earth Pressure Due To Surcharge On Wing Wall

5.8.3 Earth Pressure due to Surcharge on Return Walls : The earth pressure due to surcharge on return walls of BOX type abutments may be assumed to be dispersed below the formation level at a slope of one horizontal to one vertical. The pressure due to live load and dead load surcharge shall be calculated by the formula:

$$P_s = \frac{(S+V)h_1 K_a}{(B+2D)}$$

This pressure will be assumed to be acting at a distance of $h_1/2$ above the section considered as shown in Fig.5(a)



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IRS Substructure & Foundation Rules
5.8.3 Earth Pressure Due To Surcharge On Wing Wall

Earth Pressure Due to Surcharge on Return Walls

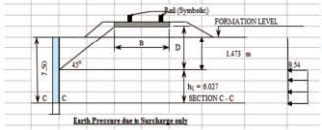
Calculation of Dead Load Surcharge

Weight of Rails:			60	Kg/m		=	3.53	Kn
Weight of Sleeper: 2 Nos.								
Size (mm):	2754	x	220	x	200	=	5.72	Kn
Weight of Ballasts:								
Size (mm):	1800	x	1000	x	200	=	22.41	Kn
					Total:	=	31.66	Kn
So, Dead Load Surcharge per meter Run						=	28.74	Kn/m

Longitudinal Distribution

Angle of internal friction				=	30.00 deg
90 - 45 = φ/2	=	30	deg		
Width of the Sliding Wedge				=	4.33 m
So, Dead Load Surcharge per meter Run				=	28.74 Kn/m

Pressure Intensity due to Surcharge with both DL & LL



Earth Pressure due to Surcharge only

Live Load Surcharge per Unit Length, S	=	134.40	kn/m
Height of the Section below the Formation Level, h	=	7.50	m
Angle of Internal Friction of Backfill Materials, φ	=	30.00	deg.
Clear Distance between the Return Walls	=	5700	mm
Top of Rail in the Track	=	10.690	m
Formation Level / Top of Abutment / Top of Return Wall	=	9.967	m
Height of Rail	=	0	mm
Thickness of Sleeper	=	0	mm
Length of the Sleeper, B	=	2754	mm
Total Depth of Surcharge	=	723	mm
Dead Load Surcharge per Unit Length, V	=	28.74	kn/m
Co-eff., Active Earth Press. (Ca)	=	0.333	
Thickness of Cushion below the Sleeper	=	723	mm
Depth at which Surcharge Pressure acts from bottom of Sleeper, D	=	1473	mm
The Depth over which the Surcharge acts, h_1	=	6027	mm
The DL and LL Surcharge Pressure Intensity = (S+V)/(B+2*D)*Ca	=	9.54	kn/m
Acting at (h_1/2) from section C-C	=	3.014	m

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IRS Substructure & Foundation Rules
5.9. FORCES DUE TO WATER CURRENT (WC)

5.9.2.1On piers parallel to the direction of water current the water pressure shall be calculated by the formula :

$P = KAV^2$,

P = Total pressure in kg due to water current.

A = Area in square metres of elevation of the part exposed to the water current.

V = The maximum mean velocity of current in metre per second

K = A constant having values for different shapes of piers as given in Clause 5.9.2.2.

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IRS Substructure & Foundation Rules
5.9. FORCES DUE TO WATER CURRENT (WC)

TABLE 4

S.No	Description	Figure	Value of K
1.	Square-ended piers.		79
2.	Circular piers or piers with semi-circular ends.		35
3.	Piers with triangular cut-and ease-waters, the angle included between the faces being 60 degrees.	60°	37
4.	Piers with triangular cut-and ease-waters, the angle included between the faces being 90 degrees.	90°	47
5.	Piers with cut-and ease- waters of equilateral arcs of circle at 60 degrees.	60°	24
6.	Piers with arcs of the cut and ease waters intersecting at 90 degrees.	90°	26

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IRS Substructure & Foundation Rules
5.12 Seismic

- 5.12.1.1 Slab, box and pipe culverts need not be designed for seismic forces.
- For design of substructures of bridges in different zones, seismic forces may be considered as given below :-
- Zone I to III : Seismic forces shall be considered only in case of bridges of overall length more than 60m or spans more than 15m.
- Zone IV and V: Seismic forces may be considered for all spans.

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IRS Substructure & Foundation Rules

5.12 Load Combination

❑ (a) Combination I –
The worst possible combination of Dead load **(DL),** Live load (LL), Dynamic augment (I), Longitudinal forces (LF), Forces due to curvature and eccentricity of track (CF), Earth Pressure (EP), Forces due to water current (WC) and Buoyancy (B), Temperature effects where considered (TMP) and Effects due to resistance of expansion bearings to movements (EXB)

(b) Combination II - The worst possible combination of forces mentioned in combination I along with Wind pressure effect (WL).

IRS Substructure & Foundation Rules

5.12 Load Combination

(c) Combination III - In case of bridges for which seismic forces have to be considered as per clause 5.12.1.1, the worst possible combination of forces in combination I plus forces and effects due to earth quake Seismic forces (SF). Wind pressure effect need not be taken into account when seismic effect is considered

(d) Combination IV - The worst possible combination of all loads and forces and effects which can operate on any part of the structure during erection. For bridges for which seismic forces have to be considered as per clause 5.12.1.1, either wind pressure effect or seismic effect whichever gives the worst effect need only be considered.

IRS Substructure & Foundation Rules

5.15 PERMISSIBLE INCREASE IN STRESSES

5.15.1 For combination of loads stated in clause 5.13, the permissible stresses on substructures shall be increased as follows : -

Combination I ..... Nil

Combination II &III.... 33 %

Combination IV 40%

IRS Substructure & Foundation Rules

5.15 PERMISSIBLE INCREASE IN Allowable Bearing Pressure

6.7 Permissible Increase In Allowable Bearing Pressure :
6.7.1 When the foundations are checked for combinations of loads as stated in clause 5.13, the allowable bearing pressures on foundations may be increased as follows :

Combination I - Nil
Combination II &III - 33 %
Combination IV - 40%

Part 3

IRS Steel Bridge Code

PRB – 230
2000

Government of India
Ministry of Railways
(Railway Board)

INDIAN RAILWAY STANDARD
CODE OF PRACTICE FOR
THE DESIGN OF STEEL OR WROUGHT
IRON BRIDGES CARRYING RAIL
ROAD OR PEDESTRIAN TRAFFIC

(Steel Bridge Code)

ADOPTED – 1941

REVISED - 1962
(Reprinted incorporating Addendum & Corrigendum Slip Nos.1 to21)

2017

ISSUED BY
RESEARCH DESIGNS AND STANDARDS ORGANISATION
LUCKNOW – 226011

03/06/2022 URP | RAJUK | S-8 COMPONENT

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IRS Steel Bridge Code

Scope

1.1 This code is primarily intended to apply to the superstructure of simply supported steel bridges of spans up to 100 m (325 ft) between centres of bearings. Where appropriate, the provisions of the code may be adopted for larger spans or other types of steel bridges, but care should be taken, in these circumstances to make whatever amendments are necessary for fixity at the supports, continuity and other indeterminate or special conditions.

1.2 Where bridges of the through or semi-through type are adopted, they must be designed to allow for clearances specified in the appropriate schedule of dimensions, for different gauges in the case of Railway bridges or bridges over Railway, and in the case of road bridges clearances as specified by the appropriate authorities.

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IRS Steel Bridge Code

3. LOADS, FORCES AND STRESSES

(a) Dead load.
(b) Live load.
(c) Impact effect.
(d) Forces due to curvature and eccentricity of Track.
(e) Temperature effect.
(f) Resistance of expansion bearings to movements
(g) Longitudinal force.
(h) Racking force.
(j) Forces on parapets.
(k) Wind pressure effect.
(l) Forces and effects due to earthquake.
(m)Erection forces and effects.
(n) Derailment loads

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IRS Steel Bridge Code

3.2 Combination of Loads and Forces-

3.2.1 The worst combination possible of dead load with live load, impact effect and forces due to curvature and eccentricity of track. When considering the member whose primary function is to resist longitudinal and racking forces due to live load, the term live load shall include these forces.

3.2.2 In case of bridges situated in seismic zones I to III as given in Bridge Rules, only bridges of overall length more than 60 m or individual span more than 15 m for the worst possible combination of any or all the items „a" to „j" & „k" or „l" listed in clause 3.1

3.2.3 In cases of bridges situated in seismic zone IV & V as given in Bridge Rules, the worst combination possible of any or all the items „a" to „j" and „k" or „l" listed in clause 3.1

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IRS Steel Bridge Code

.3 Primary and Secondary Stresses

3.2.4 The worst combination possible of loads and forces during erection.

3.2.5 In case of ballasted deck bridges, the combination of dead load and derailment load shall be considered as an occasional load.

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IRS Steel Bridge Code

3 Primary and Secondary Stresses

TABLE – 1 TOTAL VARIATION IN ALLOWABLE STRESSES

Type of Girder	Increase of allowable stresses for stress combinations as per clauses		
	3.2.1	3.2.2 & 3.2.3	3.2.4
(a) Solid Web Girder For calculated primary stress	No increase	$16\,^{2}/_{3}$%	25%
(b) Triangulated Trusses -			
(i) for calculated primary stress	No increase	$16\,^{2}/_{3}$%	25%
(ii) where primary stresses are combined with calculated secondary stresses of sub clause 3.3.2 (a) (self wt. and wind on member ignored) and with deformation stresses of sub clause 3.3.2 (b)	$16\,^{2}/_{3}$ %	$33\,^{1}/_{3}$%	40%

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IRS Steel Bridge Code

3.8 Prestressing

3.3.8. The effectiveness of prestressing in the web members of spans below 60m (200ft) and in all members of spans below 45m (150ft) shall be ignored.

3.3.9. All open web girders for railway bridges of spans 30.5 m (100ft) and above shall be prestressed. Rules for prestressing are given in APPENDIX-A

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IRS Steel Bridge Code

3.6 Fluctuations of Stress (fatigue)

3.6.1 Fluctuations of stresses may cause fatigue failure of members or connections at lower stresses than those at which they would fail under static load. Such failures would be primarily due to stress concentrations introduced by the constructional details.

3.6.4 For any structural member or connection, the fatigue design shall be done as per Appendix 'G" (Re-revised) for a specified 'Design life' and 'Fatigue Load Model'.

3.6.5 The fatigue life assessment shall normally be made for a standard design life of 100 years for a standard annual GMT of 50. However, any other design life/annual GMT may be used for design with the approval of Chief Bridge Engineer.

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IRS Steel Bridge Code

3.6 Fluctuations of Stress (fatigue)

14.6. Damage equivalence factors

14.6.1 The damage equivalent factor for railway bridges should be determined from:

$$\lambda = \lambda_1 * \lambda_2 * \lambda_3 * \lambda_4$$

subject to the condition that $\lambda \leq \lambda_{max}$ where $\lambda_{max} = 1.4$

Where,

λ_1 is a factor that takes into account the damaging effect of traffic and depends on the base length of the longest loop of the influence line diagram

λ_2 is a factor that takes into account the annual traffic volume in million tons

λ_3 is a factor that takes into account the design life of the bridge in years

λ_4 is a factor to be taken into account when the bridge structure is loaded on more than one track

λ_{max} is the maximum λ value taking into account the fatigue limit

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IRS Steel Bridge Code

3.6 Fluctuations of Stress (fatigue)

Table 14.6.2 (1): λ_1 for 25 T Loading

Span (m)	Train-1	Train-2	Train-3	Train-4	Train-5	Train-6	Train-7	Train-8	Train-9	Train-10	Train-11
0.50	1.30	1.34	1.45	1.28	1.45	1.44	1.48	1.53	1.36	1.09	0.88
1.00	1.29	1.32	1.43	1.28	1.43	1.43	1.48	1.53	1.35	1.08	0.89
1.50	1.28	1.31	1.42	1.27	1.42	1.42	1.47	1.52	1.35	1.07	0.90
2.00	1.27	1.30	1.40	1.27	1.40	1.41	1.47	1.52	1.35	1.06	0.90
2.50	1.26	1.29	1.38	1.26	1.37	1.39	1.46	1.51	1.34	1.05	0.91
3.00	1.24	1.28	1.36	1.25	1.34	1.38	1.45	1.50	1.34	1.04	0.92
3.50	1.22	1.26	1.34	1.23	1.32	1.36	1.44	1.49	1.33	1.03	0.93
4.00	1.20	1.25	1.32	1.22	1.30	1.34	1.43	1.47	1.31	1.02	0.94
4.50	1.18	1.23	1.30	1.21	1.28	1.31	1.42	1.46	1.30	1.00	0.95
5.00	1.17	1.21	1.28	1.19	1.26	1.27	1.41	1.45	1.28	0.99	0.96
6.00	1.12	1.17	1.24	1.15	1.23	1.21	1.39	1.42	1.24	0.96	0.97
7.00	1.08	1.14	1.19	1.11	1.18	1.16	1.37	1.39	1.19	0.93	0.98
8.00	1.05	1.12	1.17	1.09	1.16	1.12	1.38	1.37	1.14	0.91	1.00
9.00	1.03	1.11	1.15	1.07	1.13	1.08	1.38	1.36	1.09	0.90	1.02
10.00	0.96	1.03	1.12	1.15	1.08	1.10	1.37	1.37	1.08	0.89	0.99
12.50	0.89	0.98	1.05	1.06	1.03	1.02	1.32	1.32	1.00	0.86	1.01
15.00	0.87	0.92	1.00	1.01	1.05	1.00	1.30	1.31	0.99	0.83	1.02
17.50	0.82	0.86	0.94	0.93	1.04	0.94	1.24	1.24	0.93	0.78	0.98
20.00	0.83	0.86	0.99	0.94	1.07	0.89	1.13	1.09	0.89	0.79	1.01
25.00	0.76	0.86	0.93	0.85	1.08	0.87	1.13	1.11	0.87	0.75	0.99
30.00	0.77	0.82	0.84	0.80	1.09	0.88	0.98	1.10	0.87	0.69	0.96
35.00	0.73	0.75	0.76	0.76	0.87	0.86	0.93	1.09	0.86	0.66	0.90
40.00	0.66	0.68	0.70	0.73	0.85	0.78	0.84	1.07	0.78	0.63	0.75
45.00	0.64	0.66	0.68	0.77	0.81	0.77	0.82	1.01	0.78	0.63	0.65
50.00	0.62	0.63	0.65	0.77	0.80	0.75	0.79	0.91	0.76	0.62	0.66
60.00	0.59	0.60	0.62	0.77	0.77	0.74	0.77	0.78	0.74	0.60	0.67
70.00	0.58	0.59	0.60	0.75	0.76	0.75	0.77	0.78	0.75	0.61	0.64
80.00	0.56	0.58	0.59	0.66	0.76	0.74	0.76	0.77	0.75	0.60	0.63
90.00	0.56	0.57	0.58	0.64	0.76	0.72	0.75	0.76	0.75	0.58	0.63
100.00	0.55	0.56	0.58	0.61	0.76	0.73	0.74	0.75	0.74	0.55	0.63

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IRS Steel Bridge Code

3.6 Fluctuations of Stress (fatigue)

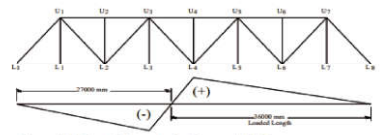

Figure 14.1 Loaded Length for Diagonal U_1-L_4

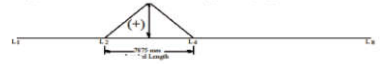

Figure 14.1 (b) Loaded Length for Verical U_3-L_3

Figure 1(c) Loaded Length for Bottom Chord L_0-L_1 & L_1-L_2

Note :- (+) For Tension and (-) For Compression

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IRS Steel Bridge Code

3.6 Fluctuations of Stress (fatigue)

14.6.4 The value of λ_2, in terms of the annual volume of traffic may be obtained from the following expression where T_a is the annual volume of traffic expressed in million tons:

$$\lambda_2 = 0.5193 * T_a^{0.2036}$$

14.6.5 The value of λ_3, in terms of the design life may be calculated from the following expression where L_D is the design life in years:-

$$\lambda_3 = 0.3899 * L_D^{0.2048}$$

14.6.6 The value of λ_4, assuming 15% of the total traffic on both tracks crosses whilst on the bridge, shall be obtained from

$$\lambda_4 = 0.7926 * a^2 - 0.7280 * a + 0.9371$$

Where $a = \Delta\sigma_1 / \Delta\sigma_{1+2}$

$\Delta\sigma_1$ = Stress range at the section being checked due to train on one track.

$\Delta\sigma_{1+2}$ = Stress range at the same section due to train load on two tracks.

The values of λ_4 may be calculated for other proportions of crossing traffic from

$$\lambda_4 = \sqrt[5]{n + (1-n)\left[a^5 + (1-a)^5\right]}$$

Where, n is the proportion of traffic that crosses simultaneously on the bridge.

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IRS Steel Bridge Code

3.6 Fluctuations of Stress (fatigue)

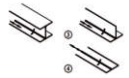
Table G-II.1 Non-welded details

Detail category	Constructional Detail	Description	Requirements
160	NOTE: The fatigue strength curve associated with category 160 is the highest. No detail can reach a better fatigue strength at any number of cycles	Rolled and extruded products: 1)Plates and flats 2)Rolled sections	Details 1) to 2) Sharp edges, surface and rolling flaws to be improved by grinding until removed and smooth transition achieved.
125		3) Sheared or gas cut plates: Machine gas cut or sheared material subsequently dressed to remove all edge discontinuities.	3) All visible signs of edge discontinuities to be removed. The cut areas are to be machined or ground and all burrs to be removed. Any machinery scratches for example from grinding operations can only be parallel to the stresses. Detail 3 - Re-entrant corners to be improved by grinding (slope < ¼) or evaluated using the appropriate stress concentration factors. - No repair by weld refill.

IRS Steel Bridge Code

3.6 Fluctuations of Stress (fatigue)

Table G-II. 2 Welded built-up sections

Detail category	Constructional Detail	Description	Requirements
125		Continuous longitudinal welds: 1) Automatic butt welds carried out from both sides. 2) Automatic fillet welds Cover plate ends to be checked using detail 5) or 6) Table G-II.5.	Details 1) to 2) No stop/start position is permitted except when the repair is performed and inspection is carried out to verify the proper execution of the repair.
112		3) Automatic fillet or butt weld carried out from both sides but containing stop/start positions. 4) Automatic butt welds made from one side only, with a continuous backing bar, but without stop/start positions.	4) When this detail contains stop/start positions category 100 to be used.
100		5) Manual fillet or butt weld. 6) Manual or automatic butt weld carried out from one side only, particularly for box girders.	6) A very good fit between the flange and web plates is essential. The web edge to be prepared such that the root face is adequate for the achievement of regular root penetration without break-out.

IRS Steel Bridge Code

3.6 Fluctuations of Stress (fatigue)

4.3.1 Detail category: The designation given to a particular structural detail for a given direction of stress fluctuation to indicate which fatigue strength curve is applicable for fatigue assessment. This is denoted by a number which represents the magnitude in N/mm2 of the stress range which is associated with an endurance of 2 million cycles for that particular category.

7.4. Values of partial safety factors: The values of the partial safety factor for fatigue loading (γ_{Ff}) and fatigue strength (γ_{Mf}) shall be taken as follows;

$\gamma_{Ff} = 1.00$

$\gamma_{Mf} = 1.15$

Example: Fatigue Stress Calculation.

Fatigue stress Calculation				Reference
For Cross Girder				
Calculation of Fatigue according to the IRS Code				
Connection Category	=	125	N/mm2	Table G-12
N	=	2000000	Cycle	
Stress range , ΔσR	=	125	N/mm2	
Damage Factor, λ=λ1*λ2*λ3*λ4				CL 14.6.1
λ1 for 32.5 t loading and span 20 m	=	1.18		Table 14.6.2 (2) 32.5t Loading
T_0= GMT= 2.08 Per Train @ 24 Train/ Day	=	50.0	GMT	
λ2=0.5193*To^0.2036	=	1.1517		14.6.4
				Cl. 3.6.5
Design Life, LD	=	100	Years	correction slip 18 GMT will be 50
λ3=0.3899*LD^0.2048	=	1.0		14.6.5
λ4	=	1	For Single Track	14.6.6
Damage Factor λ=λ1*λ2*λ3*λ4	=	1.361		
λmax	=	1.4		
Partial Factor	=	1.15	Clause Clause 7.4 , Apendix G	
OK				
λ will be taken not more than 1.40	=	1.361		
Allowable Fatigue stress= Stress Range (125) / Damage Factor (1.361) x Fartial Factor Strees (1.15)				
	=	79.88	N/mm2	
For Plate Thickness, t	=	35	mm	
Stress will be modified				
ΔσRt=ΔσRt25*(25/t)^0.2	=	74.68	N/mm2	CL. 10.2.2.3
Allowable fatigue stress	=	74.68	Mpa	

2.5 FORCES DUE TO CURVATURE AND ECCENTRICITY OF TRACK

4.16 Camber

4.16.1 Beams and plate girder spans up to and including 35 m (115 ft) need not be cambered.

4.16.2 In un-prestressed open web spans, the camber of the main girders and the corresponding variations in length of members shall be such that when the girders are loaded with full dead load plus 75 per cent of the live load without impact producing maximum bending moment, they shall take up the true geometrical shape assumed in their design.

4.16.3 Where girders are prestressed the stress camber change should be based on full dead load and live load including impact.

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IRS Steel Bridge Code

4.16 Camber

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4.16.3 Where girders are prestressed the stress camber change should be based on full dead load and live load including impact.

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IRS Steel Bridge Code

4.17 DEFLECTION

4.17 Deflection- For permanent installation other than foot-over-bridges the ratio of deflection to length of the girder shall not exceed 1/600.

In the case of foot-over-bridges, the ratio of deflection to length of the girder shall not exceed 1/325.

Note:-

With the specific sanction of the Board, the limit of 1/600 may be exceeded for girders in permanent installations.

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IRS Steel Bridge Code

7.12 HIGH STRENGTH GRIP BOLT (HSFGB)

7.12 High Strength Friction Grip (HSFG) Bolts - High Strength Friction Grip (HSFG) bolts shall be provided as per para 28.9 of IRS Specification For Fabrication And Erection Of Steel Girder Bridges And Locomotive Turn Tables (Fabrication Specification) Serial No. B1-2001. These bolts shall be designed as per provisions of IS:4000-1992 (High Strength Bolts in Steel Structures - Code of Practice) with the following over-riding clauses:

7.12.1 Provisions not covered in IS:4000-1992 shall conform to IRS Codes. For edge distance, gauge distance and pitch etc., provisions of para 7 shall be followed.

7.12.2 Joints with HSFG bolts shall be designed only as friction type joints. Bearing type joints with HSFG bolts shall not be provided.

7.12.3 Short/long slotted holes shall not be allowed.

7.12.4 The diameter of hole shall be 1.5 mm more than the nominal diameter of the

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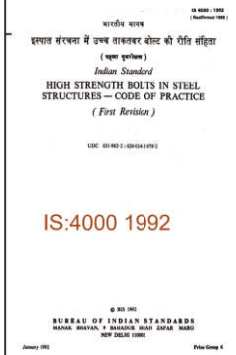
IRS Steel Bridge Code

7.12 HIGH STRENGTH GRIP BOLT (HSFGB)


भारतीय मानक

इस्पात संरचना में उच्च ताकतवर बोल्ट की रीति संहिता

(पहला पुनरीक्षण)

Indian Standard

HIGH STRENGTH BOLTS IN STEEL STRUCTURES — CODE OF PRACTICE

(First Revision)

BUREAU OF INDIAN STANDARDS
MANAK BHAVAN, 9 BAHADUR SHAH ZAFAR MARG
NEW DELHI 110002

IS:4000 1992

Grade of HSFGB

- 10.9 Grade means Ultimate strength 1000 mPa and Yield Strength 90% of Ultimate Strength 900 mPa.
- 8.8 Grade means Ultimate strength 800 mPa and Yield Strength 80% of Ultimate Strength 640 mPa.

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IRS Steel Bridge Code

7.12 HIGH STRENGTH GRIP BOLT (HSFGB)

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IRS Steel Bridge Code

7.12 HIGH STRENGTH GRIP BOLT (HSFGB)

HSFG bolt for bolt dia less than 25mm and 2mm more than the nominal diameter of the HSFG bolt for larger diameters.

7.12.5 In certain cases, HSFG bolts might be required to be provided in oversize holes. The maximum size of oversize holes shall not exceed 1.25 d or d + 4 mm, whichever is less where d is nominal diameter of HSFG bolt.

7.12.6 Design of Friction type joint shall be done as follows:

7.12.6.1 For joints subjected to shear force only- shear force applied should not exceed.

$$\frac{\text{Slip factor} \times \text{Number of Effective Interfaces} \times \text{Minimum Bolt Tension}}{\text{Factor of Safety}}$$

Minimum Bolt Tension shall be as given in table XII below:

Nominal size of Bolt	Minimum Bolt tension in KN	
	Property class 8.8	Property class 10.9
M16	94.5	130
M20	147	203
M22	182	251
M24	212	293
M30	337	466
M36	490	678

Table XII (Based on Table 3 of IS:4000-1992)

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IRS Steel Bridge Code

7.12 HIGH STRENGTH GRIP BOLT (HSFGB)

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Table XII (Based on Table 3 of IS:4000-1992)

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IRS Steel Bridge Code

7.12 HIGH STRENGTH GRIP BOLT (HSFGB)

7.12.6.2 For joints subjected to shear and tensile force.

$$\frac{\text{Caculated Shear}}{\text{Slip Factor x No. of Effective Interface}} \leq \frac{(\text{Proof Load - Calculated Tension x F})}{\text{Factor of Safety}}$$

where factor F shall be taken as 2.0 if external tension is repetitive and 1.7 if non repetitive. Proof Load shall be worked out for the yield stress of the bolt used and the stress area of thread as given in Table 2 of IS:4000-1992.

7.12.6.3 In 7.12.6.1 and 7.12.6.2, the factor of safety shall be taken as follows:

(a) 1.4 for normal loads

(b) 1.2 when wind load is considered provided that (i) connections are adequate when wind forces are not considered (ii) wind load is not the primary loading for the purpose of design.

7.12.6.4 Slip factor for design shall be taken as per table XIII

IRS Steel Bridge Code

7.12 HIGH STRENGTH GRIP BOLT (HSFGB)

S.No.	Surface preparation of the interface between plies in a HSFG bolted joint	Slip factor
1.	Surface blast cleaned and spray metallized with aluminium (thickness > 100μm), with no over coating	0.40
2.	Surfaces cleaned by wire brushing or flame cleaning, with loose rust and paint layers removed (only isolated patches of coatings/ rust can remain)	0.25
3.	Any other surface preparation	To be established as per procedure given in Annexure B of IS:4000-1992

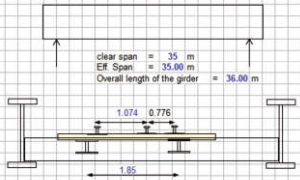
Table XIII

Example: Design of Stringer (open Deck) Cont.

Subject : Design of Semi Through Girder - 35m Span (Broad Guage) with Footpath

Calculation of Stringer (Open Deck) | Reference

1 **Design of Stringer**



Spacing of the cross girders	=	2.625	m
Effective Span of the stringer	=	2.625	m
Dead load of stringer			
(Track Load + Self wt.) ([illegible] + 0.13) t/m	=	0.78224	t/m
Dead Load on the stringer	=	2.05338	t
% of Load on the stringer due to Metro Gauge Loading	=	70.973	%
coefficient of Impact load (1+L)	=	0.955	

Loads due to Live Load	B.M(W)	Shear(P)	
LL for MMG-1988 -EUDL (T)	32.3	43.28	t
LL per Girder = 70.97 % of one track load	22.924	30.717	t
Live Load for 32.5t DFC load	65	83.42	t
LL per Girder = 50% of one track load	32.5	41.71	t
B.G Governs in the Design			

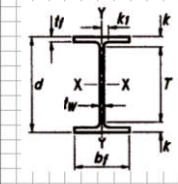
Page 1

Example: Design of Stringer (open Deck) Cont.

B.G Governs in the Design			
Impact Load per stringer	31.04	39.83	t
Dead Load of the stringer	2.053	2.053	t
Number of Stringer	2.000		
Total Load on the Stringer	**65.59**	**83.60**	t
Max. Bending Moment	**21.52**		tm
Max. Shear Force		**41.80**	t
Max. Bending Moment for live load only	20.849		tm
Max. Shear force for live load only		**40.77**	t

Sectional Properties of the Member — From AISC Manual

Member	W	16	x	89		
Yield Stress	50	ksi		345	Mpa	
Weight	89	lb/ft		132	Kg/m	



Area			=	169.032	cm²
Overall Depth			=	42.545	cm
Web Thickness			=	1.3335	cm
Flange width			=	26.3271	cm
Flange Thickness			=	2.2225	cm
Mom. of Inertia	Ixx		=	54110.1	cm⁴
Ybs = 21.3 cm	Zbs		=	2543.66	cm³
Yts = 21.3 cm	Zts		=	2543.66	cm³
Mom. of Inertia	Iyy		=	6784.57	cm⁴

Example: Design of Stringer (open Deck) Cont.

Permissible Stresses :	Compressive stresses					
Bending Compression Stress considering lateral Torsional Buckling				Cs Values		cl: 3.9 of IRS Steel Bridge Code
case 1	Flanges have equal moment of Inertial about y axis			A		
case 2	Moment of Inertia of the compression flange about y -y axis exceeds that of Tension flange			$A+k_2B$		
case 3	Moment of Inertia of the Tension flange about y-y axis exceeds that of compression flange			$(A+k_2B)Yc/Yt$		
where	$A = \frac{267730}{(l/r_y)^2}\sqrt{\left\{1-\frac{1}{20}\left(\frac{l_e}{r_y D}\right)^2\right\}}$	Yc =	Distance from N.A of girder to extreme fibre in compression			
	$B = \frac{267730}{(l/r_y)^2}$	Yt =	Distance from N.A of girder to extreme fibre in Tension			
k2 = A coefficient to allow for inequality of tension and compression flanges						
te = Effective Thickness of Flange						
Moment of Inertia of the Top flange about YY axis				= 3379.63	cm⁴	
Moment of Inertia of the Bottom flange about Yyaxis				= 3379.63	cm⁴	
It comes in Case 1,Cs = A						IRS SBC
Effective Length of the compression Flange				= 2625	mm	cl: 5.4.4
Radius of the Gyration about YY axis			ry	= 63.35	mm	
Overall Depth of the Section			D	= 425.45	mm	
Effective Thickness of the Flange	K1 = 1		te	= 22.225	mm	Table V cl:5.3
			l / ry	= 41.4365		
			D/te	= 19.1429		
			A	= 173.24	kg/mm²	
			B	= 155.931	kg/mm²	
Ratio M. of Comp. flange to M.I of compr.+tension flange			Rm	= 0.5		
			k2	= 0		Table VI
			Cs	= 173.236	kg/mm²	

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Example: Design of Stringer (open Deck) Cont.

				IRS SBC
Allowable Bending Compressive stress	Pbc	= 21.2	kg/mm²	Table VIII
for equivalent steel considered IS 2062 : Grade E350 (Fe490 W)				
Max. Permissible Bending Stress (if thickness is 20mm or less)		= 22.5	kg/mm²	Table II
(if thickness is more than 20mm		= 21.9	kg/mm²	of IRS SBC
Max. Permissible Shear Stress (if thickness is 20mm or less)		= 13	kg/mm²	Table II
(if thickness is more than 20mm		= 12.7	kg/mm²	of IRS SBC
Stresses with Fatigue Consideration				
fmin / fmax = 2.05338 / 83.60		= 0.025		
Class of Connection = Category 160 No of Cycles		= 2 Million		Appendix "G"
Allowable Fatigue Stress in Compression		= 18.1	kg/mm²	of IRS SBC
Allowable Fatigue Stress in Tension		= 10.13	kg/mm²	
Considering all above				
Actual Bending Stress Compression		= 8.46116	kg/mm²	
Allowable Bending Stress		= 18.1	kg/mm²	
		O.K		
Aactual Bending Stress Tension		= 8.46116	kg/mm²	
Allowable Bending Stress		= 10.13	kg/mm²	
		O.K		
Actual Shear Stress		= 8.227	kg/mm²	
Permissible shear stress		= 13	kg/mm²	
		O.K		

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Example: Design of Stringer (open Deck) Cont.

Design of Connections				
Design of Cleat Angle to Stringer Beam	connection is by HSFG Bolts			IS4000
Propose Cleat anlge	150 x 150 x 9.53 mm 5.9 x 6.9 x 0.375 in			
Dia of the Bolt Proposed to use		= 24	mm	
Dia of the hole		= 26	mm	
Grade of the Bolt		= 8.8		
Considering Bearing Type of joint				Table II
Max. Permissible force in the bolt		= 73.2	Kn	
Limitation capacity of the bolt	fy = 350 t = 10 mm	= 96.0	Kn	
Capacity of the Bolt		= 73.2	Kn	
No of bolts required		= 6	No's	
Hence Provide 6 No's 24 mm dia HSFG Bolts				
Considering Friction Type of joint				
Friction coefficient		= 0.3		
No of Interfaces		= 2		
Factor of safety		= 1.4		
Minimum Bolt Tension		= 212	Kn	
Permissible force in the bolt		= 90.8571	Kn	
Limitation capacity of the bolt	fy = 350 t = 10 mm	= 96.0	Kn	
Capacity of the Bolt		= 90.9	Kn	Table III
No Bolts required		= 5	No's	
Hence Provide 5 No's 24 mm dia HSFG Bolts				
Considering Fatigue for bolts				
Category		= 100	Mpa	Table 9.1
Material safety		= 1.15		of IRS SBC
λmax		= 1.4		
Maximum Allowable stress per bolt		= 62.11	MPa	
Capacity of the Bolt		= 28.10	kN	
Shear force for live load		= 40.77	t	
Nos of face		= 2		
Nos of bolts required		= 8.00	Nos.	
Number of bolts provided in each row		= 4	Nos.	
Number of rows		= 2	Nos.	
Provide Edge distance from C.G of the Bolt	= 40 mm	O.K		
	= 60 mm	O.K		
Length of the Cleat Angle	= (40 + 180 + 40)	= 260	mm	

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Example: Design of Stringer (open Deck)

Design of cleat Angle to Cross Beam			
Force on the Member		= 41.80	t
Eccentricity of the force to the Member		= 0.05	m
Moment on the Member		= 2.09	tm
Dia of the Bolt Proposed to use		= 24	mm
Dia of the hole		= 26	mm
Grade of the Bolt		= 8.8	
Considering Bearing Type of joint			
Max. Permissible force in the bolt		= 73.2	Kn
Limitation capacity of the bolt	fy = 350 t = 10 mm	= 96.0	Kn
Capacity of the Bolt		= 73.2	Kn
No of bolts required		= 6	No's
Hence Provide 2 - 8.00 No's 24 mm dia HSFG Bolts		on either side of stringer	
Shear Force on the each Bolt	V	= 26.1243	Kn
Tensile force on the Bolt due to moment	Pt	= 40.1912	Kn
Permissible Tensile force of the Bolt	Pto	= 127	Kn
By condition V / Vo + Pt / Pto		= 0.22752	<=1, O.K
Considering Friction Type of joint			
Friction coefficient		= 0.3	
No of Interfaces		= 1	
Factor of safety		= 1.4	
Minimum Bolt Tension		= 212	Kn
Permissible force in the bolt		= 45.43	Kn
Limitation capacity of the bolt	fy = 350 t = 10 mm	= 96.0	Kn
Capacity of the Bolt		= 45.4	Kn
No Bolts required		= 10	No's
Hence Provide 16 No's 24 mm dia HSFG Bolts			
Calculated Shear on the bolt	V	= 26.1243	Kn
Calculated Tension on the Bolt	Pt	= 40.1912	Kn

As per the expression Calculated shear ≤ (Proof load - Cal.Tension xF)

slip factor x N O I / Factor of safety

26.1243 ≤ 212 - 40.191 x 2.0 / 0.3 x 1 / 1.4

87.081 ≤ 94.0125 Kn O.K

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Table II

TABLE II -- BASIC PERMISSIBLE STRESSES IN STRUCTURAL STEEL

Description	Mild steel to IS: 226 and IS: 2062 with yield stress of				High tensile steel grade 58-HTC to IS: 961 with yield stress of					
	26 kg/mm²	16.5 ton/in²	24 kg/mm²	15.2 ton/in²	36 kg/mm²	22.9 ton/in²	35 kg/mm²	22.3 ton/in²	33 kg/mm²	21.0 ton/in²
1	2	3	4	5	6	7	8	9	10	11
Parts in Axial Tension On effective sectional area ...	15.4	9.8	14.2	9.0	21.3	**13.5**	20.7	13.1	19.5	12.4
Parts in Axial Compression on Effective gross section ...					**See**	**Clause**	3.8			
Parts in bending (Tension or Compression). On effective sectional area for extreme fibre stress -- (i)For plates, flats, tubes, rounds, square and similar sections.	17.0	10.8	15.7	10.0	**23.5 See**	**14.9 also**	22.8 **Clause3.9**	14.5	21.6	13.7
(ii) For rolled beams, channels, angles and tees, and for plate girders with single or multiple webs with d_1/t not greater than 85 for steel to IS:226 and IS:2062 d_1/t not greater than 75 for steel to IS:961.	16.3	10.4	15.0	9.5	22.5 **See**	**14.3 also**	21.9 **Clause 3.9**	13.9	20.6	13.1
ii) For plate girder with single or multiple webs with : d_1/t greater than 85 for steel to IS:226 and IS:2062, d/t greater than 75 for steel to IS:961.	15.4	9.8	14.2	9.0	21.3 **See**	**13.5 also**	20.7 **Clause 3.9**	13.1	19.5	12.4

NOTE:- *In the above, d_1 is the clear distance between flange angles or, where there are no flange angles, between flanges (ignoring fillets); but where tongue plates having a thickness not less than twice the thickness of the web plate are used. d_1 is the depth of the girder between the flanges less the sum of the depth of the tongue plates or eight times the sum of thickness of the tongue plates, whichever is the lesser. t is the web thickness.*

(contd.)

Table II

TABLE II -- (Contd.)

1	2	3	4	5	6	7	8	9	10	11
Parts in Shear Maximum shear stress (Having regard to the distribution of stresses in conformity with the elastic behaviour of the member in flexure) ...	11.1	7.0	10.2	6.5	15.3	9.7	14.9	9.5	14.0	8.9
Average shear stress (on the gross effective sectional area of webs of plate girders, rolled beams, channels, angles, tees) ...	9.4	6.0	8.7	5.5	13.0	8.3	12.7	8.1	12.0	7.6
←	For stiffened webs see clauses 5.8 and 5.10.									
Parts in Bearing On flat surfaces ...	18.9 kg/mm²(12.0 Ton/in.²);				26.0 kg/mm² (16.5 Tor/in.²).					

Description	Mild steel to IS:226 and IS:2062 and carbon steel (class 2) to IS:1875		High tensile steel Grade 58-HTC to IS:961 and Carbon Steel (class 4) to IS:1875	
	Kg/mm²	Ton/in.²	Kg/mm²	Ton/in.²
1	2	3	4	5
Pins				
In shear ...	10.2	6.5	14.2	9.0
In bearing ...	21.3	13.5	29.9	19.0
In bending ...	21.3	13.5	29.9	19.0
For turned and fitted knuckle pins and spheres in bearing: On projected area ...	11.8	7.5	11.8	7.5

Contd....

Table II

Table II (Contd...)

1	2	3	4	5	6	7	8	9
Parts in Bearing								
(a) On power driven shop rivets and turned and fitted bolts ...	23.6	15.0	32.3	20.5	23.6	15.0	32.3	20.5
(b) On power driven field rivets...	...	...	...	...	22.0	14.0	30.7	19.5
(c) On hand driven rivets ...	...	...	...	...	18.9	12.0	...	...
(d) On black bolts ...	15.8	10.0	...	...	...	...	...	...
(e) On precision bolts and semi-precision bolts. ...	22.0	14.0	30.7	19.5	...	...	...	...
			See	Clauses	7.6, 7.7	and 7.8.		
Welds	See I.R.S Welded Bridge Code							

Note:- *For steels to IS:226, IS:2062 and IS:961 a summary of guaranteed yield stress for various thicknesses is given below. For beams and channels, the thickness of the web governs.*

Description	Guaranteed yield stress						
	Mild steel to IS:226 and IS:2062			High tensile steel grade 58-HTC to IS:961			
	26 kg/mm²	24 kg/mm²	23 kg/mm²	36 kg/mm²	35 kg/mm²	33 kg/mm²	30 kg/mm²
1	2	3	4	5	6	7	8
Nominal thickness/ diameter of plates, sections (for example, angles, tees, beams, channels, etc.), and flats. ...	6 mm up to and including 20 mm.	Over 20 mm up to and including 40 mm.	Over 40 mm.	6 mm up to and including 28 mm.	Over 28 mm up to and including 45 mm.	Over 45 mm up to and including 63 mm.	Over 63 mm.
Bars (rounds, square and hexagonal) ...	10 mm up to and including 20 mm.	Over 20 mm.	...	-do-	-do-	-do-	-do-

Table V, VI, VII

Value of K_2 for different values of R_m, are given in the Table VI

TABLE VI – VALUES OF K_2

R_m	1.0	0.9	0.8	0.7	0.6	0.5	0.4	0.3	0.2	0.1	0.0
K_2	0.5	0.4	0.3	0.2	0.1	0.0	-0.2	-0.4	-0.6	-0.8	-1.0

Value of K_1 for different values of R_m are given in the Table V

TABLE V – VALUES OF K_1

R_t	1.0	0.9	0.8	0.7	0.6	0.5	0.4	0.3	0.2	0.1	0.0
K_1	1.0	1.0	1.0	0.9	0.8	0.7	0.6	0.5	0.4	0.3	0.2

TABLE VII – VALUES OF A & B TO BE USED FOR CALCULATING VALUES OF C_a IN kg/ mm²

Where $A = \frac{267730}{(l/r_y)^2}\sqrt{\left[1+\frac{1}{20}\left(\frac{B_c}{r_y D}\right)^2\right]}$

$B = \frac{267730}{(l/r_y)^2}$

NOTE – Where flanges are equal and of constant cross section C_a = A.

l/r_y	$\frac{D}{t_w}$	A: 8	10	12	14	16	18	20	25	30	35	40	50	60	80	100	B
40		251.0	224.6	208.7	198.6	191.8	186.9	183.2	177.6	174.7	172.8	171.5	170.1	169.1	168.4	168.0	167.4
45		212.5	187.7	172.6	162.8	156.2	151.5	147.9	142.5	139.5	137.5	136.4	134.7	134.0	133.2	132.9	132.2
50		184.1	160.6	146.5	137.0	130.7	126.1	122.7	117.3	114.3	112.4	111.2	109.8	109.0	108.2	107.7	107.1
55		162.4	140.3	126.8	117.8	111.7	107.2	103.8	98.7	95.8	93.9	92.6	91.2	90.4	89.5	89.1	88.5
60		145.2	124.4	111.5	103.0	97.0	92.8	89.5	84.4	81.4	79.7	78.4	77.0	76.2	75.4	75.0	74.3
65		131.5	111.8	99.5	91.3	85.7	81.6	78.3	73.2	70.4	68.7	67.4	66.0	65.2	64.4	63.9	63.3
70		120.0	101.4	89.8	81.9	76.4	72.4	69.5	64.4	61.6	59.8	58.6	57.3	56.5	55.8	55.3	54.6
75		110.6	93.1	81.7	74.3	69.0	65.0	62.0	57.3	54.5	52.8	51.7	50.2	49.5	48.7	48.2	47.6
80		102.5	85.7	75.1	67.9	62.7	58.1	56.1	51.5	48.7	46.9	45.8	44.4	43.6	42.9	42.5	41.9
85		95.6	79.5	69.4	62.0	57.0	53.9	51.2	46.6	43.8	42.2	41.1	39.7	38.9	38.1	37.8	37.0
90		89.5	74.3	64.6	58.0	53.1	49.6	46.9	42.5	39.8	38.1	37.0	35.6	34.8	34.0	33.7	33.1
95		84.3	69.6	60.3	54.0	49.3	45.8	43.3	38.9	36.4	34.6	33.5	32.3	31.5	30.7	30.4	29.6
100		79.5	65.5	56.5	50.4	46.0	42.7	40.2	35.9	33.4	31.8	30.7	29.3	28.5	27.9	27.4	26.8
110		71.5	58.7	50.4	44.7	40.6	37.5	35.1	31.0	28.7	27.1	26.0	24.7	23.9	23.1	22.8	22.2
120		65.0	53.2	45.5	40.2	36.4	33.4	31.2	27.2	25.0	23.5	22.4	21.1	20.3	19.7	19.2	18.6
130		59.7	48.7	41.6	36.5	32.9	30.1	28.0	24.2	22.0	20.6	19.5	18.3	17.6	16.9	16.5	15.9
140		55.1	44.9	38.1	33.4	30.1	27.4	25.4	21.9	19.7	18.3	17.3	16.1	15.4	14.8	14.3	13.7

TABLE VIII – ALLOWABLE WORKING STRESS P_{bc}, FOR DIFFERENT VALUES OF CRITICAL STRESS C_s.
(SEE ALSO CLAUSE 3.7 TABLE II)

C_s	P_{bc} for steel to IS:226 & IS:2062	P_{bc} for steel to Grade 58-HTC of IS:961	C_s	P_{bc} for steel to IS:226 & IS:2062	P_{bc} for steel to Grade 58-HTC of IS:961
Kg/mm²	Kg/mm²	Kg/mm²	Ton/in.²	Ton/in.²	Ton/in.²
3	1.5	1.5	2	1.0	1.0
4	2.0	2.0	3	1.5	1.5
5	2.5	2.5	4	2.0	2.0
6	3.0	3.0	5	2.4	2.4
7	3.5	3.5	6	2.8	2.8
8	3.8	3.8	7	3.2	3.2
9	4.2	4.2	8	3.5	3.6
10	4.6	4.6	9	3.9	4.0
12	5.3	5.4	10	4.2	4.4
14	6.0	6.2	12	4.7	5.1
16	6.7	7.0	14	5.1	5.7
18	7.2	7.7	16	5.5	6.3
20	7.6	8.4	17	5.7	6.6
22	8.0	9.0	18	5.9	6.9
24	8.4	9.6	20	6.3	7.5
26	8.8	10.2	22	6.6	8.0
28	9.2	10.8	24	7.0	8.4
30	9.6	11.4	26	7.2	8.8
35	10.5	12.7	28	7.5	9.2
40	11.2	13.7	30	7.7	9.5
45	11.9	14.6	35	8.2	10.1
50	12.4	15.3	40	8.6	10.7
55	12.9	15.9	45	8.9	11.1
60	13.3	16.5	50	9.1	11.5
65	13.6	17.1	55	9.4	11.8
70	13.9	17.4	60	9.6	12.1
75	14.1	17.8	70	9.9	12.6
80	14.4	18.2	80	10.0	13.0
90	14.9	18.8	90	10.0	13.3
100	15.3	19.4	100	10.0	13.6
125	15.8	20.5	110	10.0	13.8
150	15.8	21.2	120	10.0	14.0
200	15.8	22.2	127	10.0	14.1
215	15.8	22.4	135	10.0	14.2

3.10 Allowable Shear Stress in solid Webs of Plate Girders- The calculated average shear stress f_v on the effective sectional area of the web (see clause 4.3.2.3) shall not exceed the value given in TABLE II, clause 3.7

Table VIII

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Questions?
Thank you

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Bridge Evaluation and Rehabilitation (AASHTO)

Date: 9 March 2022
Session 47 – Bridges 05

Nur Yazdani, Ph.D. P.E., F.ASCE, F.ACI, F.SEI
Professor of Civil Engineering
The University of Texas at Arlington, USA
Yazdani@uta.edu

S-8A SEng PRP Training Program

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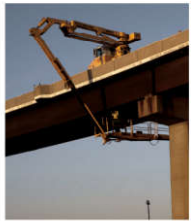
Bridge Inspection (different than Evaluation)



- ✓ Ensure public safety; preserve remaining life through the early detection of deficiencies.

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Bridge Inspection

Eight Types of Bridge Inspections

- ✓Initial (Inventory)
- ✓Routine (Periodic)
- ✓Damage
- ✓In-Depth
- ✓Fracture Critical
- ✓Underwater
- ✓Special (Interim)
- ✓Complex

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Routine Inspection

- Regularly scheduled, performed, and recorded
- Every 24 months for most bridges, and every 48 months for some concrete culverts
- Some bridges may need more frequent inspections when conditions warrant
- For inaccessible bridge undersides, use a vehicle with under-bridge platforms.

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Damage Inspection

✓After collision, fire, flood, earthquake, significant environmental changes, or loss of structural support.

✓ Report includes:

- photos and measurements
- load rating verifying capacity after event
- channel profiles (when applicable)
- repair recommendations
- load restriction recommendations

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Underwater Inspection

✓Performed every 60 months or more frequently if conditions warrant

✓Inspection methods:

Wading: The most basic, requires only a probing rod and wading boots.

Scuba diving: A more detailed examination of substructure conditions.

Surface Supplied Air Diving: Involves sophisticated diving equipment and a surface supplied air system.

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Special Inspection

✓To monitor a known or suspected deficiency

✓Can be recurring or performed on an as-needed basis

✓Special Inspection examples:

- Monitoring the settlement of a substructure every six months
- Monitoring a specific bearing condition every three months until the bridge is replaced.

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Complex Bridge Inspection

- Cable-stayed, suspension, tied-arch, movable, and pontoon bridges as complex.
- Require specialized access equipment and inspection procedures, as well as inspector qualifications.

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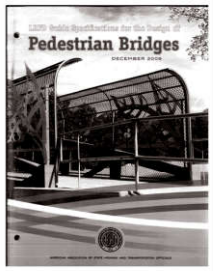
Pedestrian Bridge Inspection

- A routine safety inspection every 48 months.
- Inspection frequency may be increased to 24 or 12 months.
- Generally considered the same as routine inspection.



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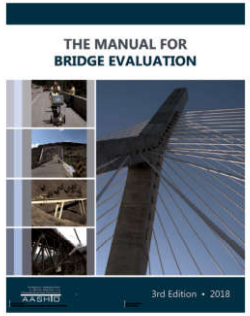
Evaluation of Bridges



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Non-Destructive Load Testing

- ✓Evaluating bridge response under predetermined loadings in the elastic range.
- ✓ Verify both components and system performance under known live loads.
- ✓An alternative to analytical computing.
- ✓Used to verify the performance of bridges compared to design predictions.

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Typical Bridge Problems

Deterioration of the deck

Load Restriction

Cracking of member (steel or concrete)

Section loss

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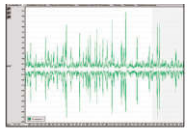
Typical Bridge Problems

Precast deck panel failure



Excessive vibration

Impact Damage

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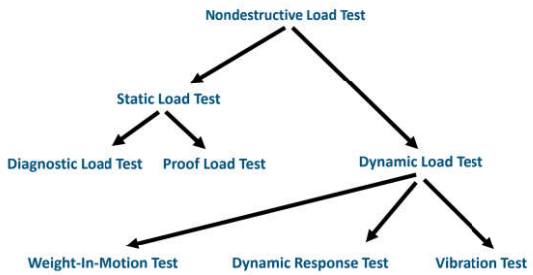
Load Test Classification



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Interesting Factors

- Unintended Composite Action
- Unintended Continuity/Fixity
- Participation of Secondary Members
- Participation of Nonstructural Members
- Portion of Load Carried by Deck

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Diagnostic Load Test Example

- ***Bridge:*** St. Francis Bridge, Dallas, Texas (North Bound lanes).
- ***Year of Construction:*** Around 1960s
- ***Span Number:*** Six spans (symmetric about the centerline of the bridge)
- ***Span Length:*** 30 ft., 60 ft., 70 ft.
- ***Steel girders supporting non-composite RC deck.***
- ***Support Condition:*** 30 ft. span simply supported, 60 ft. and 70 ft. spans continuous girder across the bent.
- ***Chosen for Instrumentation:*** Spans 2 and 3

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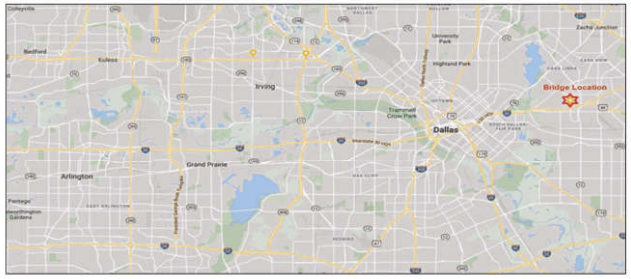
Location of the Bridge



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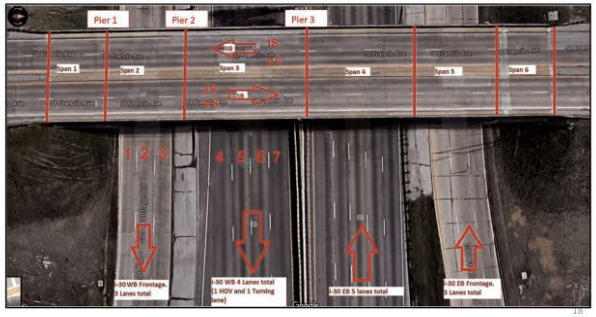
Plan View



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Bridge Profile

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Bridge Impact Video

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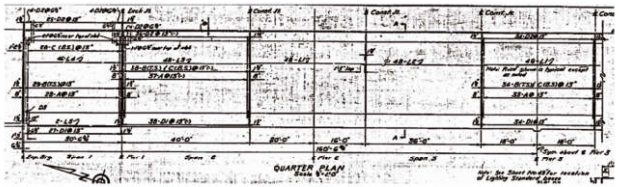
As-Built Plan



Deck rebar layout plan (Quarter plan)

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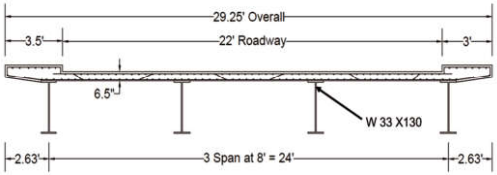
Description of the Bridge



Concrete compressive strength (f'_c)	3 ksi
Yield strength of deck steel	33 ksi
Yield strength of girder steel	40 ksi

Girder Spacing = 8 ft.
Deck thickness = 6.5 in.

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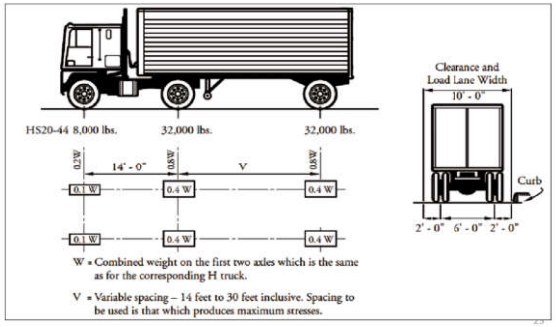
HS 20-44 live load



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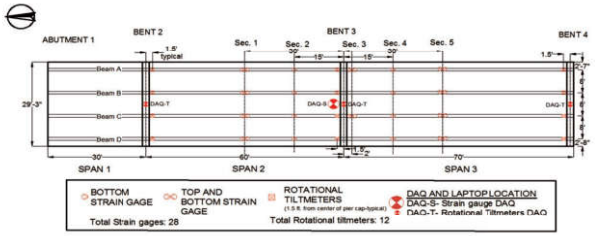
Diagnostic Load Testing Procedure

Instrumentation plan



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Diagnostic Load Testing Procedure

Instrumentation

Foil Stain Gages

Stain Transducer

Rotational Tiltmeters

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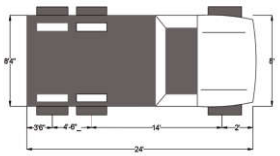
Diagnostic Load Testing Procedure



Live Load (Test Truck)

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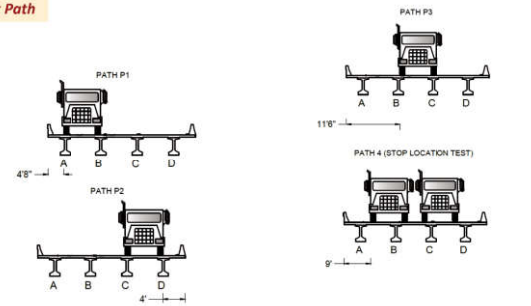
Diagnostic Load Testing Procedure

Test Path



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Diagnostic Load Testing Procedure

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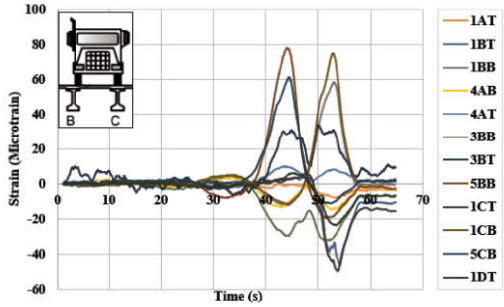
Data Analysis

Modified Strain Data (Noise Free)



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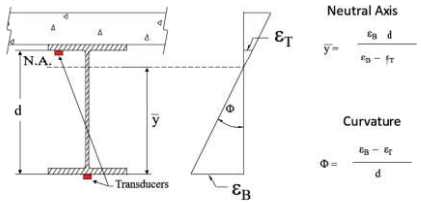
Data Analysis

Experimentally-determined Neutral Axis Location & Curvature



Neutral Axis Calculation

$\bar{y}$ = Neutral axis location from the bottom

d = distance between the top and bottom gauges

ε_B = Strain in bottom gauge ($\mu\varepsilon$)

ε_T = Strain in top gauge ($\mu\varepsilon$)

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Load Distribution Factors

$g_i = \frac{\varepsilon_i}{\sum_{j=1}^{n} \varepsilon_j}$ (One lane loaded)

$g_i = \frac{\varepsilon_i * m}{\sum_{j=1}^{n} \varepsilon_j}$ (more than one lane)

g_i = distribution factor of the ith girder

ε_i = maximum strain response recorded in the ith girder

n = total number of girders

ε_i = strain response of each of the other girders at the same time when the maximum strain was recorded in the ith girder

m = number of lanes loaded

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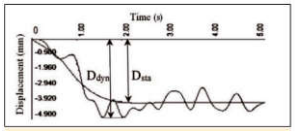
Dynamic Impact Factor

Dynamic Impact Factor or Dynamic Load Allowance, DLA = $\frac{D_{dyn}}{D_{sta}}$

D_{dyn} = the absolute maximum dynamic deflection response at any point

D_{sta} = the maximum static response obtained from the dynamic response



Dynamic and static displacement under a vehicle moving

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Load Rating

Bridge load rating provides a basis for determining the safe load capacity of a bridge.

It is usually expressed as a Rating Factor (RF) or in terms of tonnage for a particular vehicle).

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Why Do We Load Rate

- Different design vehicles used in the past for the design of bridges (e.g., H-15, HS20-44, HS25, HL93..)
- Some bridges have aged, deteriorated or became structurally deficient over time
- For the safety of general public and traffic, load rating may be needed
- Bridges having insufficient load carrying capacity are posted for restricted loads
- May be used to identify the need for bridge rehabilitation or replacement.

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Load Rating Methods

- ✓Load Rating first introduced in the 1970 AASHTO Manual
- ✓Allowable Stress Rating (ASR) - 1970
- ✓ Load Factor Rating (LFR) – 1978
- ✓ Load and Resistance Factor Rating (LRFR) - 2003

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Load and Resistance Factor Rating (LRFR)

RF = Rating Factor

C = Capacity = $\varphi_c * \varphi_s * \varphi * Rn$ (for the strength limit state)

γ_{dc}= factor for components

γ_{dw} = factor for wearing surface

γ_{ll} = Evaluation live load factor

φ_c = Condition factor

φ_s = System factor

φ = LRFD resistance factor

DC = Dead load effect for components

DW = Dead load effect due to wearing surface

LL + IM = Live load + Dynamic load allowance

$$RF = \frac{C - \gamma_{dc}(DC) - \gamma_{dw}(DW)}{\gamma_{ll}(LL + IM)}$$

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Load and Resistance Factor Rating (LRFR)

Limit States and Load Factors for Load Rating (Table 6A.4.2.2.1)

Bridge Type	Limit State*	Dead Load γ_{DC}	Dead Load γ_{DW}	Design Load Inventory γ_{LL}	Design Load Operating γ_{LL}	Legal Load γ_{LL}	Permit Load γ_{LL}
Steel	Strength I	1.25	1.50	1.75	1.35	Tables 6A.4.4.2.3a-1 and 6A.4.4.2.3b-1	—
	Strength II	1.25	1.50	—	—	—	Table 6A.4.5.4.2a-1
	Service II	1.00	1.00	1.30	1.00	1.30	1.00
	Fatigue	0.00	0.00	0.75	—	—	—
Reinforced Concrete	Strength I	1.25	1.50	1.75	1.35	Tables 6A.4.4.2.3a-1 and 6A.4.4.2.3b-1	—
	Strength II	1.25	1.50	—	—	—	Table 6A.4.5.4.2a-1
	Service I	1.00	1.00	—	—	—	1.00
Prestressed Concrete	Strength I	1.25	1.50	1.75	1.35	Tables 6A.4.4.2.3a-1 and 6A.4.4.2.3b-1	—
	Strength II	1.25	1.50	—	—	—	Table 6A.4.5.4.2a-1
	Service III	1.00	1.00	0.80	—	1.00	—
	Service I	1.00	1.00	—	—	—	1.00
Wood	Strength I	1.25	1.50	1.75	1.35	Tables 6A.4.4.2.3a-1 and 6A.4.4.2.3b-1	—
	Strength II	1.25	1.50	—	—	—	Table 6A.4.5.4.2a-1

AASHTO Manual for Bridge Evaluation

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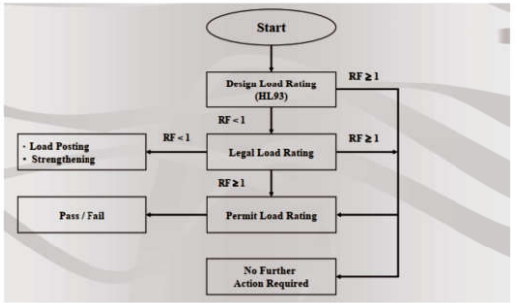
Load and Resistance Factor Rating (LRFR)



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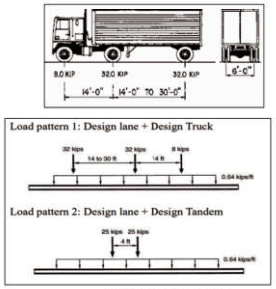
Load and Resistance Factor Rating (LRFR)

Design Load Rating

✓Assesses the performance of existing bridges using the LRFD loading (HL-93) and design standards.



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Load and Resistance Factor Rating (LRFR)

Legal Load Rating

✓Bridges without sufficient capacity under the design-load rating shall be load rated for legal loads to established the need for load posting or strengthening.

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Load and Resistance Factor Rating (LRFR)

Posting Analysis

✓When rating factor, RF, calculated for each legal truck (AASHTO vehicle) is greater than 1, the bridge need not be load posted.

✓When for any legal truck the RF is between 0.3 and 1.0, the following equation should be used for the safe load posting for that vehicle type.

$$\text{Safe load posting} = \frac{W}{0.7}\,[(RF) - 0.3]$$

W= Weight of rating vehicle.

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Load Factor Rating (LFR)

✓The Following equation may be used regardless of the method (ASR and LFR)

$$RF = \frac{C - A_1 * D}{A_2 * (L + I)}$$

Member rating = $W * RF$

W = Weight of nominal truck used in determining the live load effects.

A_1 = Factor for dead loads
A_2 = Factor for live load
C = Capacity of the member
D = Dead loads
I = Impact loads
L = Live loads

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Girder Load Rating (LFR)

Total Dead Load = 1.1 $\frac{k}{ft.}$

Dead load moment = $\frac{1}{14}(1.1) \times 70^2 = 385$ k-ft.; from SAP analysis = 411 k-ft

Capacity C = F_y * Z =33 ksi * 467= 1285 k-ft

I = $\frac{50}{L+125}$ <=0.3, AASHTO Manual for Bridge Evaluation = $\frac{50}{70+125}$
= 0.26

LL DF, g = $\frac{\varepsilon_i}{\sum_1^n \varepsilon_i}$

Live load moment calculations for girders:

- Girder A = 788 *1.26*0.12= 120 k-ft
- Girder B= 788 *1.26*0.61= 606 k-ft
- Girder C = 788 *1.26*0.4= 397 k-ft
- Girder D = 788 *1.26*0.43= 427 k-ft

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Exterior Girder Rating

Exterior girder-A (example)

- $RF = \frac{C-A1*DL}{A2*(LL+IM)} = \frac{1285-1.3*411.29}{2.17*120} = 2.88$ (Inventory level)
- Bridge Member rating = 36*2.88 = 103.6 tons = 207,360 lb.
- $RF = \frac{C-A1*DL}{A2*(LL+IM)} = \frac{1285-1.3*411.29}{1.3*120} = 4.8$ (Operating level)
- Bridge Member rating = 36*4.8 = 28.8 tons = 345,600lb.

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Summary of Girder Rating

Girder	Rating Level	Rating Factor (RF). assuming non-composite	Bridge Member Rating (lb.), assuming non-composite
A	Inventory level	2.88	207,360
	Operating level	4.8	345,600
B	Inventory level	0.57	41,040
	Operating level	0.95	68,400
C	Inventory level	0.87	62,640
	Operating level	1.45	104,400
D	Inventory level	0.81	58,320
	Operating level	1.35	97,200

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Bridge Retrofitting

Retrofitting Options	Reduction in Strain (%)	Reduction in Deflection (%)
Chipping old deck concrete	17 (4 ksi) and 33 (6 ksi)	5 (4 ksi) and 7 (6 Ksi)
CFRP strips underneath the deck	57	40
GFRP joist underneath the deck	58	30
Steel joist underneath the deck	51	17
CFRP application on girder	22	10
Local post-tensioning of steel girder	20	8
Welded shear Stud	21	27
Welded shear stud on steel plate	18	25

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Cost Analysis

Retrofitting Options	Cost ($)
Chipping old deck concrete	$ 433,860
CFRP strips underneath the deck	$ 166,880
GFRP joists underneath the deck	$ 211,584
Steel joists underneath the deck	$ 396,000
CFRP application on girder	$ 105,000
Local post-tensioning of steel girder	$ 309,875
Welded shear Stud	$ 187,900
Welded shear stud on steel plate	$ 280,062

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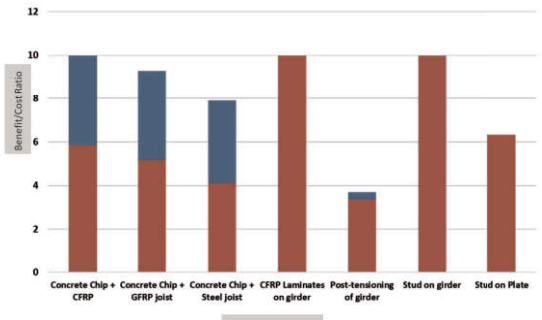
Benefit-Cost Analysis



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Video Clip

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Evaluation of Bridges with Limited or Missing Plans

- Where necessary details, such as reinforcement in a concrete bridge are not available from as-built plans or field measurement, a physical inspection of the bridge by a qualified inspector and evaluation by a qualified engineer may be sufficient to establish an approximate load rating based on rational criteria. Load tests may be helpful in establishing safe load capacity for such structures.
- A concrete bridge or concrete culvert with unknown details need not be posted for restricted loading if it has been carrying normal traffic for an appreciable period and shows no distress. The bridge shall be inspected regularly to verify satisfactory performance.

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Evaluation of Bridges with Limited or Missing Plans

✓Finite element model (via load testing, vibration testing, or combination), and simplified vibration-based assessments are two effective strategies for establishing a starting point for load rating for bridges that has missing parameters.

✓Sometimes Non-Destructive Evaluation (NDE) can help identify some missing properties.

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Why NDE for Concrete Bridges?

- Details of the structure with respect to its design, features and past performance.
- Initial visual inspection of the structure can reveal useful information about areas that need a closer look.
- Many causes for the deterioration, so it may be difficult to pinpoint the type of damage.
- However, all types of damage, whether they be load related, environment related, or hazard related, lead to similar signs of deterioration, such as cracking, scaling, delamination and discoloration.

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NDE vs. Destructive Evaluation

- Non-destructive: without causing any damage while testing
- Semi-destructive: such as removal of a piece of the material for evaluation.
- Destructive: where the material/structure is tested to failure.
- NDE techniques range in sophistication from simple ones to highly complex ones.
- A good NDE method should be:
  - Sensitive to small flaws
  - Reliable
  - Simple
  - Cheap
  - Portable

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Visual Inspection

- Simplest and cheapest.
- Narrow down the critical areas in a structure that need further investigation.
- Direct or Indirect: Telescopes, Borescopes, Magnifying lenses, Real-time video, Camera, Ruler, measuring tape, crack width gage, Light hammer, chipping / scraping tools.
- Limitations of visual observation:
  - Can only detect surface defects; a clean surface is usually necessary
  - Low reliability
  - Good lighting is necessary
  - Quality will vary with inspector vision
  - Most susceptible among all NDT methods to human factors.

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Semi-Simple NDE

- Chain drag
- Sounding: tapping with hammer.

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Semi-Simple NDE

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NDE Laboratory at The Univ. of Texas at Arlington

- Very active with state-of-the-art equipment and know how.
- Several bridge evaluation projects.
- https://blog.uta.edu/yazdani/

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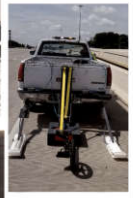
Ground Penetrating Radar (GPR)

A high-end geophysical device that uses radar pulses to image the subsurface.

Antennae with higher frequency (eg. 2.6 GHz) are used for low depth analysis (0-12 in), while lower frequency (eg. 270, 400 MHz) are used for higher depth evaluation (0-18 ft.).

- Truck Mounted Scanner
- Tri wheel Cart Scanner
- Hand Scanner



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Ground Penetrating Radar (GPR)

Concrete Bridge Inspection:

GPR imaging of concrete provides information about rebar location, rebar clear cover, rebar spacing, utility conduits inside the concrete, post-tensioning duct location, voids, water penetration, concrete deterioration, material properties and a variety of other objects inside the concrete.

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Ground Penetrating Radar (GPR)

Drawbacks:

- Variable dialectic constant of different materials for which cores are required for calibration
- Difficult to detect rebars, steel plate or any other object underneath one layer of reinforcements or wire mesh

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GPR Data Collection Video

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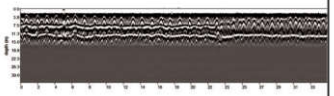
Results from GPR



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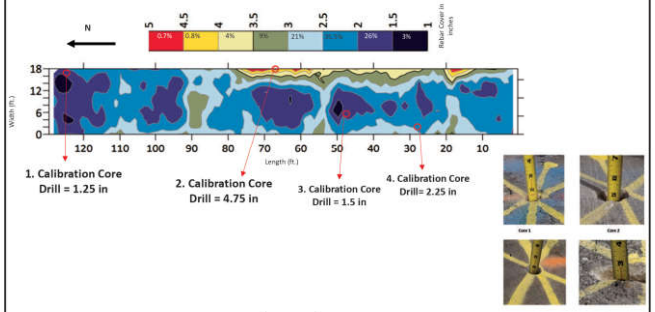
Result from GPR



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Impact Echo (IE)

- Used to determine the delamination condition and thickness of concrete structural members when voids, honeycombs, and cracks are suspected. Creates an impact at a spot on the surface and then records and analyzes the echo. It is a spot scanning device.

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Impact Echo (IE) Device

Current Limitations

- Cannot determine the exact depth of the delamination
- Cannot locate the delamination within 2 inches from the surface

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IE Data Collection Video

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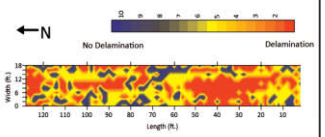
Sample Results from IE



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Ultrasonic Tomograph

Used to determine the delamination condition. Can determine the presence of delamination and the exact depth of the delamination.

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Ultrasonic Tomography Data Collection Video

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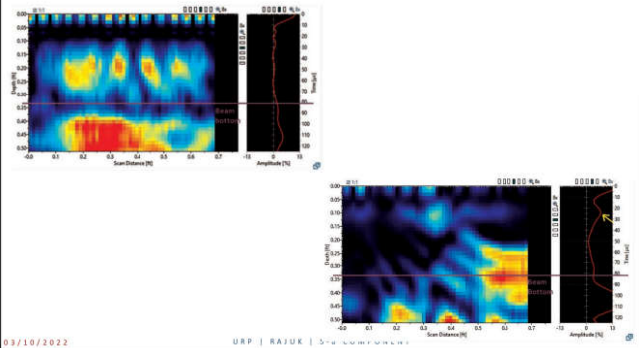
Results from Ultrasonic Tomograph



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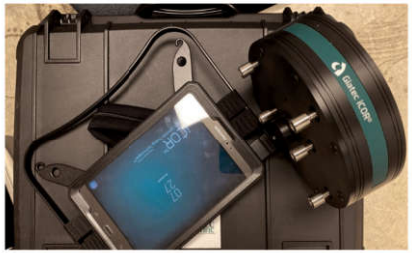
iCOR



For corrosion evaluation and condition assessment of reinforced concrete structures.

Without an electrical connection to the rebar, iCOR can provide corrosion rate mapping and corrosion potential mapping of rebar, in-situ real electrical resistivity of concrete.

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iCOR Data Collection Video

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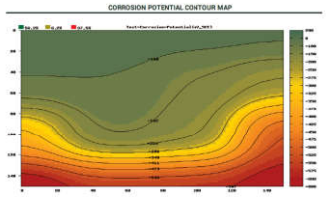
Results from iCOR



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Robotic Scanner (Lidar)

- Combines surveying and 3D scanning to capture high-accuracy measurements. Used in general and topographic surveys, roadway surveys, volumetric surveys, infrastructure surveys, power line inspection, tank calibration, and dimension control. Generates the 3D model which can then be imported into a variety of CAD software.

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Robotic Scanner

Concrete Bridge Evaluation

- For bridge inspection, can be used to monitor the movement of various components with time.

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Trimble Scanner Data Collection Video

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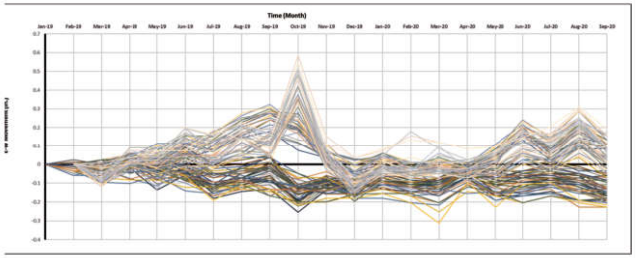
Results from Trimble Scanner (Bridge MSE Wall)



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Infrared Camera

- Can produce a thermograph or temperature profile as high as 1200°F of any surface. For condition monitoring, bridge deck diagnostics, moisture inspection, and energy losses in bridges. Can determine the delamination and bubbles close to the surface of concrete structures or FRP retrofitted concrete structures.

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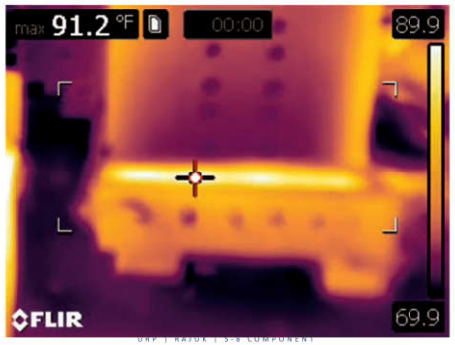
Infrared Camera Data Collection Video



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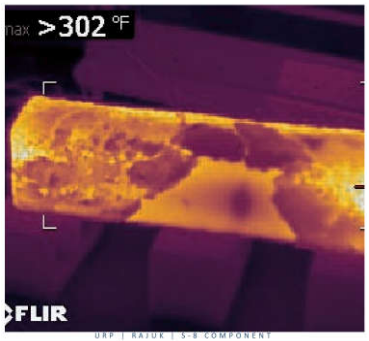
Results from Infrared Camera



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Schmidt/Rebound Hammer

- Used to measure the in-situ compressive strength of concrete.
- Measures the ratio between the rebound velocity and the impact velocity (Q).
- Can convert the rebound number automatically to compressive strengths. The range of measurement is 1450 to 14500 psi.

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Schmidt Hammer Data Collection and Results Video

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Pull-off Adhesion Testers (partially destructive)

- Quality of concrete repair is determined by the adhesive strength between the repair material and the substrate. Most widely used test method to assess bond strength.

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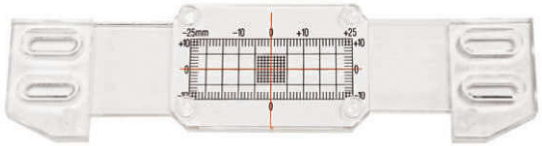
Crack Monitors

- Cracks in concrete structures, roads, buildings, or bridges can signal underlying problems and should be monitored to avoid serious functionality or stability issues.



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Concrete Moisture Testing Equipment

- Can detect moisture present in or migrating through concrete slabs.

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Unmanned Aerial Vehicle (UAV) Bridge Inspection

- UAV or Drones: emerging technology for bridges.
- Promising for bridge inspection due to the logistical challenges to efficiently and effectively visually inspect in challenging locations.
- More suitable as a tool for larger bridges.
- Defects can be identified and viewed with a level of detail equivalent to a close-up photo.
- Right of way and privacy issues are some current limitations.

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UAV Video

- https://www.youtube.com/watch?v=a4QcwQZPwcU

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References

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- ✓ Bridge Inspection Manual, TxDOT, March 2020.
- ✓ Toan, X., and Duc. V. (2015). "Determination of Dynamic Impact Factor for Continuous bridge and Cable-stayed bridge due to vehicle braking force with experimental investigation." 16th Asia Pacific Vibration conference, HUST, Vietnam.

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THANK YOU

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SEISMIC BEHAVIOR AND DESIGN OF HIGHWAY BRIDGES

Date: 10 March 2022
Session 48

Ahmad M. Itani, Ph.D., P.E., S.E.
Professor
University of Nevada, Reno

S-8A SEng PRP Training Program

1

Outline

- Introduction
- Highway Bridge Performance in Recent EQ
- AASHTO Seismic Design Specifications and Performance Objectives
- Seismic Design Methods: Force and Displacement Based
- Steps for Seismic Design

2

S. K. Ghosh Associates LLC.
International Code Council (ICC)

Brief Self Introduction

- PhD, University of Michigan, Ann Arbor in 1991
- Bridge Engineer and Instructor, Caltrans, 10/91-6/94
- Licensed Professional Eng in NV, CA and Structural Eng in CA
- UNR Professor of Civil and Env Eng Department, 7/94-Present
- UNR-CEE Department Chair, 7/2013-6/19
- UNR-Associate Vice President for Research; 7/2019-present

Research Interests

– Behavior and design of highway bridges under extreme events
– Large scale structural experiments

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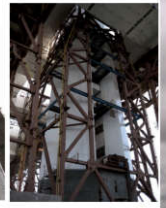
Development of Seismic Code and Specifications



AASHTO-2012

6.16—PROVISIONS FOR SEISMIC DESIGN

6.16.1—General

The provisions of Article 6.16 shall apply only to the design of slab-on-steel-girder bridge superstructures at the *extreme event limit state*.

In addition to the requirements specified herein, minimum support length requirements specified in Article 4.7.4.4 shall also apply.

A clear seismic load path shall be established within the superstructure to transmit the inertia forces to the substructure based on the stiffness characteristics of the concrete deck, cross-frames or diaphragms, and bearings. The flow of the seismic forces shall be accommodated through all affected components and connections of the steel superstructure within the prescribed load path, including, but not limited to, the longitudinal girders, cross-frames or diaphragms, steel-to-steel connections, deck-to-steel interface, bearings, and anchor bolts.

C6.16.1

These Specifications are based on the recent work published by Imai et al. (2010), NCHRP (2002, 2006), MCEER/ATC (2003), Caltrans (2008), AASHTO's *Guide Specifications for LRFD Seismic Bridge Design* (2009), and AISC (2005 and 2010b). The Loma Prieta earthquake of 1989, Petrolia earthquakes of 1992, Northridge earthquake of 1994, and the Hyogoken-Nanbu (Kobe) earthquake of 1995 provided new insights into the behavior of steel details under seismic loads. The Federal Highway Administration, Caltrans, and the American Iron and Steel Institute sponsored a number of research projects that have produced information that is useful for both the design of new steel-girder structures and the retrofitting of existing steel-girder structures.

This new information relates to all facets of seismic engineering, including design spectra, analytical techniques, and design details. Bridge designers working

6

Introduction

- Bridges give the impression that are simple structural systems.
- Historically highway bridges have not performed well.
- Poor performance attributed to design philosophy with lack attention to details.
- Collapse due to structural discontinuity and lack of redundancy.

7

Bridge Response to EQ Loading

- EQ loading is a dynamic force generated at the mass: F= m a= (W/g) a=W (a/g)
- Most of the bridge mass is in the superstructure
- Lot of efforts used to estimate “a”
- Forces will flow through the various components of the bridge causing deformations or movements.

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Bridge Performance in Recent EQ

1. Inadequate seismic load path
2. Unseated spans due to insufficient support lengths
3. R/C Column failure in flexure and shear due to inadequate detailing for ductility
4. Damage at expansion joints and at supports (abutments and bents)

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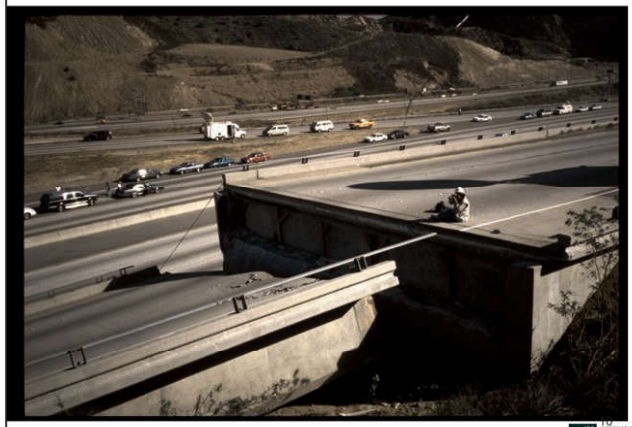

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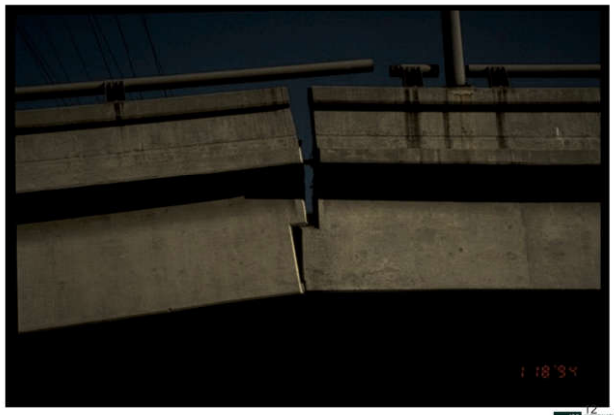

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Lesson Learned

- Unseating of superstructure at expansion joints and seat type abutments
  - Support Length
- Slippage of main reinforcement-Lap splice at column ends
  - Development lengths
- Crushing of concrete in columns
  - Lateral steel for confinement
- Shear failure in columns
  - Lateral steel for shear resistance

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Basic Questions after 1971 EQ

- Is it possible to design a highway bridge to respond elastically due to the maximum credible earthquake?
- Not economically! For a box girder bridge with integral cap and seat type abutment, the column, footing and superstructure size will be large!
- Allowing R/C column to yield during seismic event will lower the seismic forces
- Provide adequate seat width at expansions

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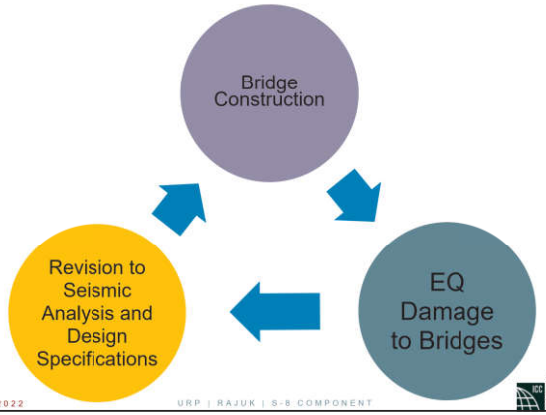
Bridge Seismic Specifications



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AASHTO Seismic Specs Timeline

- Pre 1973-Art 1.2.20, EQ=cW (c=2%-6%)
- 1975 Interims of Standard Specs-modified CT Procedures
- 1981 ATC-6 Seismic Design Guidelines
- 1983 AASHTO Seismic Design Guidelines
- 1990 AASHTO Guide Specs into Division IA
- 1994 First Edition AASHTO LRFD Specs

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AASHTO Seismic Timeline

- 2003 ATC/MCEER-49 Recommended LRFD Guidelines
- 2008 Interims to 4th Edition, updated seismic provisions
- 2009 1st Edition, AASHTO Guide Specs for LRFD Seismic Bridge Design
- 2016 7th AASHTO LRFD Edition updated seismic provisions for Steel Bridges
- 2011 2nd Edition, AASHTO Guide Specs for LRFD Seismic Bridge Design

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Seismic Response of Highway Bridges

- Bridge designer controls the seismic response of bridges
- Seismic Performance Objectives
- Computational Modeling and Analysis
- Structural Design and Details

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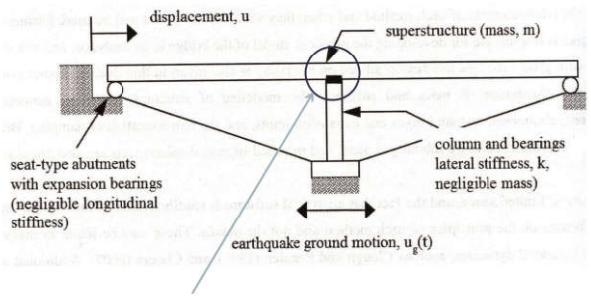
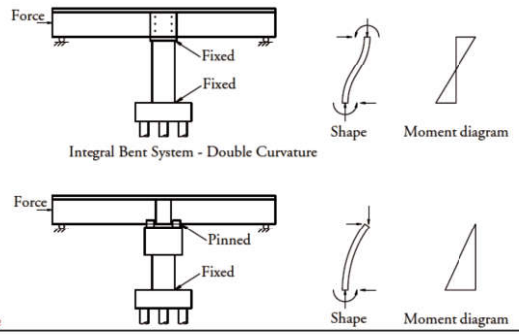
Response to Ground Motion



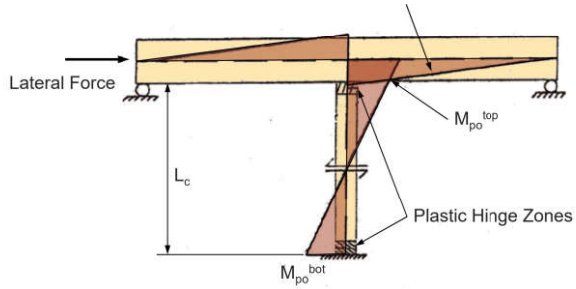
37

Transverse Direction
Integral vs Dropped Cap



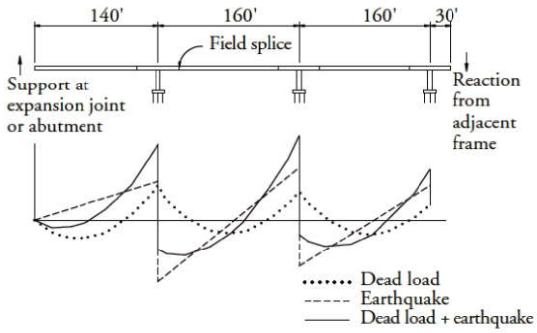
38

Longitudinal Direction



39

Longitudinal Direction



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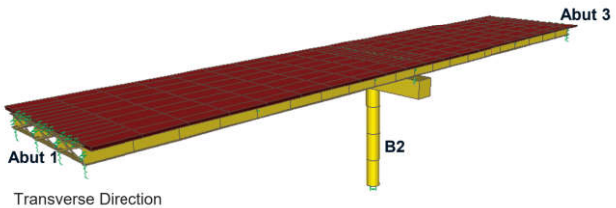
Load Path in Steel Plate Girder Bridges



Transverse Direction

- R/C Deck to Plate Girders through Shear Connectors
  - Shear connectors must be designed for seismic forces, otherwise they will fail and the deck will slide over plate girder
- Cross Frames at supports locations
  - Cross frames must be designed for seismic forces otherwise they will yield and buckle
- Bearings or shear keys

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Basic Seismic Concepts

- Capacity Protected Design
- Damage is limited to selected elements Damage must be controlled and limited to ***ductile*** elements
- Damage should be accessible for inspection and repair.
- All other elements must stay essentially elastic

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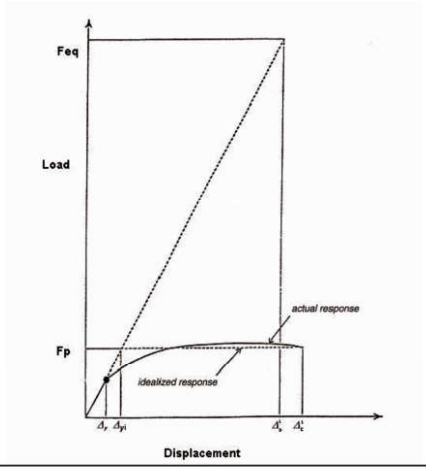
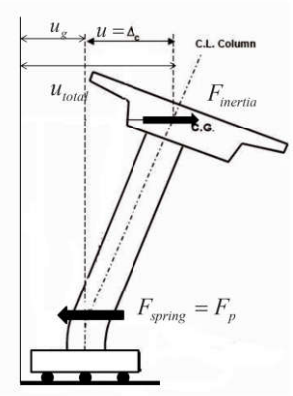
Why Ductile Response

- The internal forces are limited by the yielding elements forming plastic mechanism regardless of the earthquake shaking amplitude.
- However, large earthquakes produce larger deformations, thus proper detailing is required for the yielding element.

43



44



$$m\ddot{u}_{total} + ku = 0$$

$$m(\ddot{u}_g + \ddot{u}) + ku = 0$$

$$F_{inertia} + F_{spring} = 0$$

$$(F_{spring} = V_p) = -F_{inertia}$$

u=relative displacement to ground
u_g=ground displacement

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Effectiveness of Ductile End X-Frame

End X-Frame		V/W	Lateral Displacement (in)
Elastic		1.24	0.21
Ductile	Ductile Angles	0.76	0.80
	BRB	0.86	0.58

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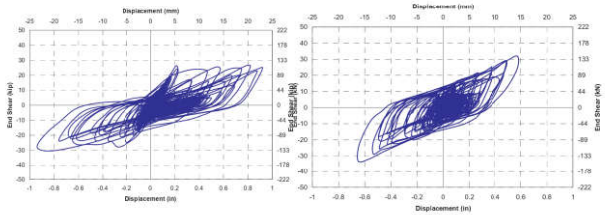
Response of Ductile End X-Frame



Single Angles X-Frame

BRB X-Frame

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AASHTO Seismic Performance Criteria

- Bridges shall be designed for life safety
- Seismic hazard corresponding to a 7% probability of exceedance in 75 years.
- Implies low probability of collapse.
- Significant damage and disruption to services
- Partial or complete replacement may be required.

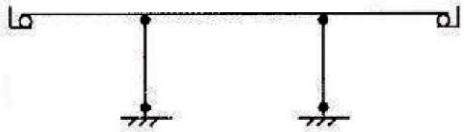
Earthquake Resisting Systems

- Earthquake Resisting System (ERS):
  - Provides reliable and uninterrupted load path for transmitting seismic forces
  - Energy dissipation and/or restraint to control seismic displacements
- Essentials for ERS
  - Simplicity: clear load path
  - Integrity: adequate connection
  - Symmetry: balance in stiffness, mass, strength

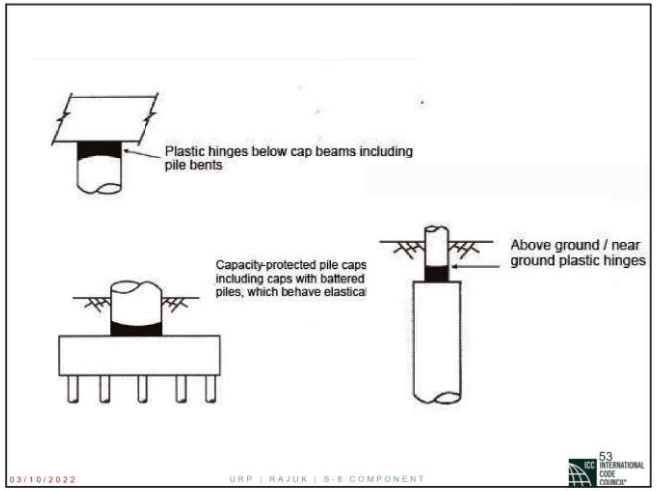
Seismic Design Strategy

- *Type 1*- Ductile Substructure with Essentially Elastic Superstructure
- *Type 2*- Essentially Elastic Substructure with Ductile Superstructure
- *Type 3*- Elastic Superstructure and Substructure with a Fusing Mechanism Between the Two Interface

Longitudinal Response



- Plastic hinges in inspectable locations or elastic design of columns.
- Abutment resistance not required as part of ERS
- Knock-off backwalls permissible


Plastic hinges below cap beams including pile bents
Capacity-protected pile caps including caps with battered piles, which behave elastical
Above ground / near ground plastic hinges
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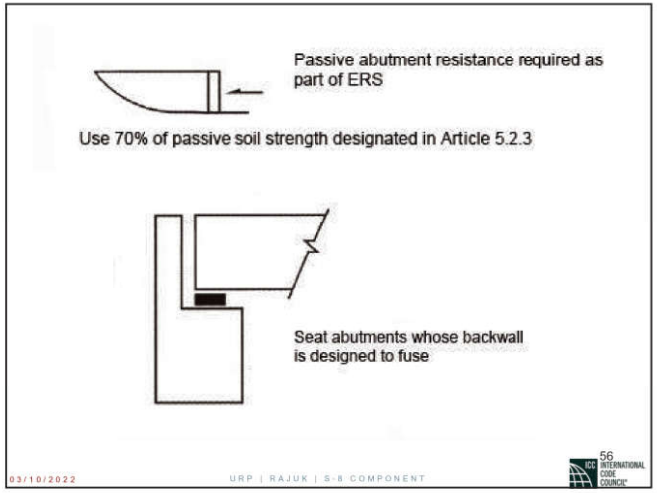
Passive abutment resistance required as part of ERS
Use 70% of passive soil strength designated in Article 5.2.3
Seat abutments whose backwall is designed to fuse
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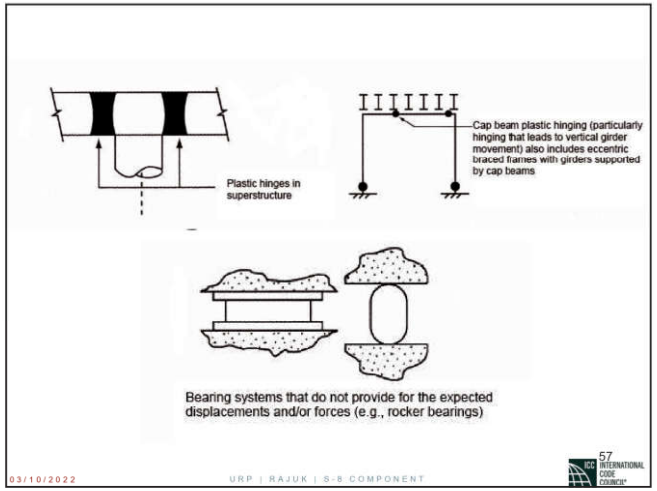
Plastic hinges in superstructure
Cap beam plastic hinging (particularly hinging that leads to vertical girder movement) also includes eccentric braced frames with girders supported by cap beams
Bearing systems that do not provide for the expected displacements and/or forces (e.g., rocker bearings)
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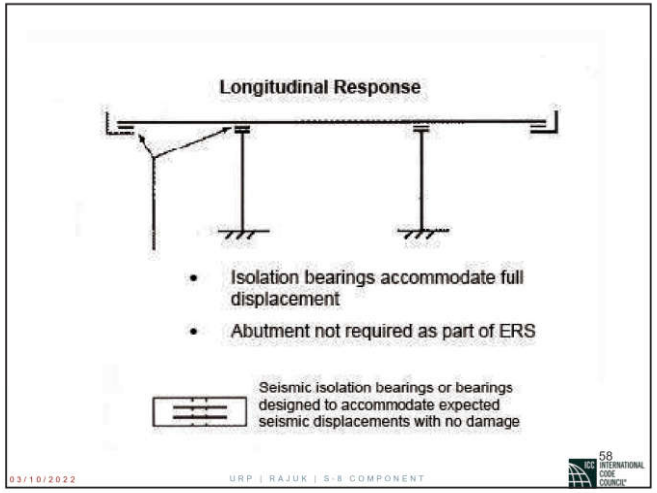
Longitudinal Response
Isolation bearings accommodate full displacement
Abutment not required as part of ERS
Seismic isolation bearings or bearings designed to accommodate expected seismic displacements with no damage
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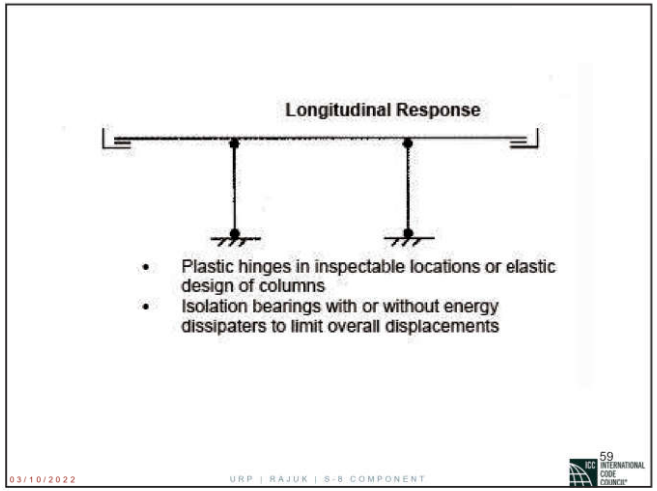
Longitudinal Response
Plastic hinges in inspectable locations or elastic design of columns
Isolation bearings with or without energy dissipaters to limit overall displacements
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New Details
60

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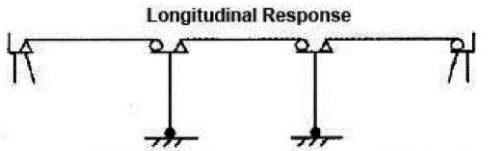
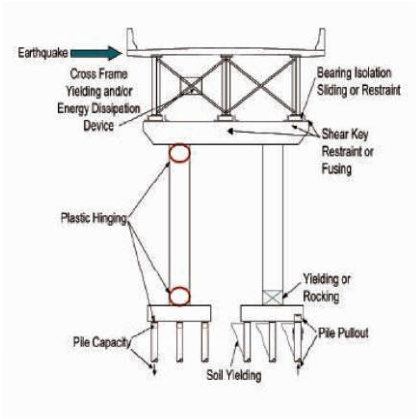


- Multiple simply-supported spans with adequate support lengths
- Plastic hinges in inspectable locations or elastic design of columns

61



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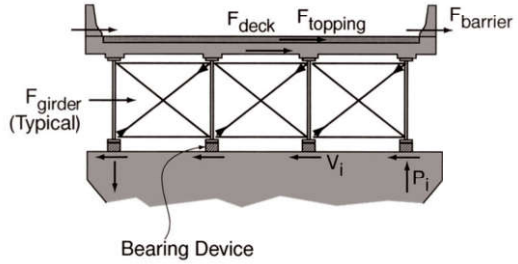
Steel Plate Girder

Pier / Cross Frame / Superstructure



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Steel Plate Girder Bridge during EQ

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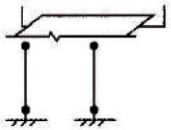
Transverse Response



- Plastic hinges in inspectable locations or elastic design of columns
- Abutment not required in ERS, breakaway shear keys permissible

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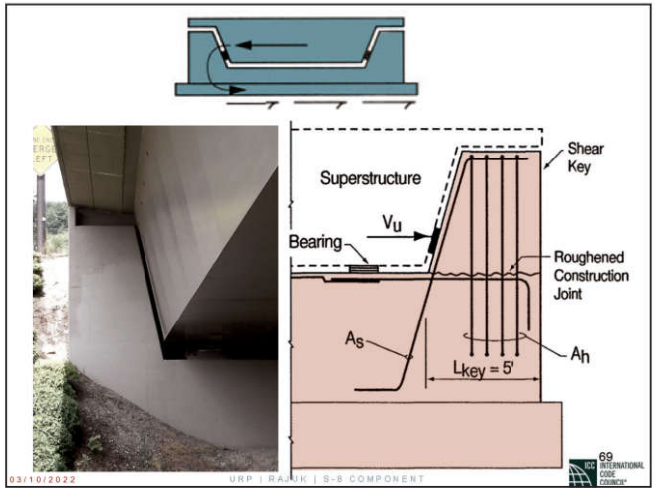

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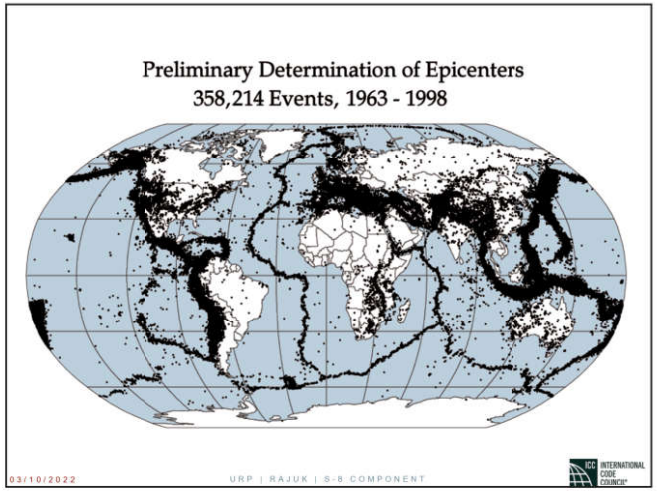
Seismic Hazard

- Seismic Hazard is the hazard associated with potential earthquakes in a particular area.
- The seismic hazard level at a bridge site is determined by the intensity of ground shaking in the rock below the site and the amplification of this motion by the overlaying soil.

70



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AASHTO/USGS Design Hazard Maps

- AASHTO adopted a 1,000 year return period in 2007 for no-collapse design limit state.
- The Hazard Maps show the distribution of earthquake ground shaking that have 7% Probability of Exceedance (PE)
- The goal of Hazard Map is to predict the potential of shaking hazard from future earthquake

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Probability Poisson Law

- Probability of an event is related to the annual frequency of ground motion γ and the exposure time t

$$P=1-e^{(-\gamma t)}$$

$$\gamma=-[\ln(1-P)]/t$$

- Probability of 10% of exccedance in 50 years, P=0.1 and t=50, γ=0.002107 which is 1/475 years
- Probability of 7% of exceedance in 75 years, P=0.07 and t=75, γ=0.001 which is 1/1033 years (1000 years)

AASHTO/USGS Hazard Maps

- Provide a uniform margin against collapse at the design ground motion.
- Ground Motion Hazard are defined in terms of **D**esign **E**arthquake motion.
- Three Sets of Hazard Maps for rock spectral acceleration
  - Peak Acceleration **PA** (PGA)
  - 0.2 sec **S_s Map**
  - 1 sec **S_1 Maps**

AASHTO Seismic Coefficient Spectra

- Shorter period range: deterministic cap on probalistically-derived spectrum
- Longer period range: smoothed and assumed to be inversely proportional to the modal period, T

AASHTO Seismic Coefficient Spectra

- Plot is NOT acceleration response spectrum. Modified for design purposes. AASHTO refers to it as 'seismic coefficient'.
- Similar to AASHTO NEHRP and IBC modify acceleration Spectrum.
- Pre 2007 PGA was assumed to be equal to 0.4 times spectral acceleration at 0.2 sec.

Seismic Parameters

- S_s Mapped spectral response, acceleration at short periods for site Class B (California rock).
- S_1 Mapped spectral response acceleration at 1.0 second period for site Class B (rock).
- Need to modify these values to account for local sites

Effect of Local Soil

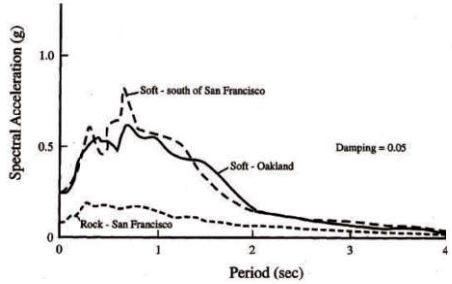

Figure C3.3.2-1. Average spectra recorded during the 1989 Loma Prieta earthquake in San Francisco Bay area at rock sites and soft soil sites (modified after Housner, 1990).

AASHTO Site Class Definition

- Sites are classified by their stiffness as determined by shear wave velocity in the upper 100 ft (30m-V_{s30}), SPT, blow counts and undrained shear strength, s_u
- Site Classes
  1. Class A: Hard Rock
  2. Class B: Rock
  3. Class C: Very dense soil and soft rock
  4. Class D: Stiff Soil
  5. Class E: Soft Soil
  6. Class F: Soils require site-specific evaluations

Site Factors

- Three values approach to account for local soil conditions
- Anchoring spectral acceleration for design spectrum are modified by site response factors F_{pga}, F_a, F_v

$$A_s = F_{pga}\ PGA$$
$$S_{DS} = F_a\ S_S$$
$$S_{D1} = F_v\ S_1$$

Elastic Seismic Response Coefficient

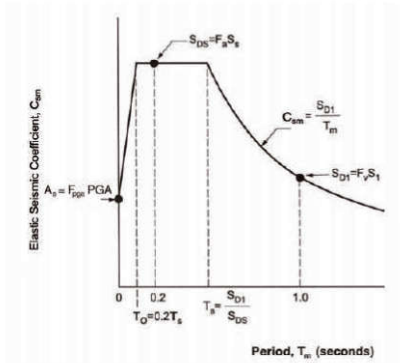

Figure 3.10.4.1-1—Design Response Spectrum

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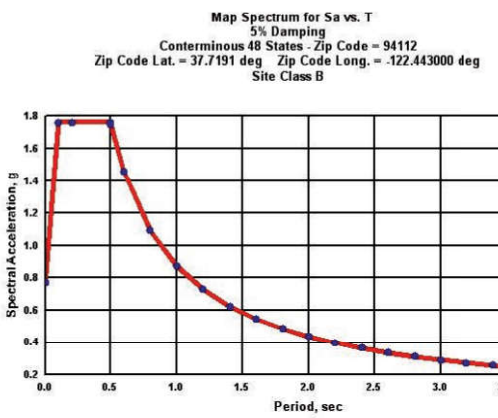

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Seismic Performance Zones

Table 3.10.6-1—Seismic Zones

Acceleration Coefficient, S_{D1}	Seismic Zone
$S_{D1} \le 0.15$	1
$0.15 < S_{D1} \le 0.30$	2
$0.30 < S_{D1} \le 0.50$	3
$0.50 < S_{D1}$	4

- Analysis methods of analysis
- Minimum support lengths
- Column design details
- Foundation and abutment design procedure

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Seismic Design Methodology

- Force Based Method (FBM)
- Displacement Based Method (DBM)
- Both methods seek the same objective: “collapse prevention”
- Both methods relay on ductility and load path to resists EQ larger than the design level.

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Force Based Method

- Used in AASHTO LRFD Specifications.
- Develop seismic demand **forces** for the yielding elements by dividing the elastic seismic forces by appropriate Response Modification Factor (R).
- Perspective details are followed in these elements to ensure adequate ductility.
- Provide adequate load path and displacement for other bridge elements.

Operational Classification

- Bridge Operational Categories
- Critical
  - Operational to all traffic (3% PE-2,500 year return period)
- Essential
  - Operational to emergency vehicles
- Other

Table 3.10.7.1-1—Response Modification Factors—Substructures

Substructure	Operational Category		
	Critical	Essential	Other
Wall-type piers—larger dimension	1.5	1.5	2.0
Reinforced concrete pile bents			
• Vertical piles only	1.5	2.0	3.0
• With batter piles	1.5	1.5	2.0
Single columns	1.5	2.0	3.0
Steel or composite steel and concrete pile bents			
• Vertical pile only	1.5	3.5	5.0
• With batter piles	1.5	2.0	3.0
Multiple column bents	1.5	3.5	5.0

Table 3.10.7.1-2 Response Modification Factors—Connections

Connection	All Operational Categories
Superstructure to abutment	0.8
Expansion joints within a span of the superstructure	0.8
Columns, piers, or pile bents to cap beam or superstructure	1.0
Columns or piers to foundations	1.0

Displacement Based Method

- Used in AASHTO Guide Specifications for LRFD Seismic Design.
- Proportion the yielding system to ensure the demand **displacement** is less than displacement capacity of the yielding element-adjust element detailing
- Provide adequate strength for other elements based on the ultimate capacity of the yielded element.

Design Methods Comparison

- FBM, R values are prescribed and used to size yielding element. No verification of the actual displacement or ductility demand is made and prescribed detailing is used to ensure adequate ductility
- DBM, displacement capacity of each yielding element is evaluated and compared with the displacement demand on the elements. Detailing can be customized to improve required ductility.

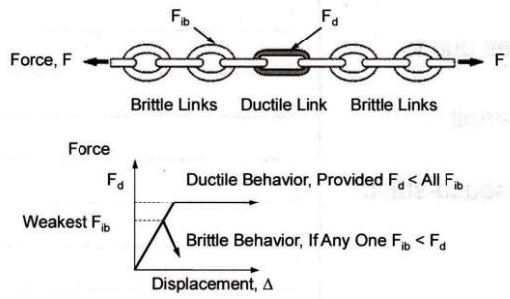
Elements of Capacity Design

- Select the location where damage is preferred
- Design these elements to be ductile
- Protect all other elements from failure

- Designer control the behavior and response of the bridge!

S. K. Ghosh Associates LLC.
International Code Council (ICC)

Capacity Protection Concept



What Elements Need Capacity Protection

- Column Shear
- Column Joint at the footing and bent cap
- Footing
- Superstructure

Steps for Seismic Design

1. Determine Seismic Input
   a) Seismic Hazard
   b) Site Classification
   c) Response Spectra
2. Establish Design procedures
   a) Seismic Zone
   b) Importance
   c) Analysis Procedure
   d) Foundation Modeling

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3. Identify Earthquake Resisting System and Global Design Strategy
   a) Where yielding will be accepted
   b) What system of deformation will be used?
   c) Participation of abutments
4. Demand Analysis
   a) Bridge modeling
   b) Analysis method (linear, nonlinear, etc)
5. Design and Check Earthquake Resisting Elements
   a) FBM or DBM
   b) Detail ductile elements

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6. Capacity Protect Elements
   a) Shear in ductile earthquake resisting elements
   b) Bent Caps
   c) Foundations and Abutments
   d) Superstructures, connections, bearings, support lengths.

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Seismic Design of Highway Bridges- Accelerated Bridge Construction (ABC) and Innovative Materials

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http://wolfweb.unr.edu/homepage/saiidi/
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Director, Center for Advanced Technology in Bridges and Infrastructure
Co-Director, ABC-UTC

NCHRP

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NEES

1

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Sina Zolfaghary, PhD Student
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2

TOPICS

1. **Next generation of bridge columns for accelerated bridge construction (ABC)- Caltrans**
2. **Seismic design of ABC column connections in the 2020 proposed AASHTO guidelines- NCHRP**
3. **Seismic resistance of columns with CFRP tendons- USDOT**
4. **Deconstructible bridges w/ advanced materials- NSF**

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1- Next generation of bridge columns for ABC

- What is "Next generation?"
- Prefabricated columns, **including connections,** that
  - Meet seismic performance requirements
  - Constructible
  - Do not introduce unusual durability, maintenance, or inspectability issues
- Resilient- minimum damage; serviceable after EQ
- Conform to current ban on splices in plastic hinges

Caltrans

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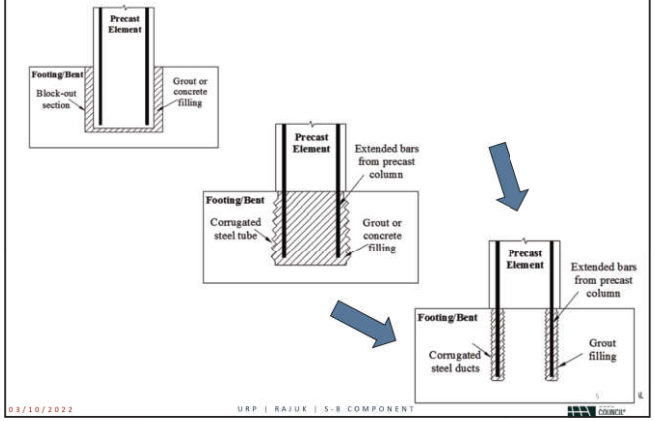
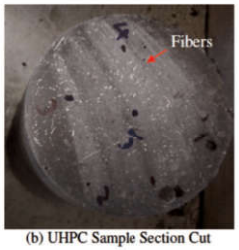
OPTIONS W/ NO-COUPLERS



5

Ultra High Performance Concrete, UHPC

(a) Steel Fibers



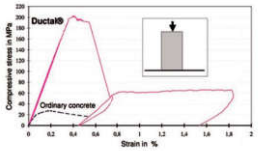
(b) UHPC Sample Section Cut

6

UHPC Materials

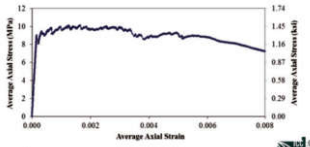
- Ultra-high Performance Concrete (UHPC)
  - Significantly higher compressive and tensile ductility

} Compared to Conventional Concrete



Courtesy: Lafarge.com

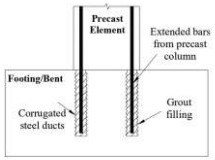
Compression



Tension

7

ABC Connections w/ UHPC



Grouted Ducts w/ UHPC

Two parts:

1- Bond strength studies

2- Implementation in large-scale columns tested under cyclic loading

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UHPC-Filled Duct Connections

- 14 Pullout Tests
- Variables:
  - Embedment Length: $3d_b$, $5d_b$, $8d_b$, and $12d_b$
  - Bar Size: #8 (Ø25 mm) and #11 (Ø36 mm)
  - Duct Size: nominal 3 in. (75 mm), 4 in. (100 mm) and 5 in. (125 mm)
  - Bundled Bar: a pair of bars in a connection
  - Multiple Ducts: two ducts in a connection

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UHPC-Filled Duct Connections

Connection Bond performance: *Test Results*

Duct Pullout

SP4- Group I
Bundled #8 Bars
Emb. Length: $8d_b$

SP11- Group I
Single #11 Bar
Emb. Length: $8d_b$

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UHPC-Filled Duct Connections

Bond performance: *Test Results*

- Bond strength of bar in UHPC was eight times higher than that in conventional concrete
- Design Development Length:

$$L_d = \max\left(L_{d,duct},\ L_{d,bar}\right)$$

$$L_{d,duct} = \frac{d_b^2 \cdot f_s}{27 d_d \cdot \sqrt{f'_c}}$$

$$L_{d,bar} = \frac{d_b \cdot f_s}{120\sqrt{f'_{UHPC}}} \qquad (psi,\ in)$$

Compared to conventional connections designed according to:

- ACI 318
- AASHTO LRFD Grout-Filled Ducts

At least 50% Reduction in Required Embedment Length

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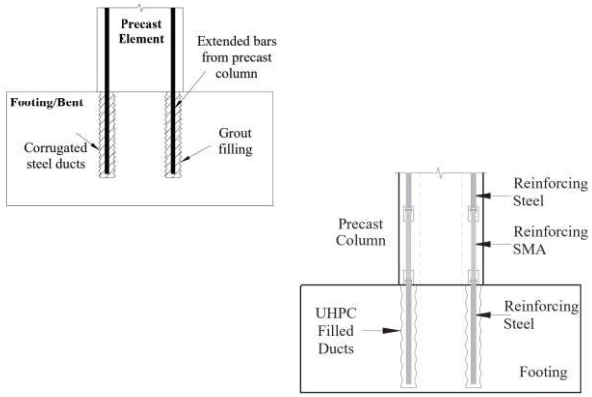
UHPC-Filled Duct Column Connections

- **Two Column Models**
  - Conventional Materials in Plastic Hinge ("PNC")
  - SMA-ECC in Plastic Hinge ("HCS")
- **Connection to Footing**
  - UHPC-Filled Duct Connections
- **Column Geometry**
  - Half-Scale; Hollow; Filled SCC after connectin
  - Height: 9 *ft* (2.74 *m*)
  - Diameter: 24 *in*. (610 *mm*)
  - 11-#8 (Ø25 *mm*) Longitudinal Bars (ρ_l=1.92%
  - Spiral, ρ_s=1.03%
  - Axial Load Index: 10% (200-kip axial load on specimens)

12

Column ABC Connections w/ UHPC Grouted Ducts



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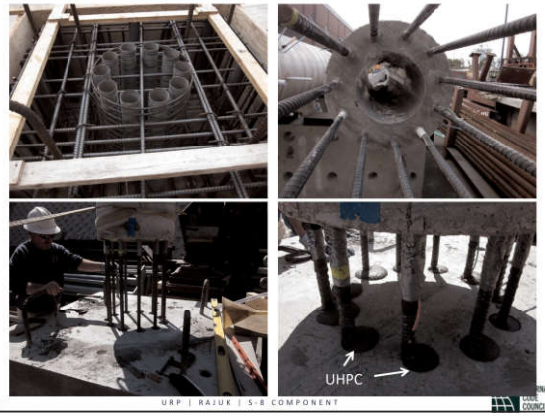
UHPC-Filled Duct Columns Connections - Construction



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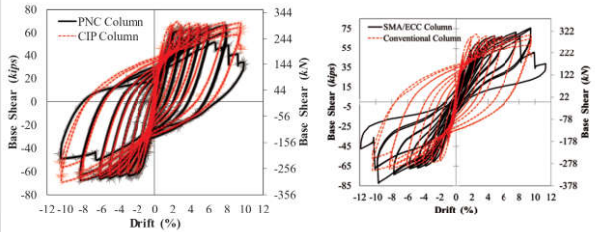
UHPC-Filled Duct Column Connections - Performance



9% Drift Capacity

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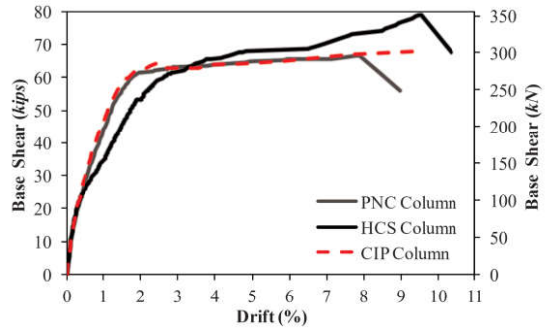
UHPC-Filled Duct Column Connections- Test Results

PNC, HCS, and CIP Test Average Envelopes



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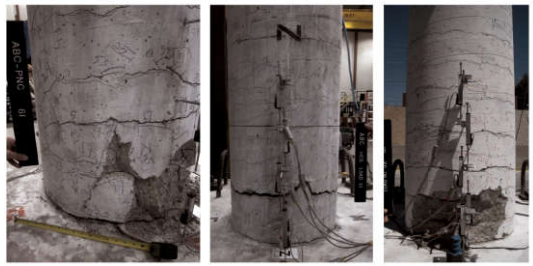
UHPC-Filled Duct Connections

@ 5% Drift Ratio

Columns w/ UHPC-Filled Duct Connections



PNC HCS CIP

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TOPIC 2-

Seismic design of ABC column connections in the 2020 proposed AASHTO guidelines- NCHRP

- PI: M. Saiid Saiidi
- Co-PIs: A. Itani, D. Sanders, UNR; M. Tazarv, S. Dakota State U.; T. Murphy, Modjeski & Masters; M. Reno, Quincy

NCHRP University of Nevada, Reno South Dakota State University QUINCY ENGINEERING MODJESKI—MASTERS

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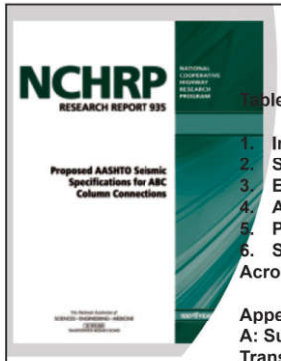

Published April 2020

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2. State-of-the-Art Literature Review
3. Experimental Programs
4. Analytical Programs
5. Proposed Specifications and Examples
6. Summary and Conclusions
Acronyms

Appendices
A: Survey of State Departments of Transportation
B: Proposed AASHTO/ASTM Standard Test Method for Bar Couplers
C: Proposed AASHTO Specifications ... ABC Columns
D: Detailed Design Examples

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Objectives of NCHRP 12-105

- Develop proposed AASHTO displacement-based design and construction specifications for ABC column connections in moderate and high seismic regions.
- Include mechanical bar couplers, grouted ducts, pocket, and socket connections.

Include analytical and experimental studies

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Task 13.1

Seismic Design of Columns with ABC Connections

Task 13.2

AASHTO Standard Test Methods for Seismic Couplers

Task 13.3 – Examples

Design Examples for ABC Column Seismic Connections

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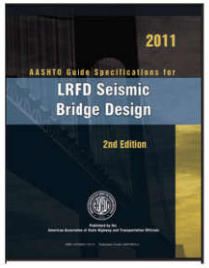
Guidelines as Appendix to "SGS"

- Additional requirements for seismic connections using ABC details
- Does not supplant the requirements of the Seismic Guide Specifications – **adds to them**
- Three main sets of provisions
  - Bar couplers
  - Grouted duct connections
  - Pocket/socket connection details



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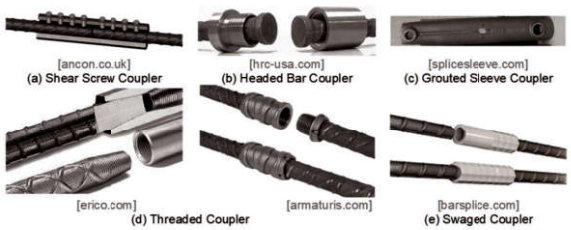
Mechanical Bar Splices (Couplers)

Current Bridge Seismic Design: Couplers are banned in plastic hinges in moderate and high seismic zones



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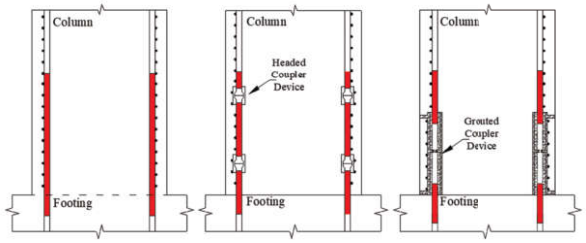
Reason?



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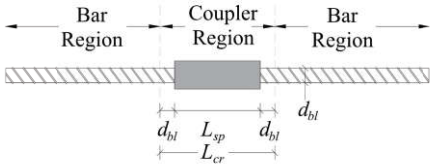
Couplers

- **Coupler length definition**:

$$L_{cr} = L_{sp} + 2d_{bl}$$



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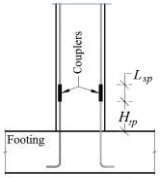
Plastic Hinge Length

- Presence of couplers accounted for by using altered plastic hinge length

$$L_p^{sp} = L_p - \left(1 - \frac{H_{sp}}{L_p}\right)\beta L_{sp} \le L_p$$



- Strain adjustment for section analysis:

$$\frac{\varepsilon_{sp}}{\varepsilon_s} = \frac{L_{cr} - \beta L_{sp}}{L_{cr}}$$

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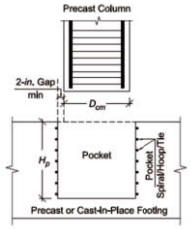
Pocket/Socket Connections

- Pocket filler material requirements
  - Concrete, SCC, or grout
  - Footings and fully precast columns require grout
- Corrugated pipe material requirements
- Gaps and surface preparation requirements
- Pocket depth requirement:

$$H_p \ge D_{cm}$$
$$H_p \ge 0.79 d_{bl} \frac{f_{ye}}{\sqrt{f'_c}}$$
$$H_p \ge 24 d_{bl}$$



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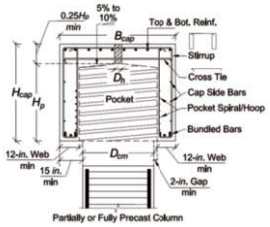
Bent Cap Details

- **Minimum cap depth**
- **Minimum cap width**
- **Vent hole size and top surface slope**
- **Pocket reinforcing requirements**
- **Bent cap longitudinal reinforcing**



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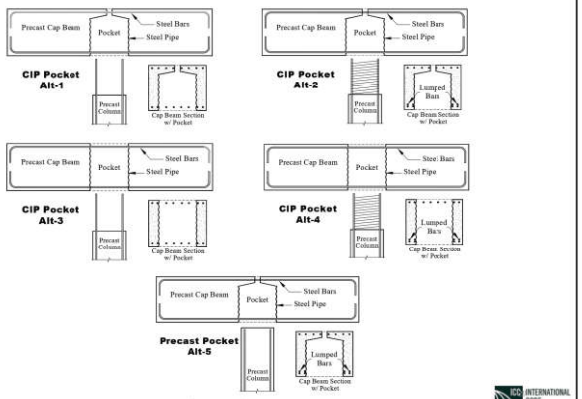
Detailing Alternatives for Pocket/Socket Connections



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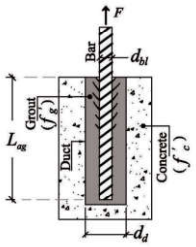
Grouted Ducts

- Requirements for duct material
- Duct size and spacing requirements
- Requirements for length of embedment in grouted ducts
  - Bar-to-grout bond
  - Duct-to-concrete bond

L_{ag} = Max

$$l_{ab} = \frac{0.68 d_{bl} f_{ye}}{\sqrt{f'_g}}$$

$$l_{ad} = \frac{2.25 d^2_{bl} f_{ye}}{d_d \sqrt{f'_c}}$$



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1/3rd Scale Concrete Bridge Model w/ Grouted Duct Connections- 200% Design- Overview

Bridge System Seismic Research for Accelerated Bridge Construction (ABC) - Calt-Bridge 1
Bridge Overview SW

20/9/2017
Run 8 – 1.070 x Sylmar
(PGATRANS=0.66g PGALONG=0.98g)

PI: Dr. M. 'Saiid' Saiidi
Co-PI: Dr. Ahmad Itani
Graduate Assistant: José Benjumea, Ph.D. Student

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1/3rd Scale Concrete Bridge Model w/ Grouted Duct Connections- 200% Design- Closeup

Bridge System Seismic Research for Accelerated Bridge Construction (ABC) - Calt-Bridge 1
Bent North View

20/9/2017
Run 8 – 1.070 x Sylmar
(PGATRANS=0.66g PGALONG=0.98g)

PI: Dr. M. 'Saiid' Saiidi
Co-PI: Dr. Ahmad Itani
Graduate Assistant: José Benjumea, Ph.D. Student

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1/3rd Scale Steel Bridge Model w/ Grouted Duct & Base Pocket Connections and Pocket- 200% Design- Closeup

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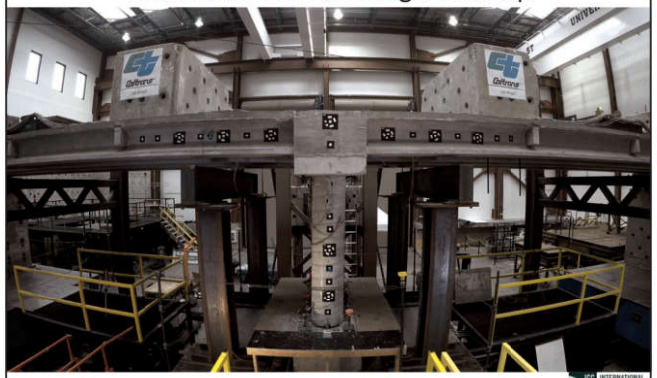
1/3rd Scale Concrete Bridge Model w/ Pocket Connections- 200% Design- Closeup



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3- Seismic Evaluation of a Precast PT/UHPC Bridge Column with Pocket Connection

- Evaluate seismic response of a precast square column connected to a precast footing using a pocket connection subjected to earthquakes simulated on a shake table.
- Determine the effectiveness of ultra-high performance concrete (UHPC) in the plastic hinge zone in minimizing column damage,
- Investigate effectiveness of unbonded carbon fiber reinforced polymer (CFRP) tendons in recentering columns under strong earthquakes.

University of Nevada, Reno
Statewide • Worldwide

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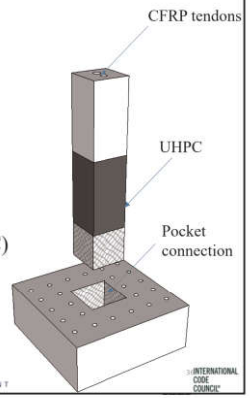
Novel Concepts

- Precast column and footing with pocket connection
- Post-tensioning using unbonded Carbon Fiber Reinforced Polymer (CFRP) tendons
- Ultra-High Performance Concrete (UHPC) in plastic hinge



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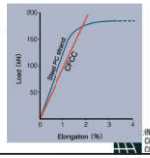
CFRP Tendons

CFRP main features:
- High tensile strength (~300 ksi, effective area=0.88 in^2)
- High tensile modulus (~21,030 ksi)
- Noncorrosive

(Tokyo Rope)



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Precast Column

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Pocket Connection

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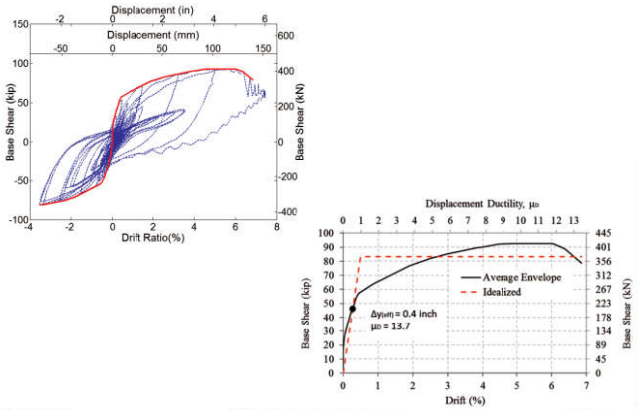
Hysteresis and Pushover Reponse



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Damage

Run #6
Max. Drift: 6.82%

UHPC Plastic Hinge

Conventional Concrete Plastic Hinge

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4- Deconstructible bridges w/ advanced materials

Objectives:

Develop bridge columns that
1- Withstand strong earthquakes with no or minor damage so they are useable after earthquakes.
2- Can be disassembled and reused.

5% of CO_2 emission in the world is from cement factories.

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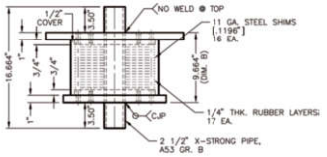
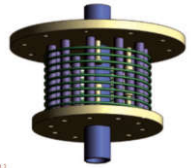
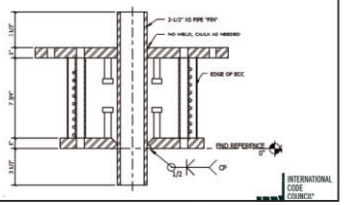
Plastic Hinge Elements







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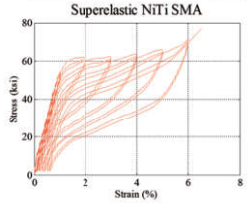
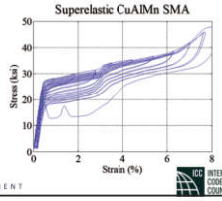
Longitudinal Reinforcement in PH Shape Memory Alloys (SMA)

Ni-Ti



Cu-Al-MN



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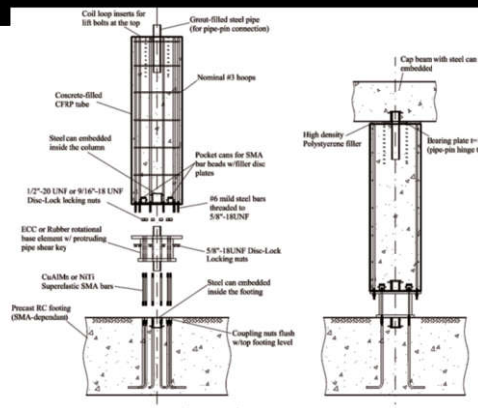
Capacity-protected column outside PH



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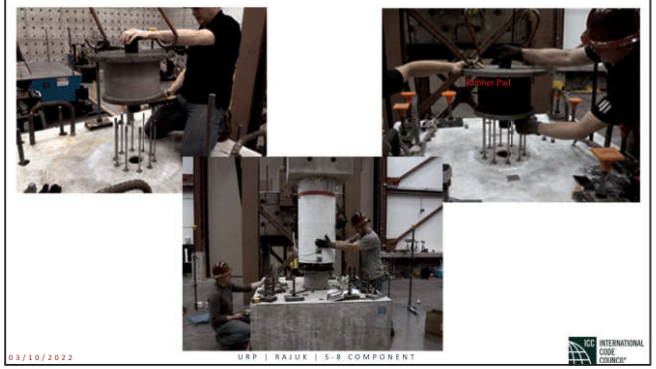
ECC; NiTi; Copper Based SMA; Rubber; CFRP Shell
(Patent Filed)



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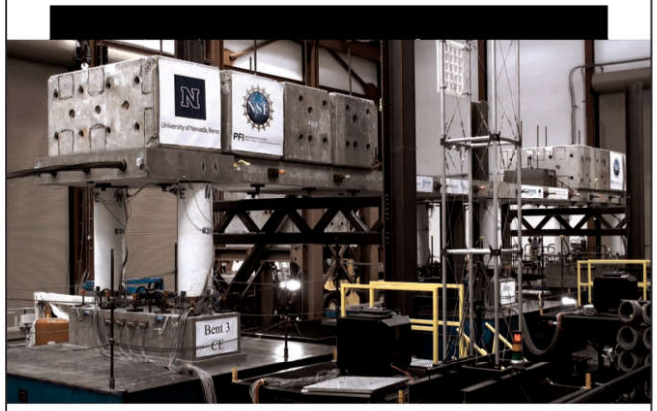

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Reassembled Bridge

Overview - Shake table test of a reassembled precast modular 2-span bridge model with innovative materials (Bridge #2)

2/6/2015
Run 7 - 1.225 x Rinaldi (PGA=1.2 g)

PI: Dr. M. 'Saiid' Saiidi
Graduate Assistant: Sebastian Varela, PhD student
University of Nevada, Reno

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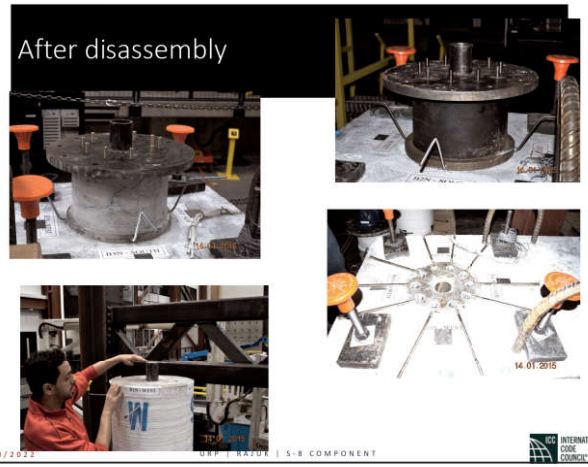
After disassembly



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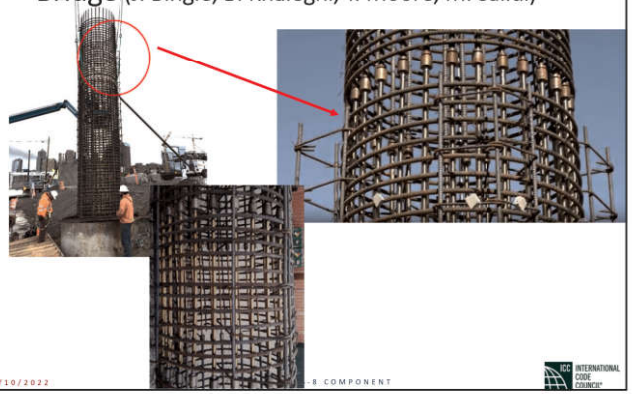
Implementation of SMA/ECC in Showcase Bridge (J. Bingle, B. Khaleghi, T. Moore, M. Saiidi)



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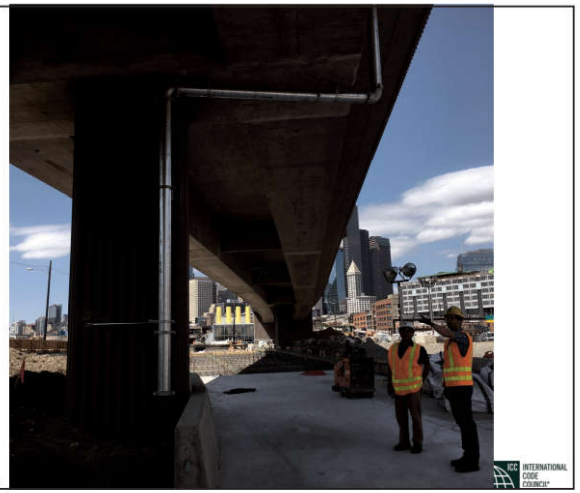

03/10/2022

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Message

- Recent research enabled the development of ABC seismic design connection guidelines with confidence.
- The proposed guidelines expand the current SGS provisions to cover ABC as an appendix.
- Precast column connections without using couplers are feasible and being implemented.
- Superelastic SMAs are being considered as an alternative to improve seismic performance of structures.
- Unbonded CFRP tendons present a good alternative to steel tendons to reduce permanent drifts without concern about corrosion.

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