CENG-5132
FUNDAMENTALS OF BRIDGE DESIGN

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Reference

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3. Bridge Management, M. J. Ryall, 1st edition, 2001
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Chapter 1

Introduction

1. Introduction

Background

- A bridge is a structure which is built over obstacles such as rivers, valley, gorges ,… exist and its purpose is to provide crossing over that obstacle.
- Bridges are constructed primarily to carry communication routes over an obstacle.
- Bridges are key elements of the road network by desirable quality of their strategic locations and important component in the transportation infrastructure of nation.
- Bridges provide a critical link for transportation systems and economic growth.

1. Introduction

Background

Bridge Engineering covers :

- Planning, design (preliminary design phase and Final design phase), construction and operation of structures that carry facilities for the movement of humans, animals, or materials over natural or created obstacles.

- The History of development of bridges is closely linked with the history of human civilization.

1. Introduction

Historical Development

The History of development of bridges is closely linked with the history of human civilization.

- The world's first modern cantilever bridge (129 m) was built in 1867 across the river Main at Hassfurt, Germany.
- The world's most famous cantilever bridge is the Firth of Forth bridge in Scotland, spans of 521 m, built in 1889.
- The world's longest span cantilever bridge was built in 1917 at Quebec, over the St. Lawrence river, with a main span of 549 m.

- Ethiopia is one of a mountainous country in Africa.
  - Topographic condition: rough terrain, deep gorges, rivers, …..

- Roads without constructing crossways is difficult. Thus, Bridges are vital structures required during roads construction.

1. Introduction

Historical Development of Bridge in Ethiopia

- During the period following Fasiladas (after 1667) it is said that many bridges were constructed in Gonder and Lake Tana area.
- During the Italians occupation (1935-1941) numerous bridges was initiated and constructed.

Number of bridges were constructed by Italians during 1935-1941 :

- Assab-Serdo-Dessie road –80 major bridges were constructed
- Asmara-Addis road up to Debre Sina –24 major bridges were constructed
- Tekezie to Tanna 52 major bridges were constructed
- Addis to Mille-50 major bridges were constructed

Bridge Construction in Ethiopia

1. On Blue Nile near Alata, Almeida Bridge, thick log is placed. (The 1st Bridge)

2. The first stone bridge constructed in Ethiopia by the Portuguese on path leading to the Blue Nile Falls (Portuguese Bridge was constructed around 1625)

3. Abay No.4 bridge was constructed in 1959 (13 span RC girder bridge – located in Bahir dar)

4. For instance in Addis Ababa, Kebena and Ras Mekonnen (1902) bridges were constructed.

The first stone bridge constructed in Ethiopia by the Portuguese (on path leading to the Blue Nile Falls)

Bridge between Gonder and Gojam- Portuguese Bridge (was built in around 1625)

Bridge Construction in Ethiopia cont'd

- 1935-1941 – Many Bridges were constructed during Italian's occupation and serving for more than 7 decades

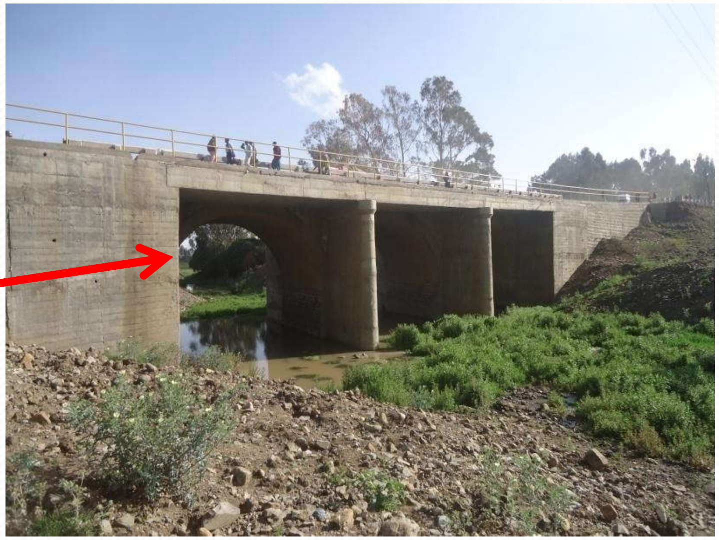
- During the last 30 years (many bridges were constructed)
- Slab ,T- and Box Girder bridges made using RC, are the most commonly used bridges in Ethiopia.(almost 85%)

Jewha Bridge is constructed in 1938. The Bridge is located at 237.7Km from A.A in Robit- Ataye road segment.

Bridge Construction in Ethiopia cont'd

- Most of the bridges so far constructed in the country have now been widened and replaced
  1. due to their roadway widths were too restrictive for the safety of accommodating modern traffic.
  2. due to aging, currently they are not capable & suitable to the current traffic flow and heavy loads, reduction in load carrying capacity and accidental impact.

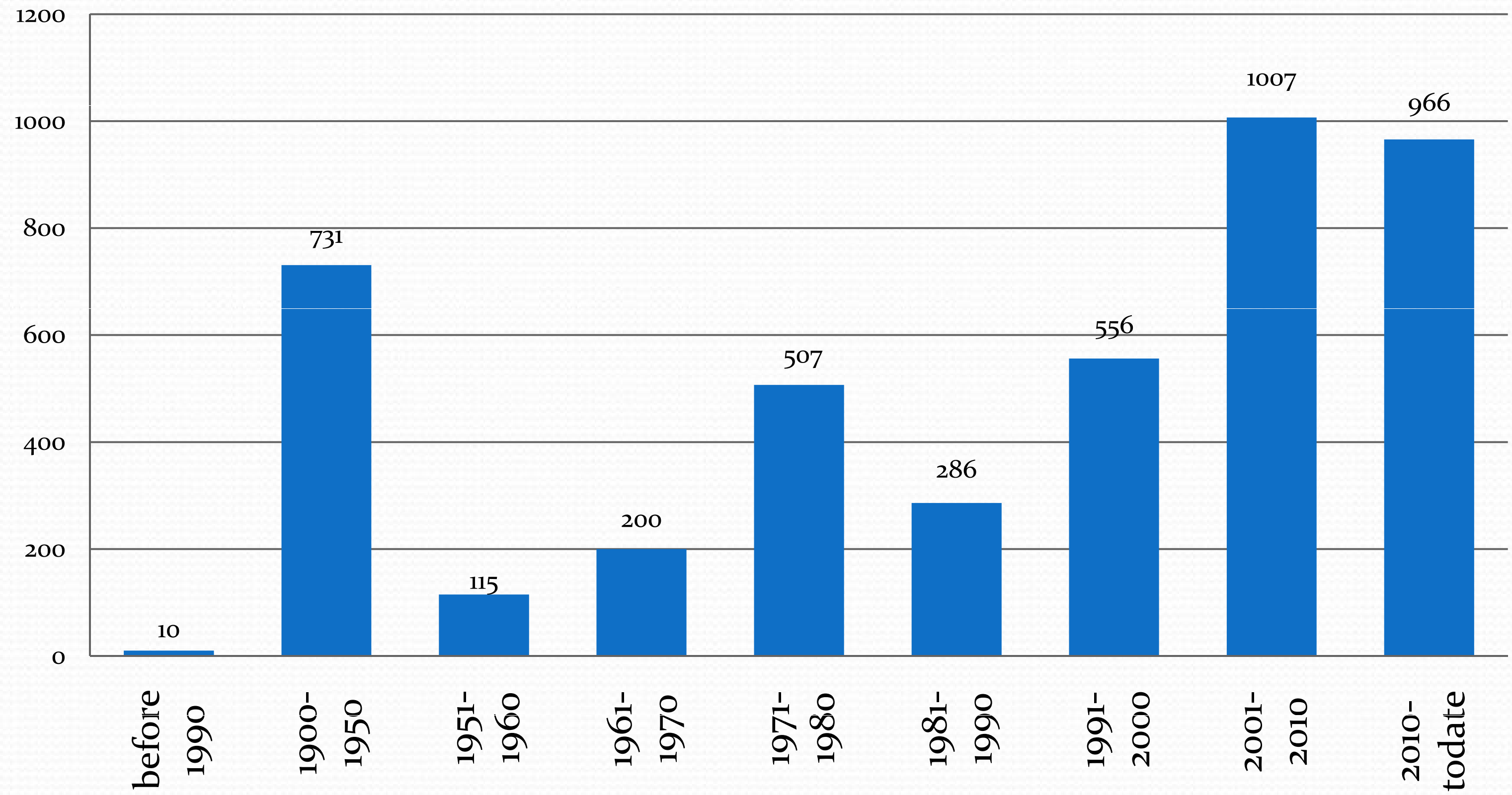
Bedessa Bridge -The bridge is Masonry Arch and **extended slab bridge** on both sides.The bridge is located at 127.02Km from A. A in D/Berhan – Gudoberet road segment.



Introduction

Bridges by year of construction

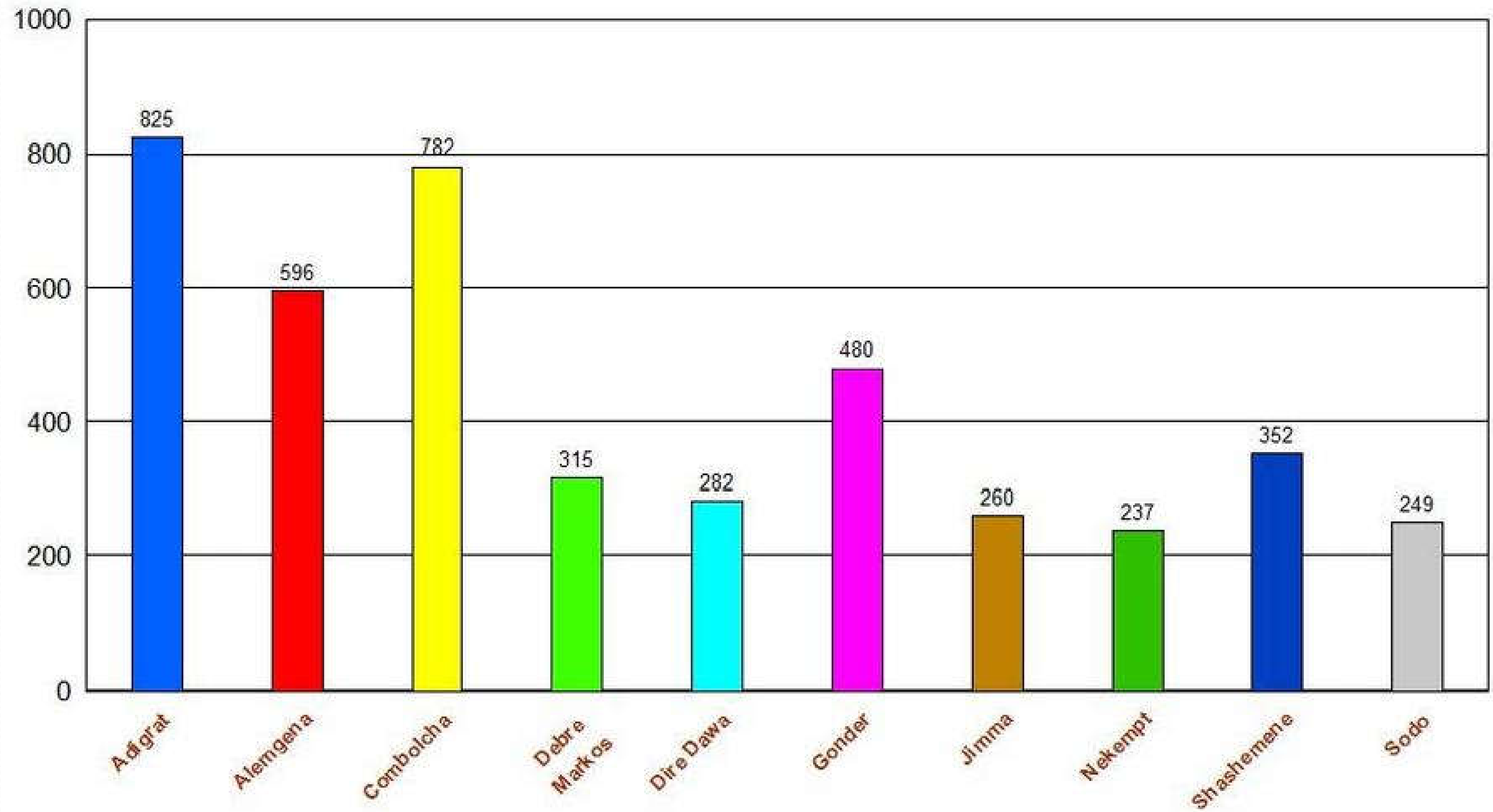
(Bridges along Federal road Network)



Almost **40%** of the bridges are found in ***fair & bad conditions***

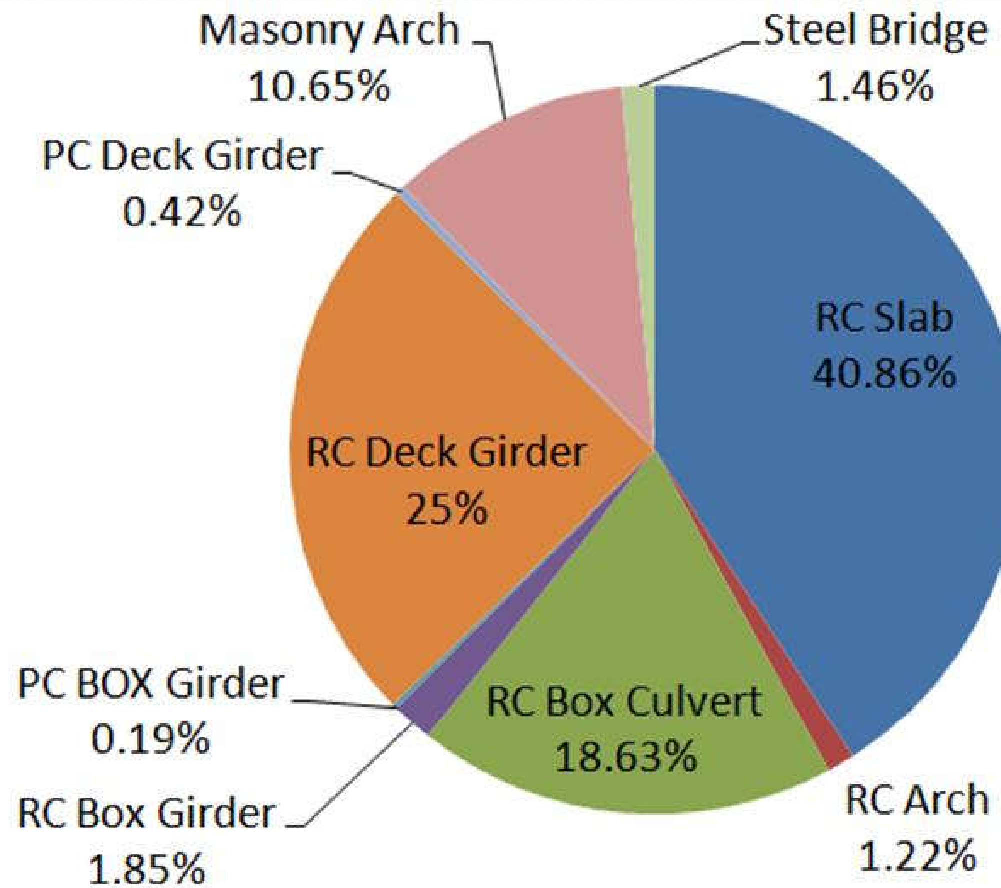
ERA (Ethiopian Roads Authority) Data ERA-BMS Software (as of 2018)

Bridges by District Offices



Total No. of Bridges= 4,378 (as of 2018)

Bridge by Type (Bridges along Federal road Network)

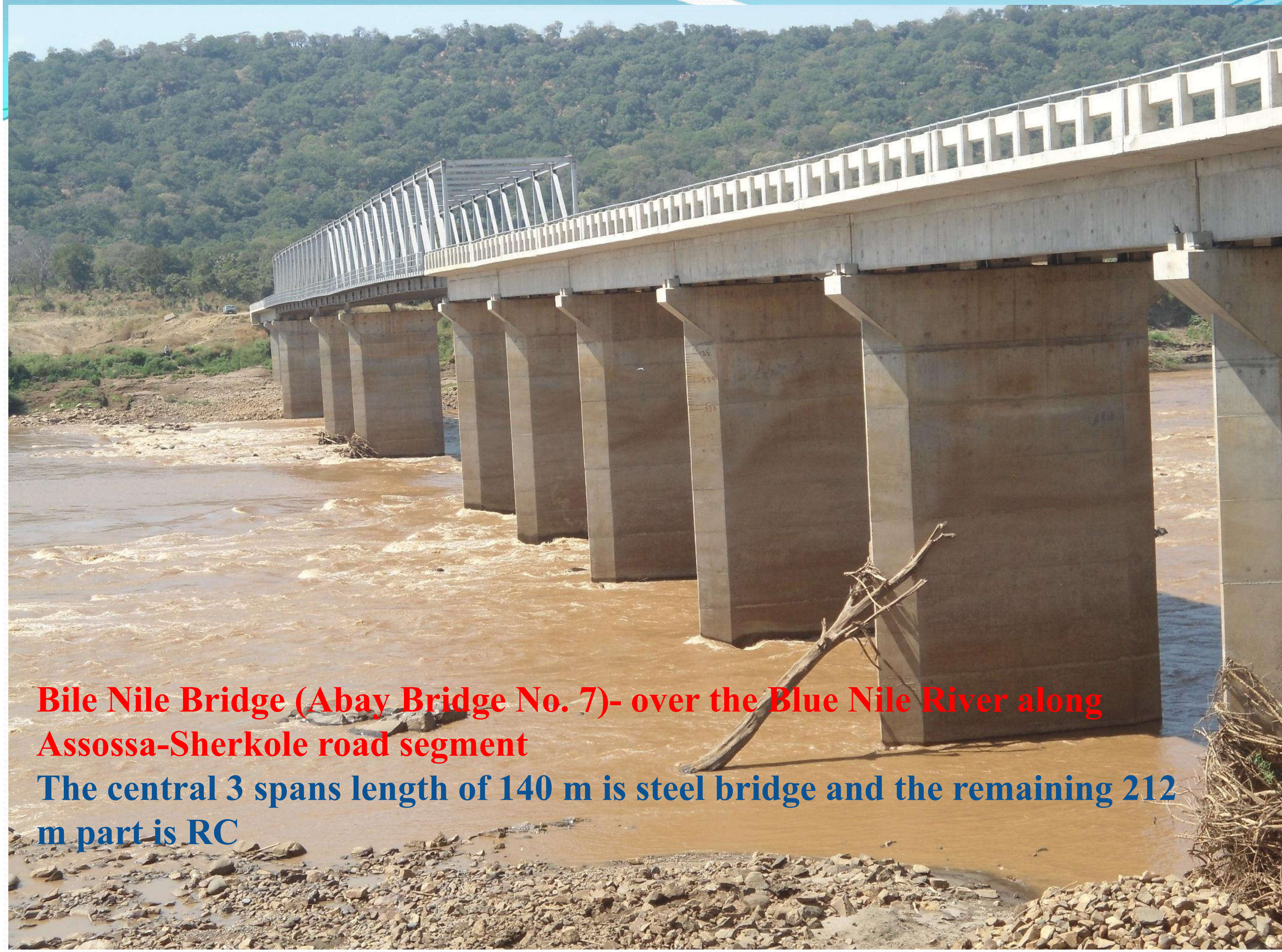


ERA (Ethiopian Roads Authority) Data ERA-BMS Software

Major Bridges in Ethiopia

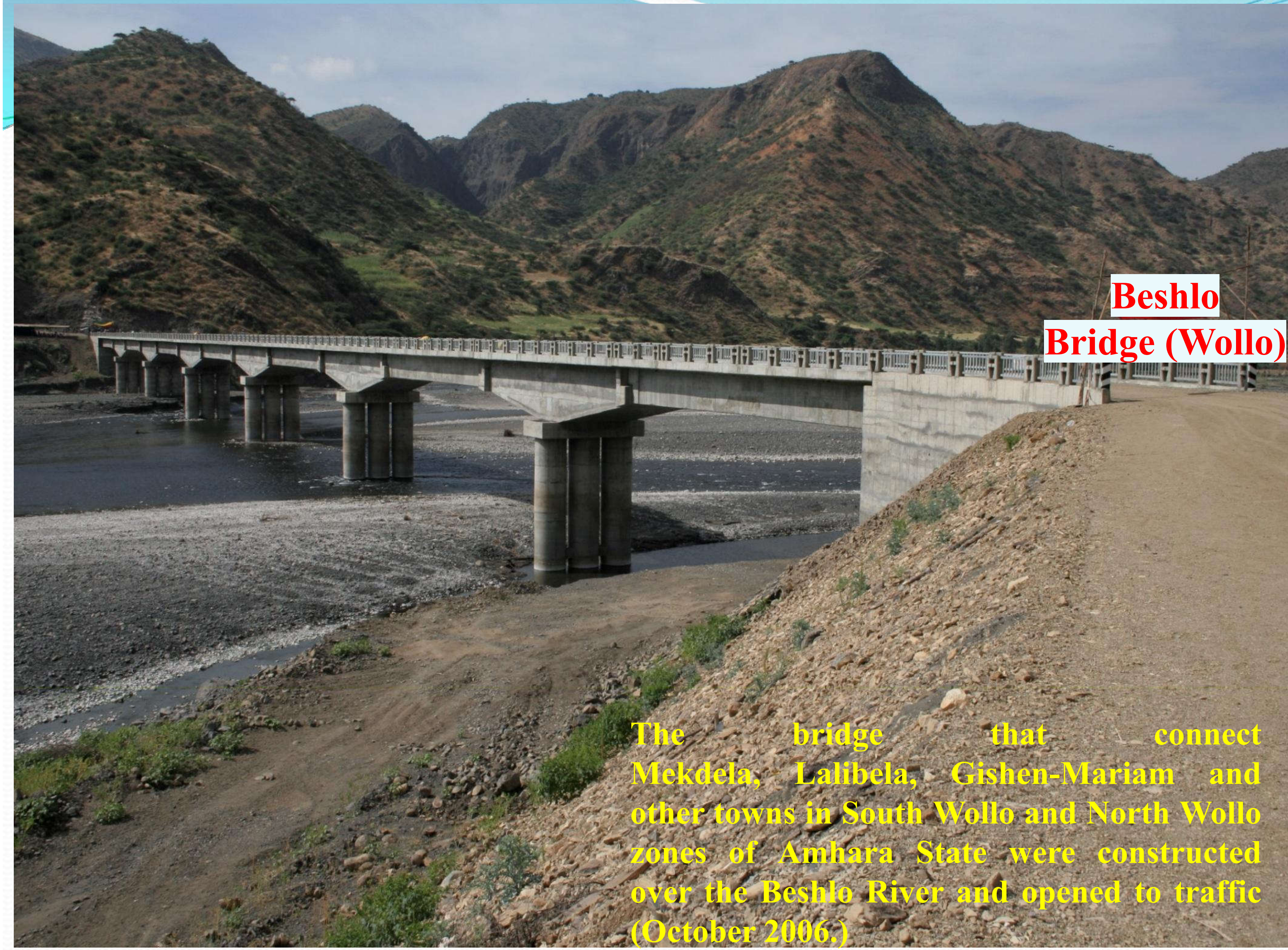
No.	Bridge Name	Type	Length	Year	Remark
1	Tekeze	RC Deck Girder	424	2014	
2	Blue Nile Abay Bridge No. 7	RC Deck Girder and Steel Bridge	355	2010	Along Sherkole-Blue Nile road segment (Asossa)
3	Beshilo	RC Deck Girder	319	2002	7-span
4	Hidasse Abay Bridge No. 6	Extradose, PC	303	2009	3-span
5	Tekeze	Steel Truss	280	2001	The longest steel Bridge
6	Baro	RC Deck Girder	276	1981	Multi span bridge
7	Abay Bridge No.5	PC Box Girder	236	1992	Variable depth of Box Girder
8	Abay Bridge No. 4	13-span RC Girder	184	1959	Bahir Dar connecting Gojam and Gondar


Tekeze Bridge



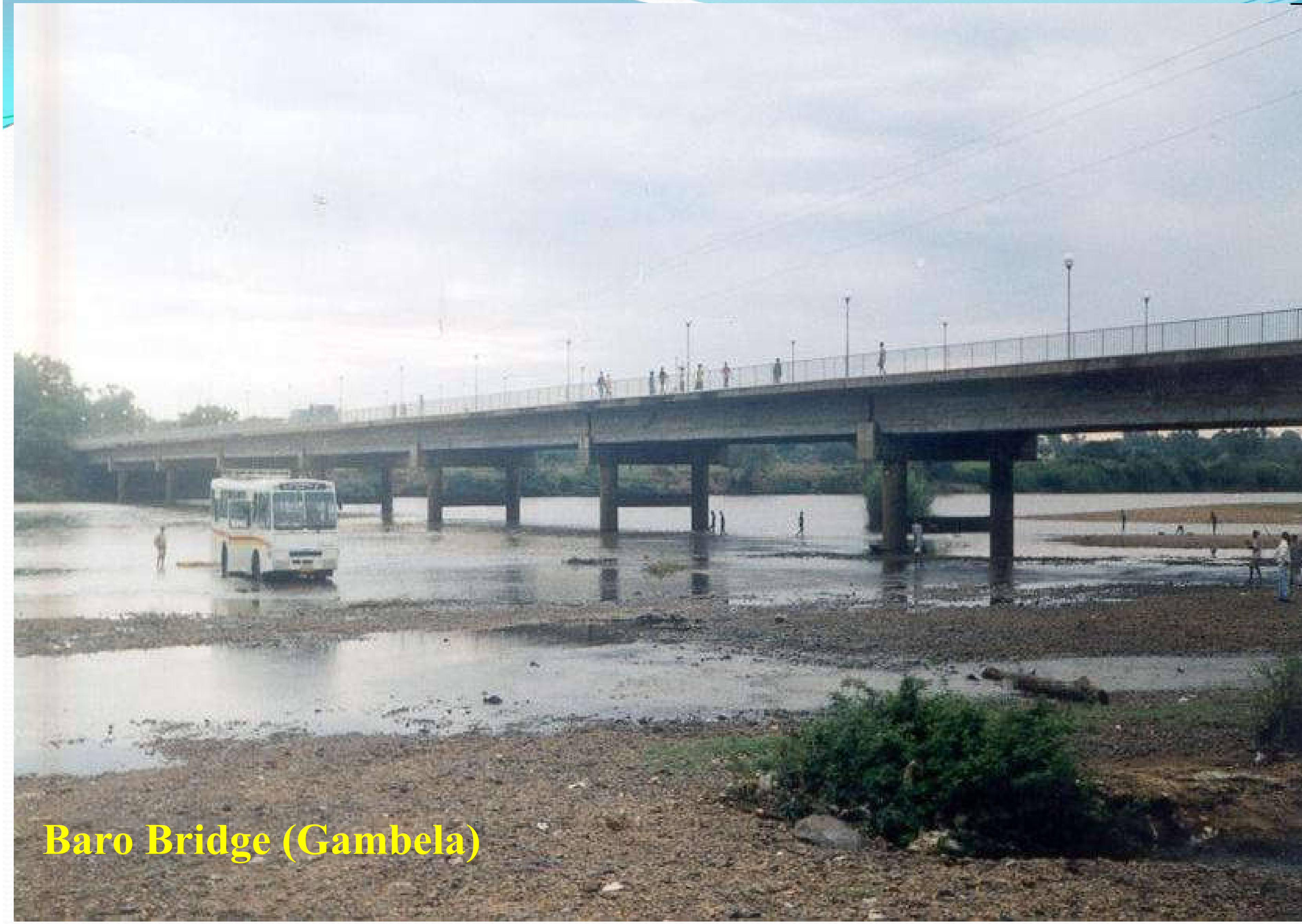
Bile Nile Bridge (Abay Bridge No. 7)- over the Blue Nile River along Assossa-Sherkole road segment

The central 3 spans length of 140 m is steel bridge and the remaining 212 m part is RC


Beshlo
Bridge (Wollo)
The bridge that connect Mekdela, Lalibela, Gishen-Mariam and other towns in South Wollo and North Wollo zones of Amhara State were constructed over the Beshlo River and opened to traffic (October 2006.)

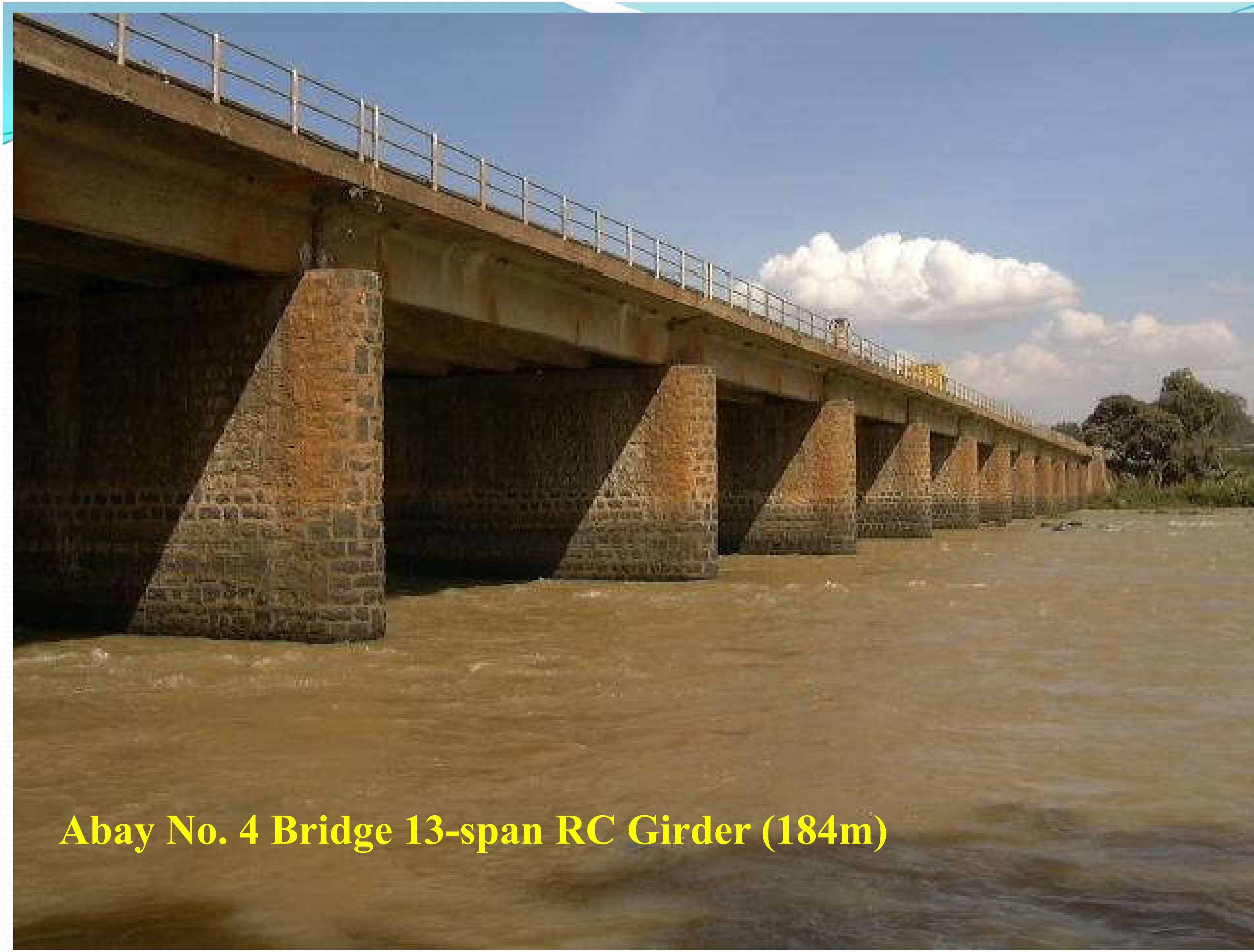

Abay Hidassie Bridge

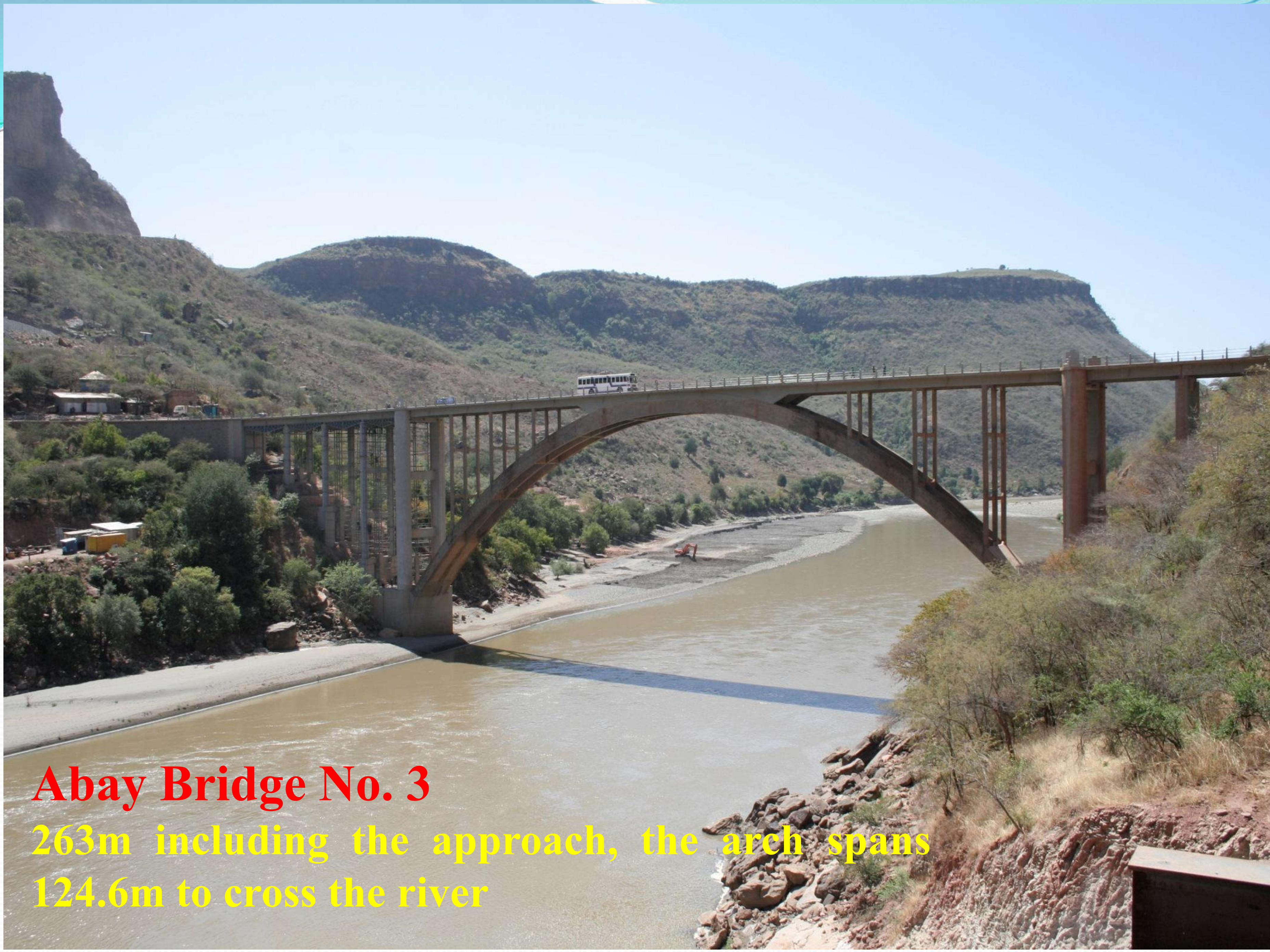

Tekeze Bridge

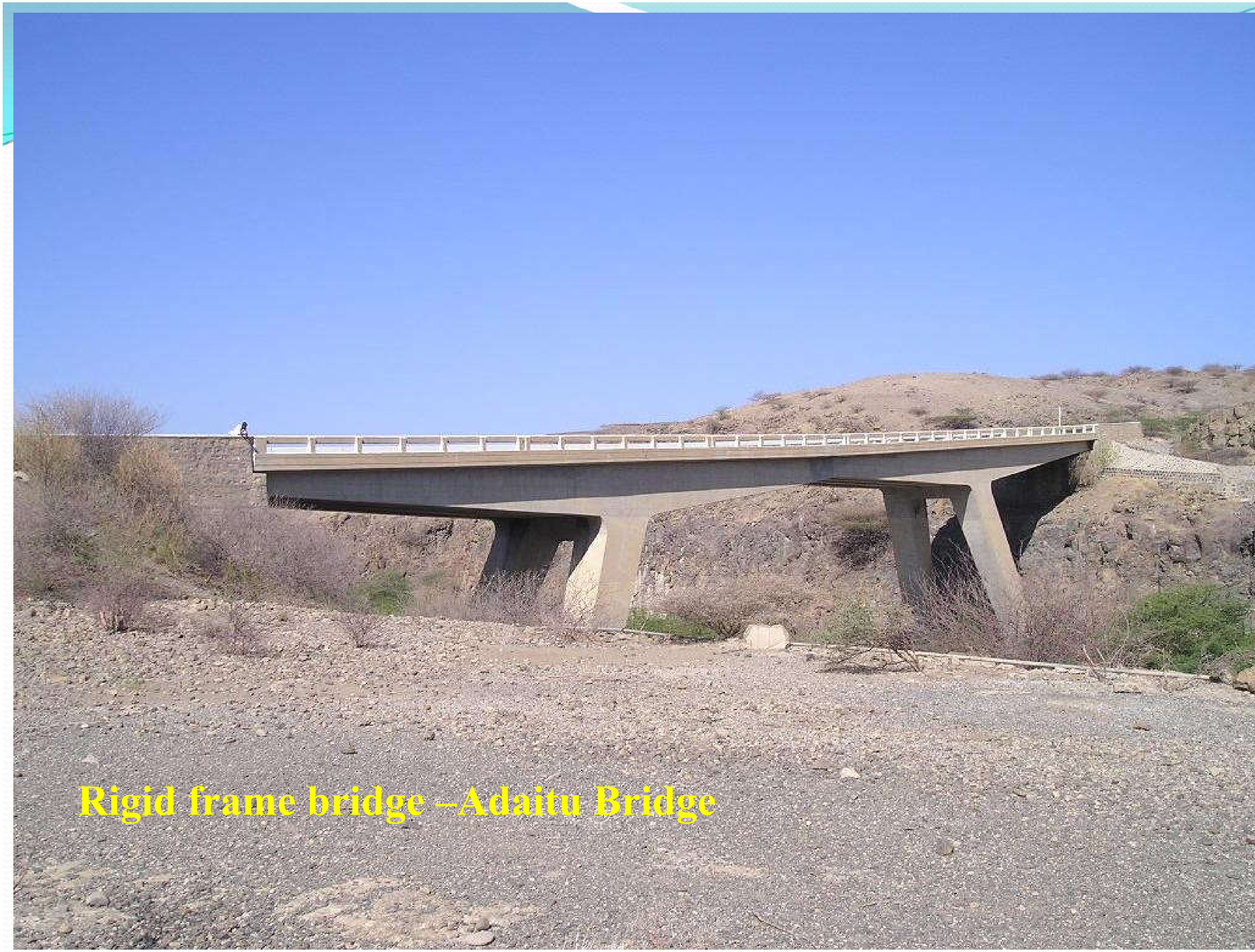

Baro Bridge (Gambela)

Abay No. 5 Bridge

The first Post Tensioned Box bridge type having long continuous superstructure in the country connecting Nekempt and Bure


Abay No. 4 Bridge 13-span RC Girder (184m)


Abay Bridge No. 3
263m including the approach, the arch spans
124.6m to cross the river


Rigid frame bridge –Adaitu Bridge

Longest HW Bridge

The Bang Na Expressway is one of the longest road bridges in the world (54,000 m)
Completed in 2010
It is the 6th longest bridge in the world
*Rank 1-5 are RW bridges

Suspension bridges

The world's **longest suspension bridges** are listed according to the length of their main span (i.e. the length of suspended roadway between the bridge's towers). The length of main span is the most common method of comparing the sizes of suspension bridges.

The Akashi Kaikyo Bridge has the longest central span of any suspension bridge.

[Akashi Kaikyō Bridge](), Japan
Main span=1,991m
Year opened=1998

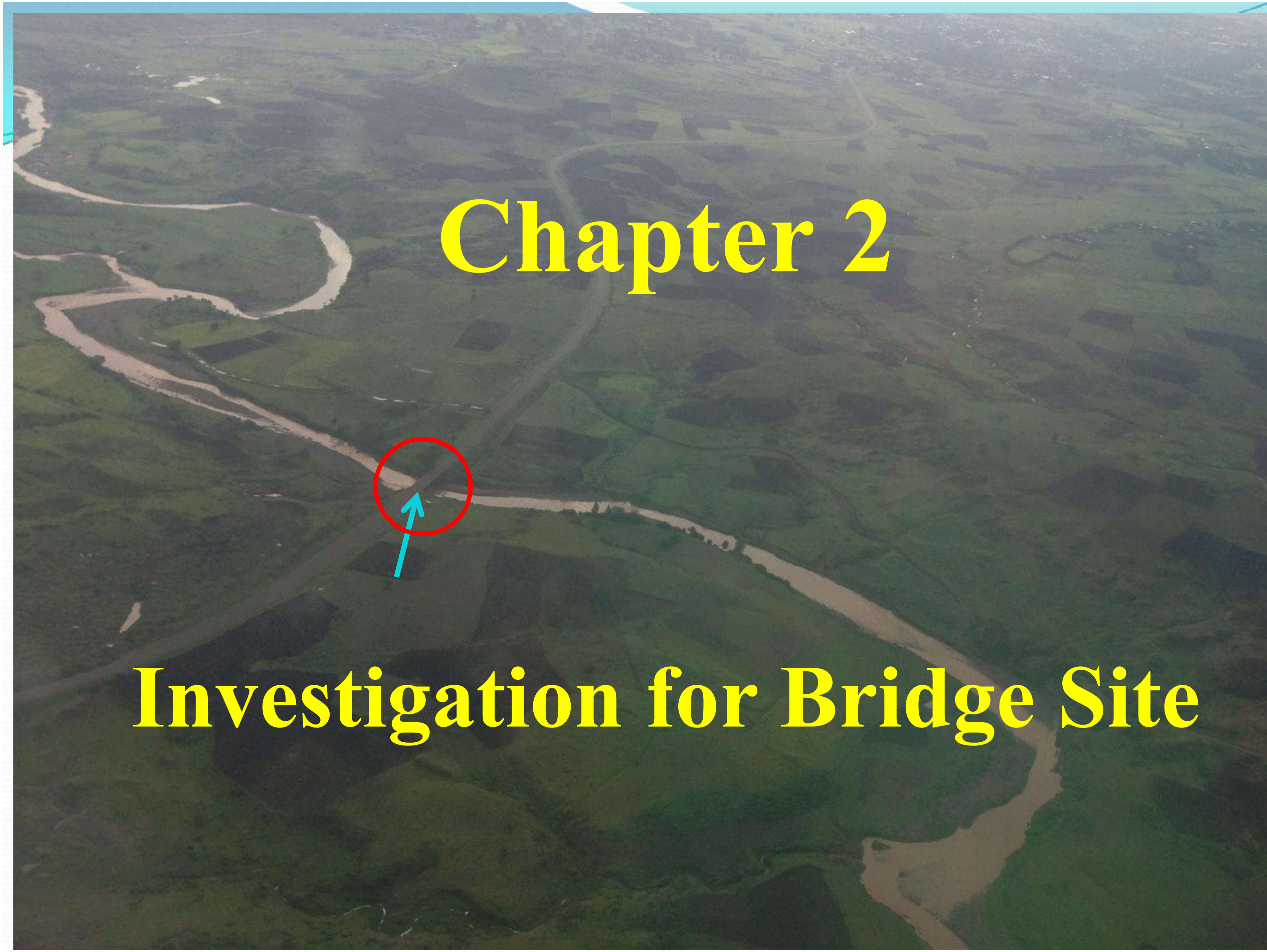
Second Humen Bridge [zh] (East main span), China
Main span=1,688m
Year opened=2019

Xihoumen Bridge, China
Main span=1,650m
Year opened=2009

Great Belt Bridge, Denmark
Main span=1,624m
Year opened=1998

Osman Gazi Bridge, Turkey
Main span=1,550m
Year opened=2016

The world's **longest suspension bridges**
1- Japan
2-China
3-China
4-Denmark
5-Turkey



Chapter 2

Investigation for Bridge Site

2. INVESTIGATION FOR BRIDGE SITE

The aim of the investigation is to select a suitable site from possible alternative at which bridge can be built economically ,at the same time satisfying the demands of safety, traffic ,the stream ,and aesthetic.

To achieve this field surveys and gathering of information on lists below is required

- the proposed road alignment
- the local terrain and site condition
- the required design life of the bridge
- the likely traffic volumes and the resources available for the project

2. INVESTIGATION FOR BRIDGE SITE

Bridge Site Selection

Preliminary Survey

Objective to study more than one alternative bridge sites by using topographic map.

Selection of Bridge Site

There are three initial consideration to bear in mind in selection of the bridge site :

- A bridge site must offer appropriate vertical and horizontal alignment

  (skew angle above 20^0 should be avoided due to increased bridge cost)

2. INVESTIGATION FOR BRIDGE SITE

Bridge Site Selection

In locating a bridge crossing the following should be considered

- The reach of the river should be straight
- The channel in the reach should be well defined (Used mainly for the flood analysis)
- The crossing site should be as narrow as possible
- The crossing site should have firm high banks which are fairly inerodable.
- The site should be selected where skewness can be avoided

River Survey

Information required by the designer for analysis and design include not only the physical characteristics of the land and channel, but all features that can affect the magnitude and frequency of the flood flow which will pass the site under study.

- These data may include *climatological characteristics, land runoff characteristics, high water marks ...*

High water marks can be obtained from: the hydrologic characteristics of the basin or watershed of the stream under study are needed for any predictive methods used to forecast flood flows, gauges or from local people.

Soil Investigation

- Once the Engineer has identified a likely site for the bridge, he/she needs to obtain field information on the catchment area and run off, local terrain, river conditions and water levels, navigational and other clearance requirements, and soil information.

Once at the site it is easy and of great value to sample for soil, rock, stone, water, etc.

  - Soil investigation is required to get soil profile, engineering property of the foundation material and foundation level of the abutments and piers for design of the foundation.
  - boreholes, test pits or geophysical surveying shall be conducted

Soil Investigation

- Its soil must have sufficient strength to ensure stability of the structure
- The bridge and its associated works should not have an adverse impact on adjoining land or building ,or be susceptible to damage from/to the local environment

For a river crossing it is important to identify the type of river to be crossed . There are two types of rivers namely alluvial and incised.

- **Alluvial rivers** are winding and they erode their banks and scour their beds; They are continually active, scouring and depositing materials on the banks and transporting sediments.
- **Incised rivers** have a relatively stable banks and arc generally narrower and deeper than alluvial rivers.

Soil Investigation

- Soil investigation Studies should provide the following Information:
  - Selection of the type and the depth of foundation suitable for a given structure.
  - Evaluation of the load-bearing capacity of the foundation.
  - Estimation of the probable settlement of a structure.
  - Establishment of ground water table.

Data Collection

Once the engineer has identified a likely site for the bridge, it is necessary to obtain field information on the catchment area and run off, local terrain , river conditions and water levels, and soil information.

Factors that most often need to be confirmed by field inspection

- High-water marks or profile and related frequencies
- Evaluation of apparent flow directions and diversions,
- Flow concentration (main stream)
- Observation of land use and related flood hazards , and
- Soil conditions

Field Sketching and Photos

- It has proved very practical to make a simple ***sketch*** of the bridge site with approximate water shores, existing structures, scour holes, main stream location, etc including very rough dimensions with approximate measurements.

Drawings

- ***Index Map -*** showing the proposed location of the bridge, the alternative bridge sites investigated and rejected, towns and villages in the vicinity and the general topographic of the area.
- ***Contour Survey Plan-*** of the river showing all topographic feature for a sufficient distance on either side of the site to give indication of the features.
- ***Site plan -*** showing the details of the selected site and of the stream to a distance of 100 to 200m upstream and downstream of the selected site.

ERA Site Investigation Manual, 2013

Hence, the investigation of potential sites and alignments is a vital and integral part of the location, design and construction of a road and its associated structures. It provides essential information on the following:

- Characteristics of the soils along the possible alignments;
- Availability of construction materials;
- Topography;
- Land use;
- Environmental issues; and
- Socio-political considerations.

ERA Site Investigation Manual, 2013

Typical uses of the information are;

- Selection of the route/alignment of the road;
- Location of water crossings and drainage structures;
- Provision of design information for the road pavements, bridges and other structures;
- Identification of areas of possible geotechnical problems requiring specialist investigation;
- Identification of areas of possible problem soils requiring additional investigation and treatment;
- Location and assessment of suitable, locally available, borrow and construction material.

1.4 Stages of Site Investigation

Some form of site investigation is required at all stages in the development of a road project. In general there are four stages leading up to and including Final Engineering Design. These are;

1 Identification and general planning;

2 Pre-feasibility study;

3 Feasibility Study or Preliminary Engineering Design;

4 Final Engineering Design.

Span Determination

Span determination is usually dictated by the hydraulic requirement. However, there are conditions where lengthen spans are chosen for the sake of road alignment.

1. Economical Span

Structural types, span lengths, and materials shall be selected with due consideration of projected cost. The cost of future expenditures during the projected service life of the bridge should be considered. Regional factors, such as availability of material, fabrication, location, shipping, and erection constraints, shall be considered.

(For a given span the most economical span is the length at which superstructure cost equals to substructure cost.)

2. Hydraulic requirement (DFL,OFL,…)

- Bridges are designed to accommodate **design discharge** at design flood. When a river has a wide flood plain, the economical solution may be using short span bridge with proper scour and erosion protection for the embankment, abutments and piers.

Design flood is the maximum flow that can pass through the bridge with out

- Causing unacceptable disruption to traffic
- Endangering the pier and abutment foundation with scour
- Damaging approach embankments
- Causing flood damage on the upstream side of embankments

Estimating Design Flood

Flood Peak Discharge at Stream or River Location Depends upon:

- Catchment Area Characteristics
  - Size and shape of catchment area
  - Nature of catchment soil and vegetation
  - Elevation differences in catchment and between catchment and bridge site location
- Rainfall Climatic Characteristics
  - Rainfall intensity duration and its spatial distribution
- Stream/River Characteristics
  - Slope of the river
  - Baseline flow in the river
  - River Regulation Facilities/ Dams, Barrages on the river

Methods of Estimating Design Flood

- Empirical Methods

  $Q=C*A^{1/2}$

- Rational Formula

  $Q=C*I*A$

- Velocity Methods (Manning formula)

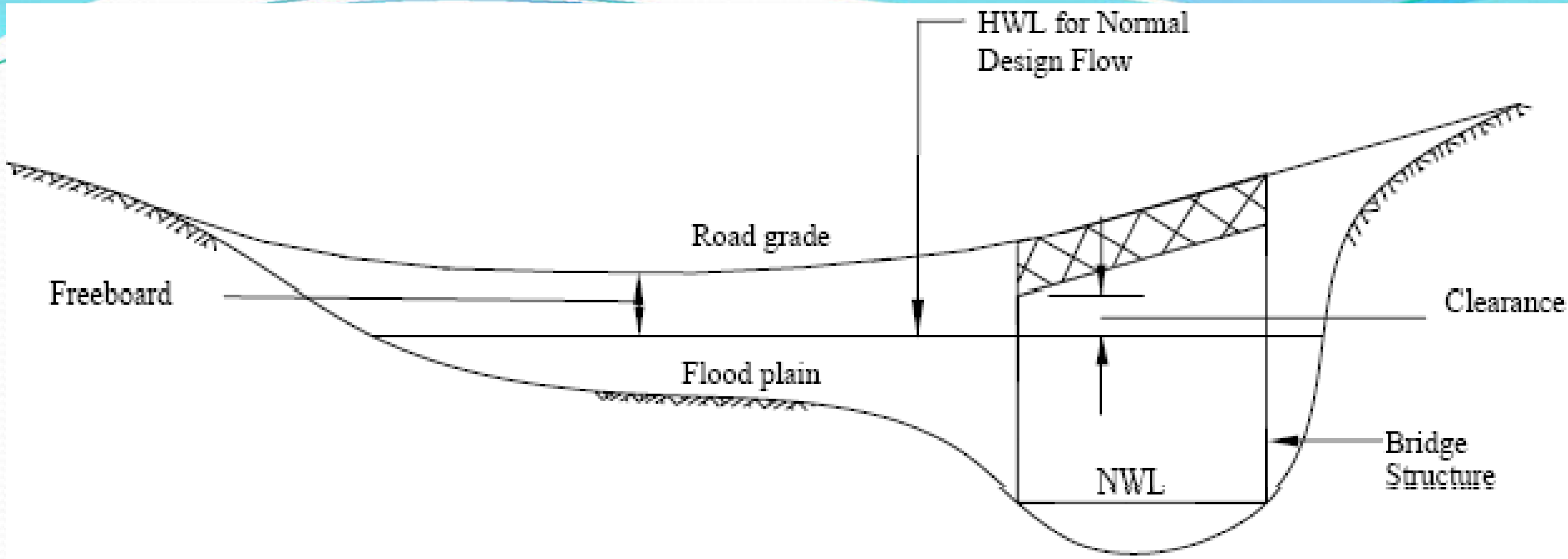
$$Q = \frac{1}{n} A R^{\frac{2}{3}} S^{\frac{1}{2}}$$

3. Location of Piers

- Piers should be located in such a manner that they can provide the required lineal waterway and navigational clearance.
- The alignment of piers and abutments should, if possible, be set parallel to the direction of flow during maximum flood.

4. Free Board

- The waterway below the superstructure must be designed to pass the design flood and the floating debris carried on it.
- These clearance measurements should be increased for backwater effects when the flow is restricted by short span bridge.



Discharge (m^3/s)	Vertical Clearance/ Free board (m)
0 to 3.0	0.3
3.0 to 30.0	0.6
30.0 to 300	0.9
> 300	1.2

5. Grade Requirement of the road

Often in mountainous areas the road way grade is governed by the capacity of heaviest vehicle to climb, vertical curve and sight distance. These requirements may increase the span beyond the hydraulic requirement.

Selection of Feasible site for Bridges

To select an optimum and the most feasible site , different options should be considered. The factors include:

- Geometry of Approach Road
- Approach Road Earth work volume and its impact
- Opening size
- ROW (Right of Way) and others
  - The Right of way along the Bridge crossing option consists of houses, fences and service lines.

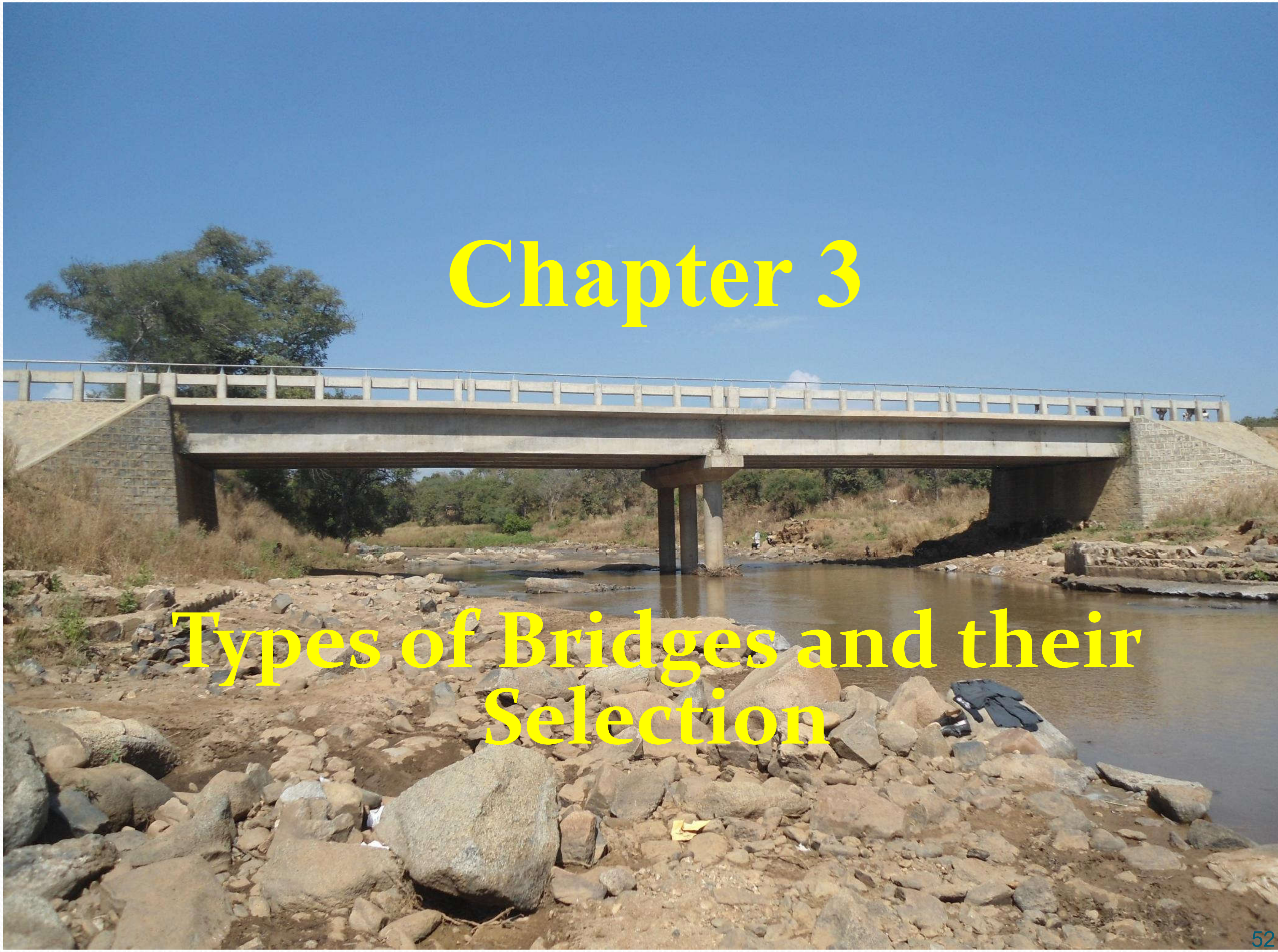
Location of Bridge's Crossing Options

- In order to select the best crossing, the relevant geometric design parameters of the approach road to the bridge crossing have to be considered in line with an appropriate Geometric Design manual.

Example

Alignment Option	Opening Size	General Remark
1	40.00 m	Requires longer approach road fill and retaing walls
2	80.00 m	Requires lesser RW
3	120.00 m	Requires longer RW and fill beside having large cut prohibiting easy access on the Philipos Church side

Options	Cost for Earth works	Cost for Embankment	Cost for Retaining wall	Cost for Approach Road Pavement	Cost for Bridge structure	Other costs	Total Cost
1							
2							
3							



Chapter 3

Types of Bridges and their Selection

Classification of Bridges

Bridges can be classified on the basis of the following Characteristics

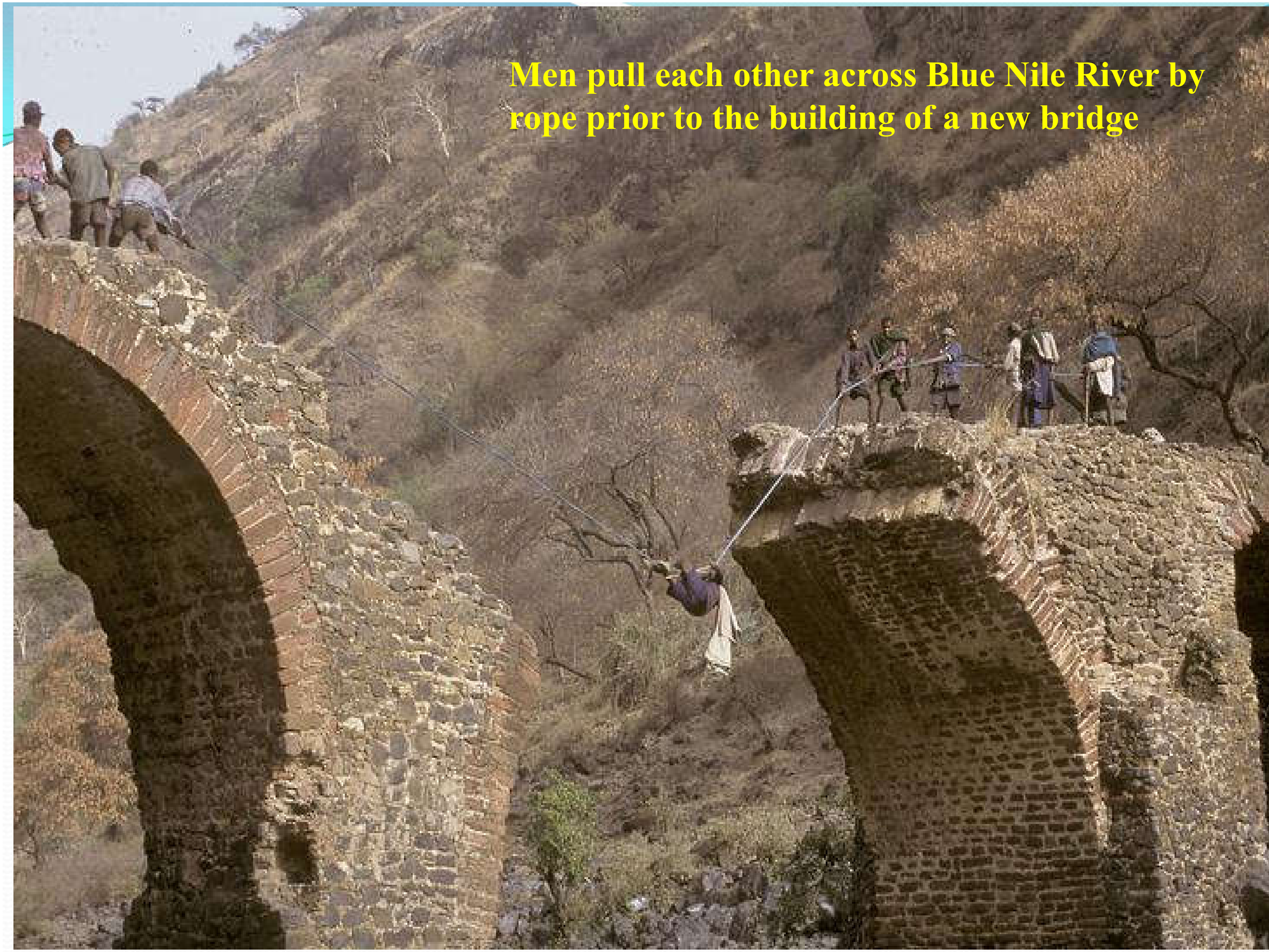
- Functionality as Road bridge, Railway bridge, Pedestrian bridge, military bridge, ...
- Construction material as Steel, Reinforced Concrete, Masonry, Timber or combination of any two or more.
- Structural forms as Slab, T-Girder, Box Girder, Arch, Rigid, cable stayed, Suspension etc.

Classification of Bridges

- Span type as simply support, continuous , cantilever.
- Movements as movable or fixed Bridges
- Arrangement as curved, skewed, inclined, straight, ..
- Life span as temporary, permanent
- Span length as short, medium ,large or extra large.

  - $L \leq 4m$ (Culvert)
  - $7m < L \leq 15m$ (Small span bridges)
  - $16 \leq L \leq 50m$ (Medium span Bridges)
  - $50 \leq L \leq 150m$ (Large Span Bridges)
  - $L \geq 150m$ (Extra Large Span Bridges)

Pedestrian Bridge


Men pull each other across Blue Nile River by
rope prior to the building of a new bridge

Highway Bridge

Railway Bridge

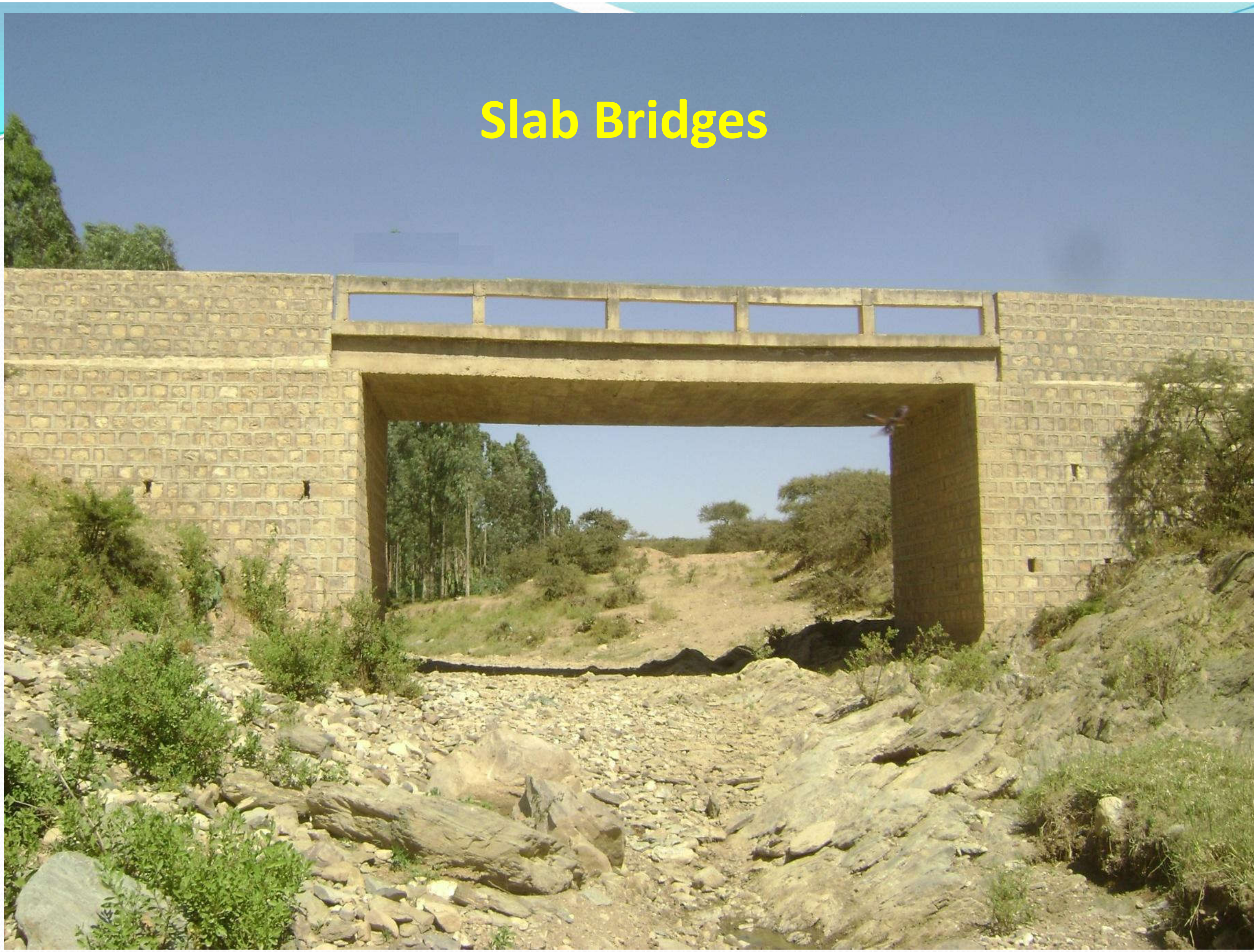
Military Bridge

Timber Bridge

Structural Forms

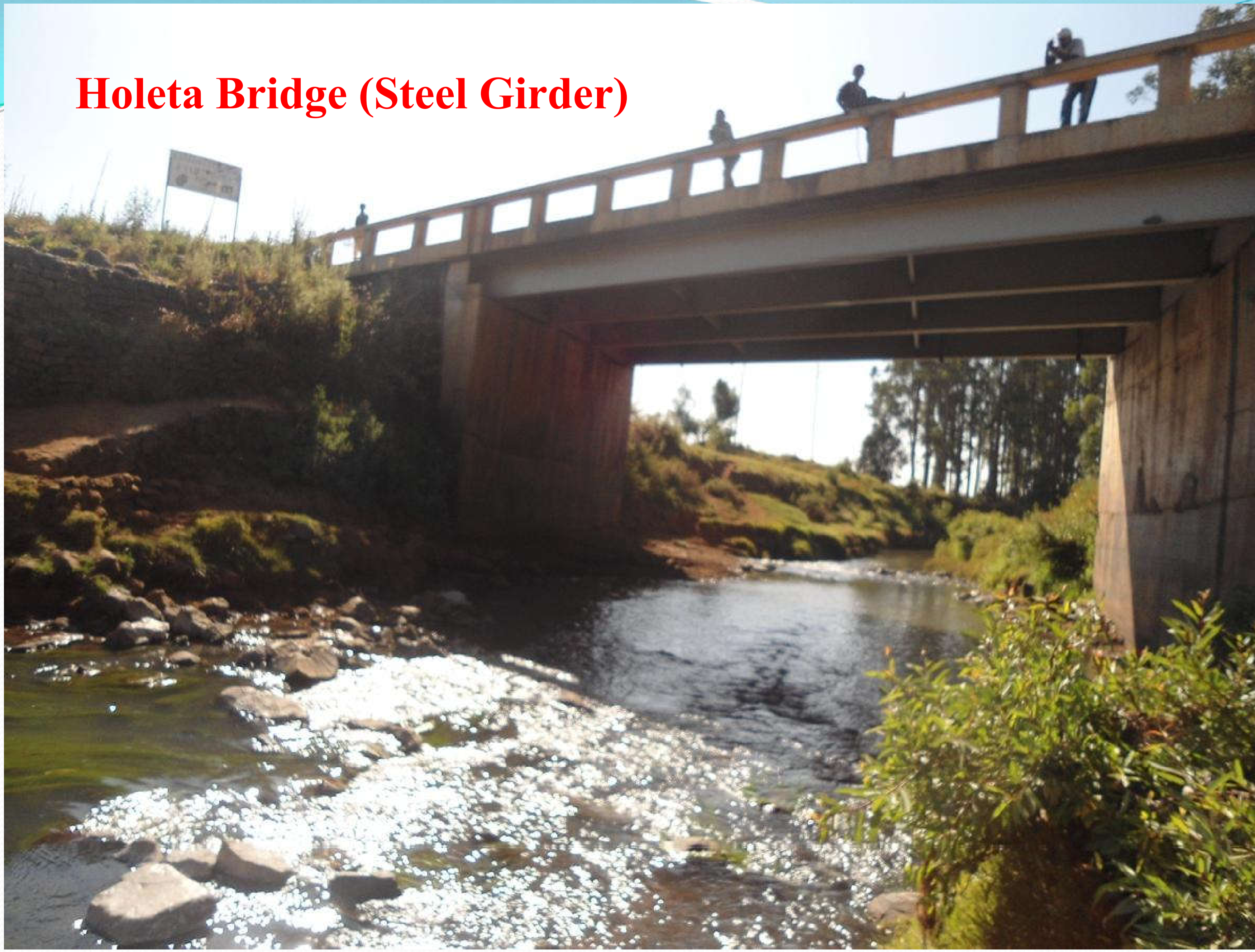
- \- Slab Bridges
- \- Girder (Deck girder Bridges)
- \- Box Girder
- \- Arch Bridges
- \- Truss Bridges
- \- Rigid Bridges
- \- Plate Girder Bridges
- \- Cable Stayed Bridges
- \- Suspension Bridges
- \- Box Cell/ Box culvert

Slab Bridges



Girder (Deck girder Bridges)

Holeta Bridge (Steel Girder)



Bridge Superstructure (U.S. Bridge Tour)


Slab Bridges
T-Girder Bridge
Chifra Bridge- Afar

Box Girder

Arch Bridge

Truss Bridge

Rigid Frame Bridge

superstructure and substructure are integrated

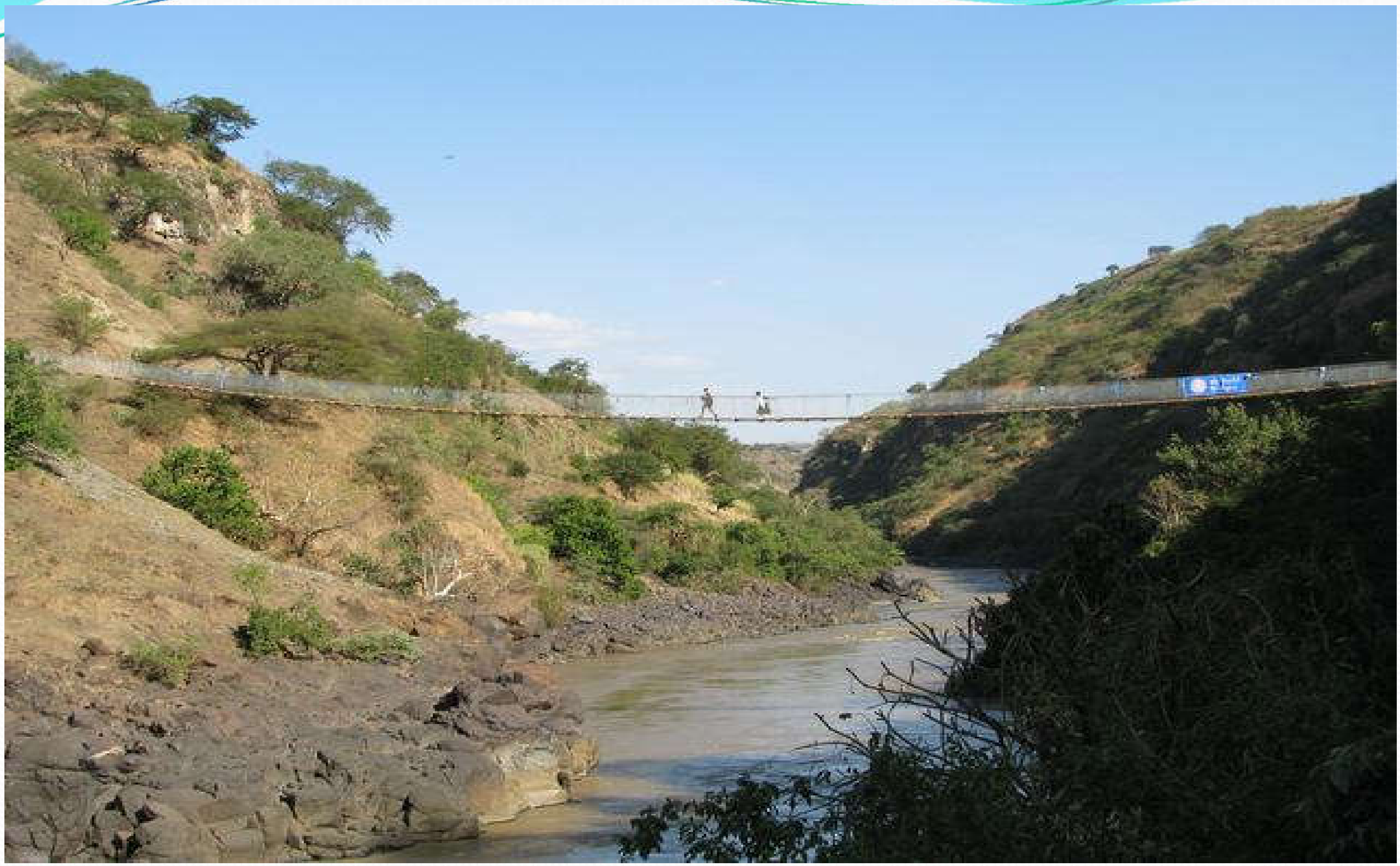
Adaitu Bridge

Steel deck plate Girder bridge

PC Bridge

Cable Stayed Bridge

Suspension Bridge



Pedestrian cable bridge over the Blue Nile River in Ethiopia (completed in 2009)


Akashi Kaikyo Bridge (1998)
Japan
1991m central span

Movable Bridge

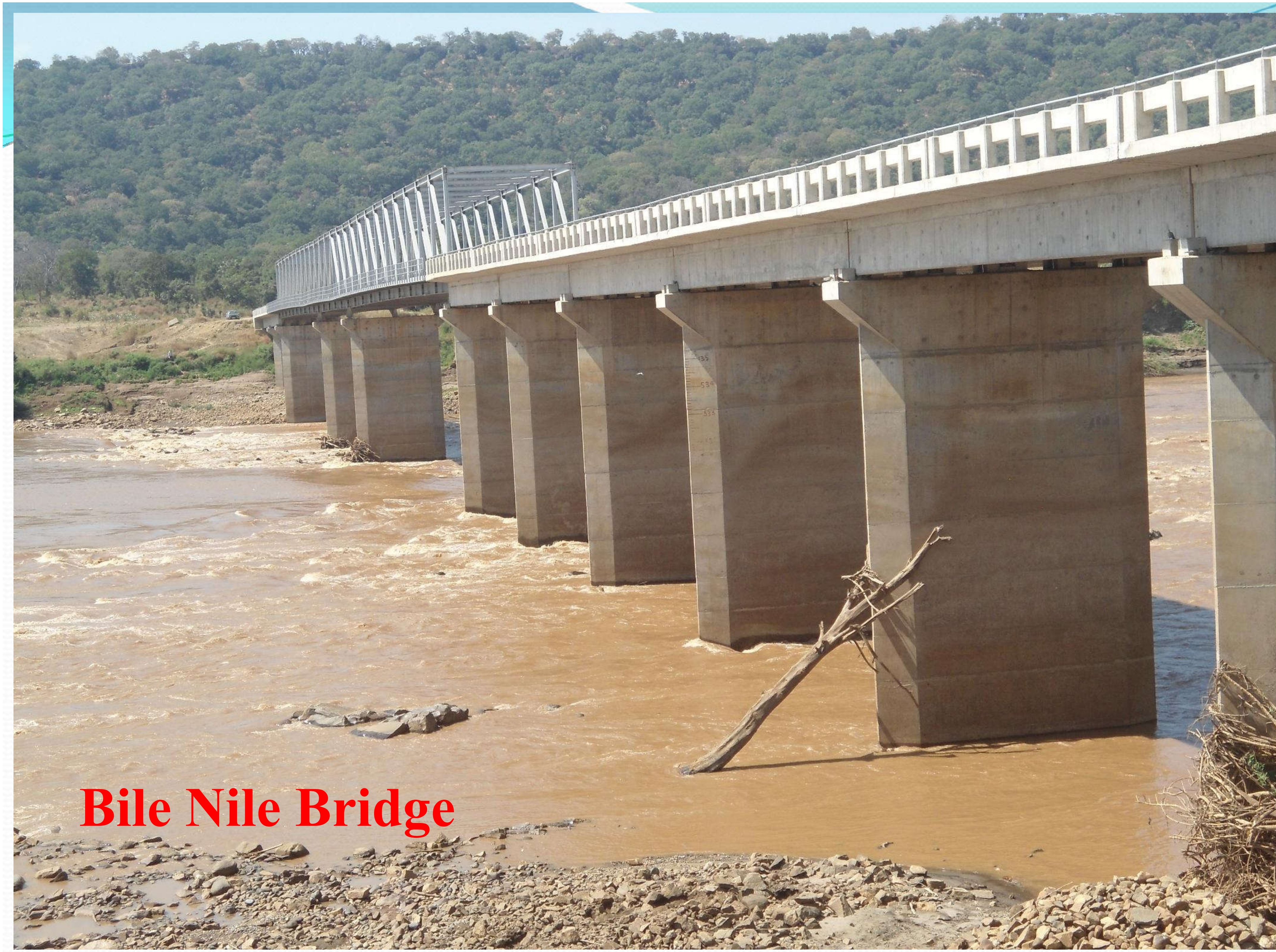
Lift Opening Bridge

France

Curved Bridge

Skewed Bridge

Box Cell/ Box culvert


Bile Nile Bridge

Span Support

This is on the basis of the Bridge's support.

- ✓ If all the spans of the bridge are simply supported (a pinned support and a roller support)- *Simple Span Bridge*
- ✓ A bridge supported by combinations of more than two pinned and roller supports - *Continuous Span Bridge*
- ✓ A bridge that is built-in at one end and free at the other - *Cantilever Span Bridge*

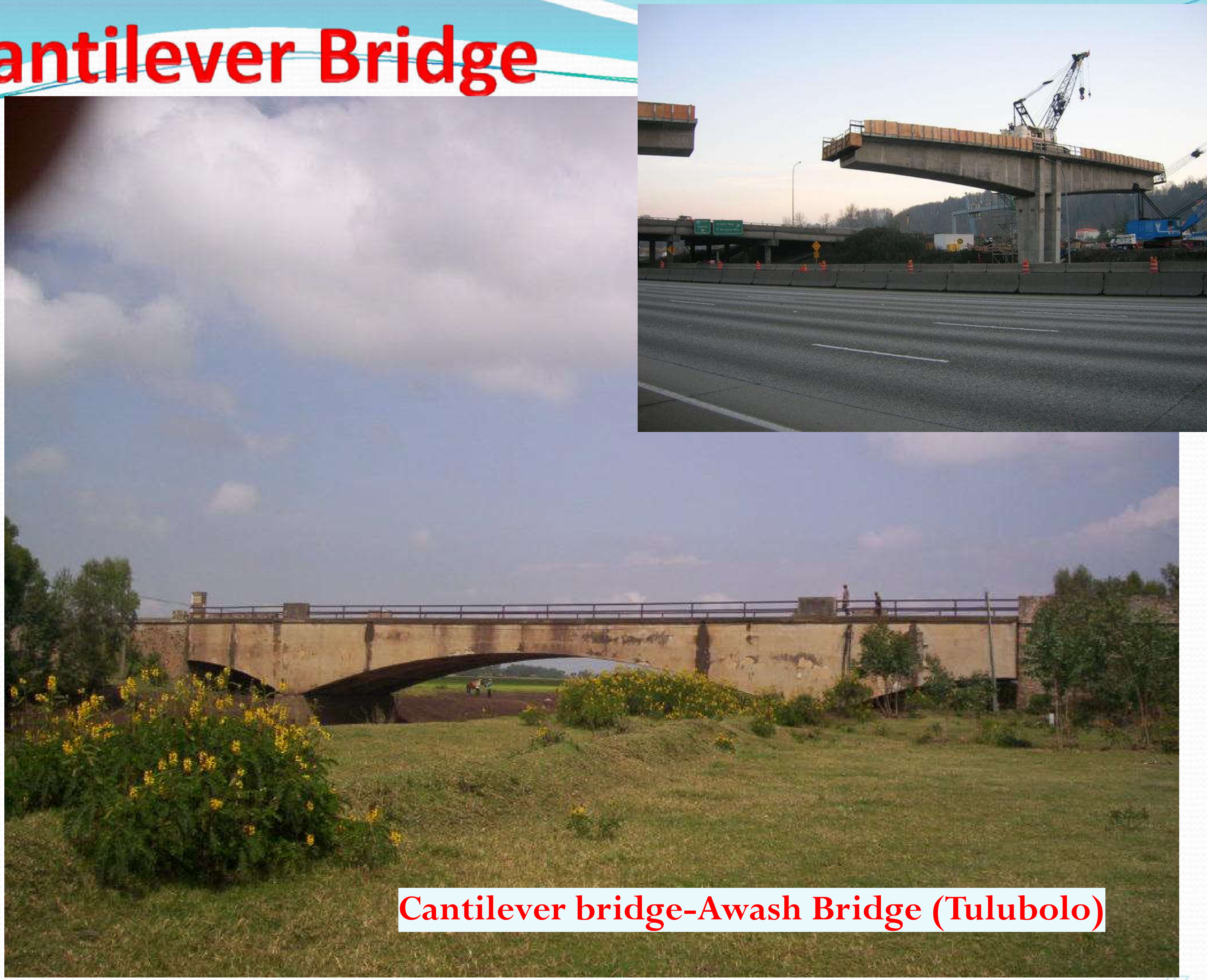
Simple Span Bridge

Multiple simple span bridge

Hunched Girder:- One method of increasing load capacity of the girder, while minimizing the web depth, is to add haunches at the supported ends.

Continuous bridge- Modjo Bridge

Cantilever Bridge



Cantilever bridge-Awash Bridge (Tulubolo)

Particular Problem of Selection

- Different Manuals recommend different span lengths to be criteria of selection of bridge types.
- No clear demarcations on the selection of spans for slab and T- Girder bridges if the span lies with in the intervals on the basis of economy. This is a serious design problem that Engineers face during selection of bridge type.
- Thus, in selecting bridge spans regional factors, such as availability of construction material, fabrication, location, transportation, erection constraints, inspection, maintenance, repair, and/or replacement shall be considered.

Selection of Bridge Type

- Typically there are three to four viable structure types for each span length. Criteria to select bridge include:
  - Geometric Condition of the Site
  - Subsurface Conditions of the Site
  - Functional Requirements
  - Economy and
  - ease of maintenance,
  - aesthetics, etc

Selection of Bridge Type

Economy:

- A general rule is that the bridge with minimum number of spans, fewest deck joints and widest spacing of girders will be the most economical.
- By reducing the number of spans, the construction cost of one pier is eliminated.

Construction and erection considerations:

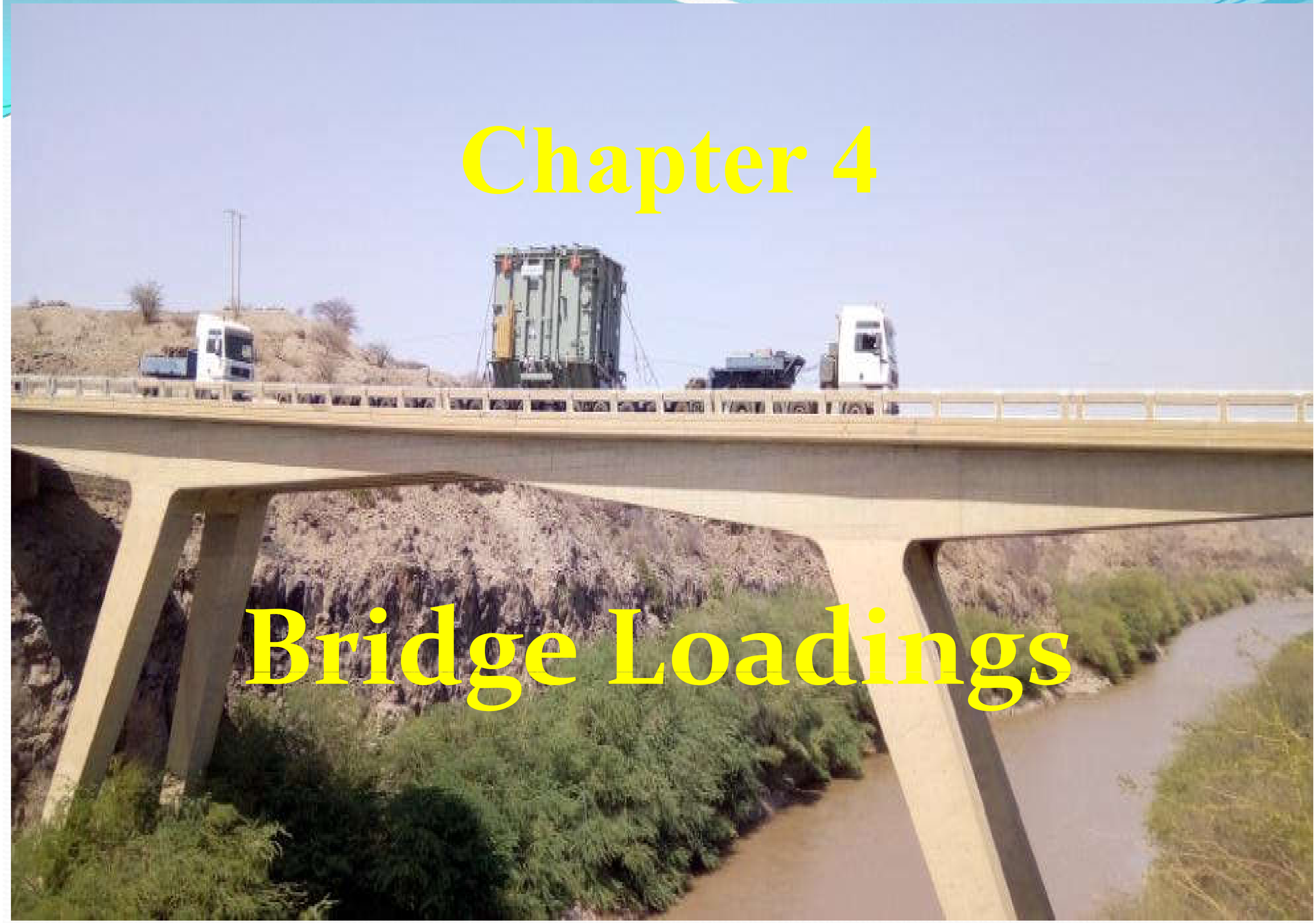
- In general, the larger the prefabricated or precast member, the shorter the construction time.
- However, the larger the members, the more difficult they are to transport and lift into place.
- The availability of skilled labour and specified materials will also influence the choice of a particular bridge type.

Criteria used for Determination of Location of a Bridge (Span and span arrangement)

- **Geometric requirements for bridges:** These are the requirements of the road alignment such as vertical and horizontal curves including grade.
- **Environmental concern:** bridge and its associated works should not have an adverse impact on adjoining land or buildings, or the bridge itself be susceptible to damage from the local environment.
- **Legal Considerations**: Applicable laws like environmental laws also govern the type of bridge.

Chapter 4

Bridge Loadings



- Bridges serve their function under different types of loads. Loads determine the safety and serviceability of structures.
- The Engineer must consider all the loads that are expected to be applied to the bridge during its service life.

Type of Loads

- **Permanent Loads**: Dead and Earth Loads
- **Transient Load:** Live, Water, Wind
- **Dynamic Loads**: Earthquake Loads
- Force effects due to superimposed deformations (temp gradient, shrinkage, creep, settlement, . .)
- **Friction Forces**
- **Vessel Collision**
- **Other stresses**

1. Permanent Loads

Permanent loads are loads that are always present in or on the bridge and do not change in magnitude during the life of the bridge. The LRFD Specification refers to the weights of the following as “permanent loads”:

- The structure
- Formwork which becomes part of the structure
- Utility ducts or casings and contents
- Signs
- Concrete barriers
- Wearing surface and/or potential deck overlay(s)
- Other elements deemed permanent loads by the design engineer and owner
- Earth pressure, earth surcharge and down drag

1.1 Gravitational Dead Loads

- **DC** – dead load of all of the components of the superstructure and substructure, both structural and non-structural.
- **DW** – dead load of additional non-integral wearing surfaces, future overlays and any utilities crossing the bridge.
- **EV** – vertical earth pressure from the dead load of earth fill.

Cont'd

1.2 Earth Pressures

- **EH** – horizontal earth pressure.
- **ES** – earth pressure from a permanent earth surcharge (e.g., an embankment).
- **DD** – loads developed along the vertical sides of a deep-foundation element tending to drag it downward typically due to consolidation of soft soils underneath embankments reducing its resistance.

2. Transient Loads

- Transient loads are loads that are not always present in or on the bridge or change in magnitude during the life of the bridge. Specific transient loads include :

2.1 Live Loads

- **LL –** Static vertical gravity loads due to vehicular traffic on the roadway
- **PL –** Vertical gravity loads due to pedestrian traffic on sidewalks.
- **IM –** dynamic load allowance to amplify the force effects of statically applied vehicles to represent moving vehicles.

Cont'd

- **BR –** horizontal vehicular braking force.
- **CE –** horizontal centrifugal force from vehicles on a curved roadway.

2.2 Water Loads

- **WA–** pressure due to differential water levels, stream flow or buoyancy.

2.3 Wind Loads

- **WS –** horizontal and vertical pressure on superstructure or substructure due to wind
- **WL –** horizontal pressure on vehicles due to wind.

2.4 Extreme Events

- **EQ –** loads due to earthquake ground motions.
- **CT –** horizontal impact loads on abutments or piers due to vehicles or trains.
- **CV –** horizontal impact loads due to aberrant ships or barges.

2.5 Superimposed Deformations

- **Force Effects Due to Superimposed Deformations: TU, TG, SH, CR, SE**
- **Uniform temperature (TU)–** uniform temperature change due to seasonal variation. It is used to calculate thermal deformation effects.
- **Temperature Gradient (TG)–** temperature gradient due to exposure of the bridge to solar radiation. Temperature rise can differ on the top and bottom surfaces of a bridge.

2.5 Superimposed Deformations cont'd

- **Differential Shrinkage (SH) –** differential shrinkage between different concretes or concrete and non-shrinking materials, such as metals and wood.
- **Creep (CR)** - dependence on time and changes in compressive stresses shall be taken into account.
- **Settlement (SE) -**the effects of settlement of substructure units on the superstructure. This will cause internal forces in continues structures.

Dead Load

- Dead load shall include the weight of all components of the structure, appurtenances and utilities, earth cover, wearing surface, future overlays, and planned widening. In the absence of more precise information, the unit weights, specified in AASHTO, Table 3.5.1-1, may be used for the computation of dead loads.

Dead Load

Densities and Force Effects of Different Materials

MATERIAL		DENSITY (kg/m^3)	Force effect (kN/m^3)
Bituminous Wearing Surfaces		2250	22.5
Cast Iron		7200	72
Cinder (volcanic stone) Filling		960	9.6
Compacted Sand, silt, or Clay		1925	19.3
Concrete	Normal	2400	24
Loose Sand, Silt, or Gravel		1800	18
Soft Clay		1700	17
Rolled Gravel or Ballast		2250	22.5
Steel		7850	79
Stone Masonry		2725	27.3
Wood	Hard	960	9.6
	Soft	800	8
Water	Fresh	1000	10

Vehicular Live Loads

Vehicular live load is designated as **HL-93** and shall consist of a combination of the followings:

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

Except as modified in Article 3.6.1.3.1 of ERA Bridge Design Manual, each design lane under consideration shall be occupied by either the design truck or tandem, coincident with the lane load, where applicable. The loads shall be assumed to occupy 3000mm transversely within a design lane

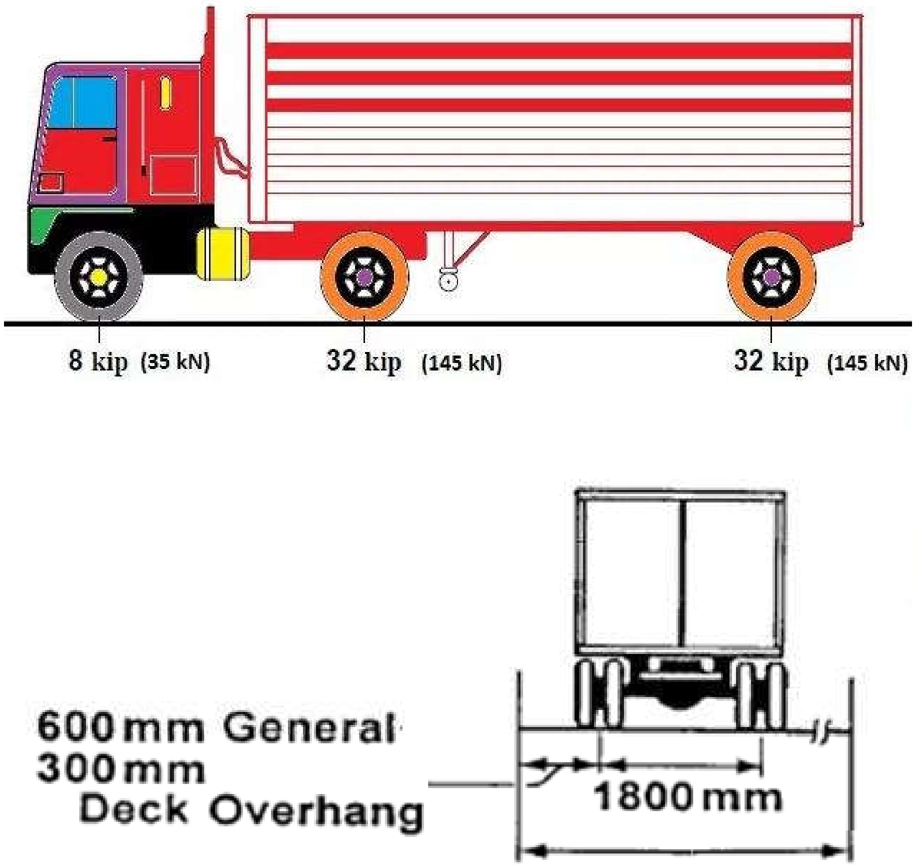
Vehicular Live Loads

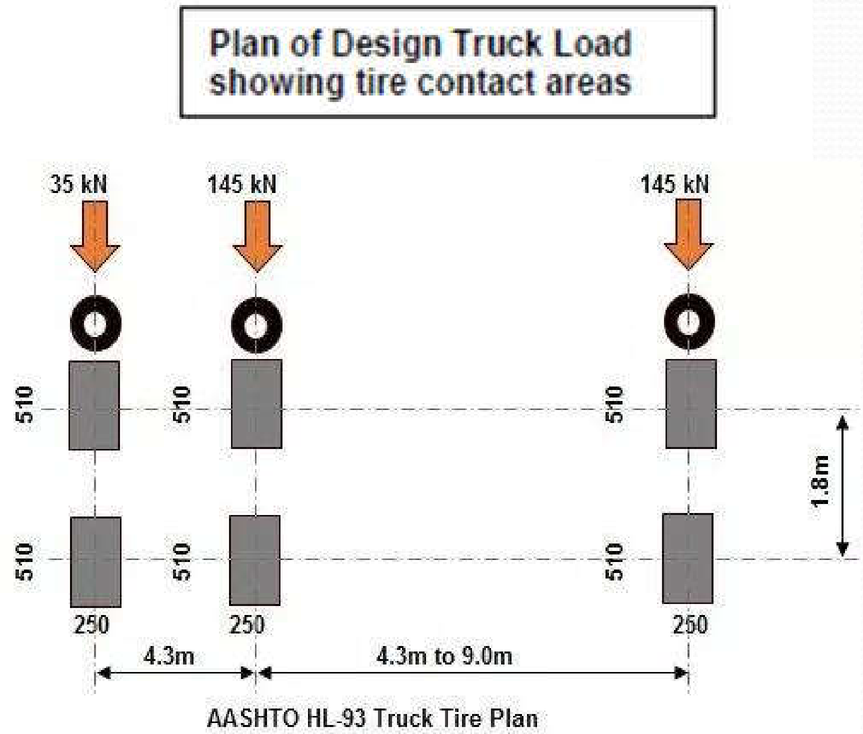
Design truck:- consists of three axles, front and two rear axles with front axle weighing 35 kN and two rear axles weighing 145 kN. The weights and spacing of axles and wheels for the design truck shall be as specified in Figure 1 (HS-20 Loading). The spacing between the two 145kN axles shall be varied between 4.3m and 9m to produce extreme force effects. A dynamic load allowance shall be considered.

ERA Bridge Design Manual 2013, Articles 3.6.1.3.1 and 3.6.1.4.1

Design tandem:- shall consist of a pair of 110kN axles spaced 1200mm apart. The transverse spacing of wheels shall be taken as 1800mm. A dynamic load allowance shall be considered as specified in ERA Bridge Design Manual 2013, Article 3.6.2.

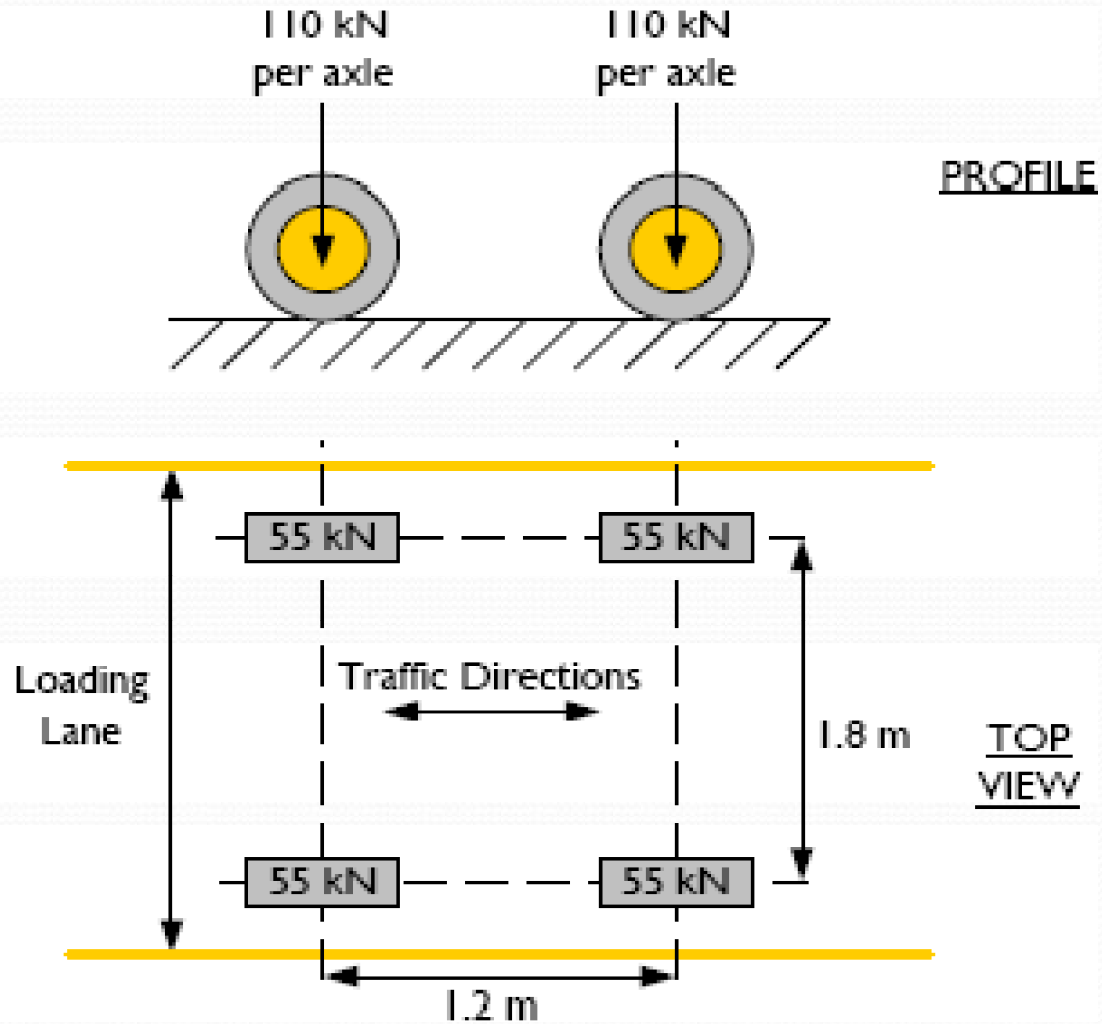
Design Truck Load





Design Truck Load

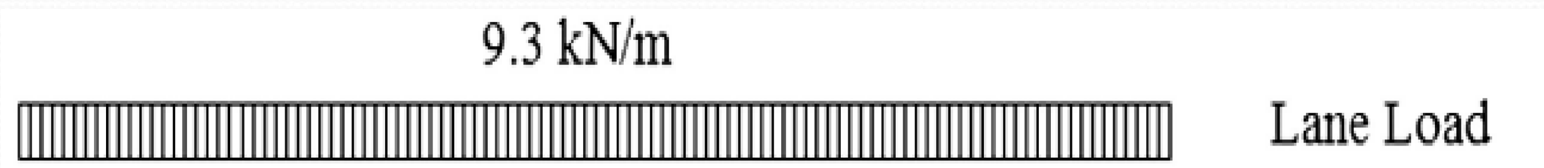
Design Tandem Load



Design Tandem Load

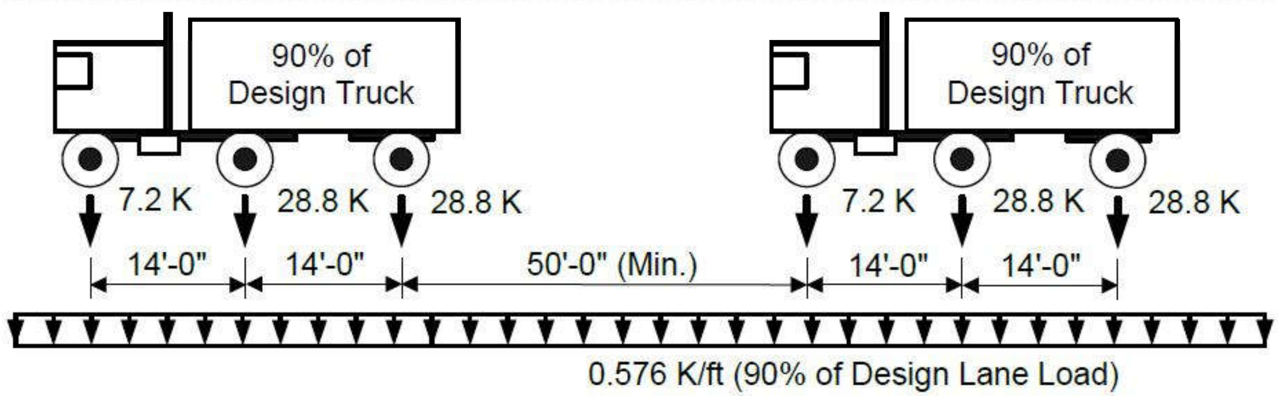
Design Lane Load:

- The design lane load shall consist of a load of **9.3kN/m,** uniformly distributed in the longitudinal direction.
- Transversely, the design lane load shall be assumed to be uniformly distributed **over a 3.0m width**.
- The force effects from the design lane load shall not be subject to a dynamic load allowance. (AASHTO, ERA Design manual)



Double Design Vehicular Live Loads

For negative moments and reactions at piers, the following condition is also considered. Two design trucks are applied, with a minimum headway between the front and rear axles of the two trucks equal to 15m. The rear axle spacing of the two trucks is set at a constant 4.3m. 90% of the effect of the two design trucks is combined with 90% of the design lane load. This loading is described in **LRFD** **[*AASHTO: Article 3.6.1.3.1*].**



WisDOT Bridge Manual, Jan 2019
State of Wisconsin

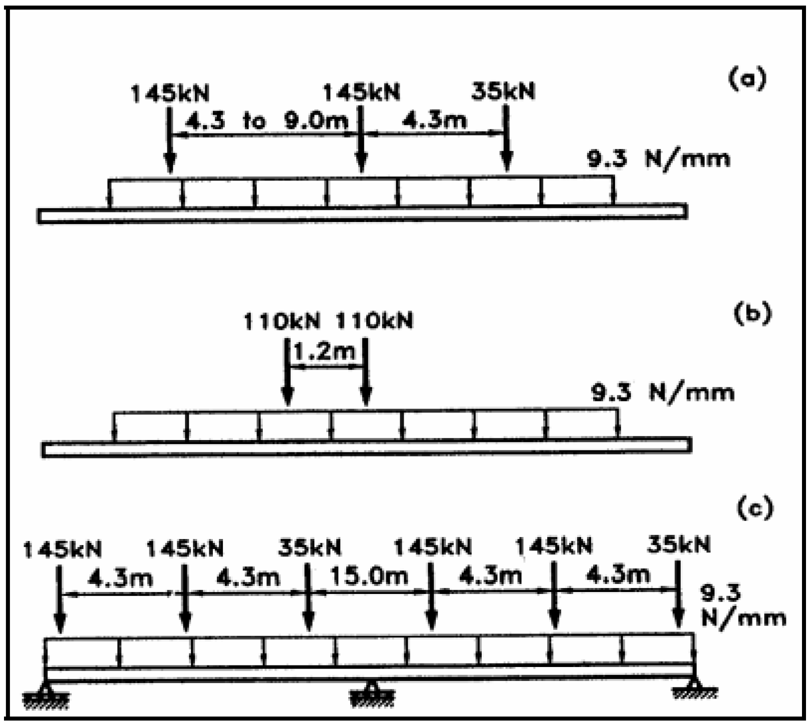
Application of Design Vehicular Live Loads

Unless otherwise specified, the extreme force effect shall be taken as the larger of the following:

(i) The effect of one design truck with the variable axle spacing specified in AASHTO Section 3.6.1.2.2, combined with the effect of the design lane load; or

(ii) The effect of the design tandem combined with the effect of the design lane load; and

(iii) For both negative moment between points of contraflexure under a uniform load on all spans and reaction at interior piers only, 90% of the effect of two design trucks spaced a minimum of 15.0m between the lead axle of one truck and the rear axle of the other truck, combined with 90% of the effect of the design lane load. The distance between the 145kN axles of each truck shall be taken as 4.3m.



AACRA BDM -2004 article 5.3.2(c)

Positions of Loads

The design truck or tandem shall be positioned transversely such that the center of any wheel load is not closer than:

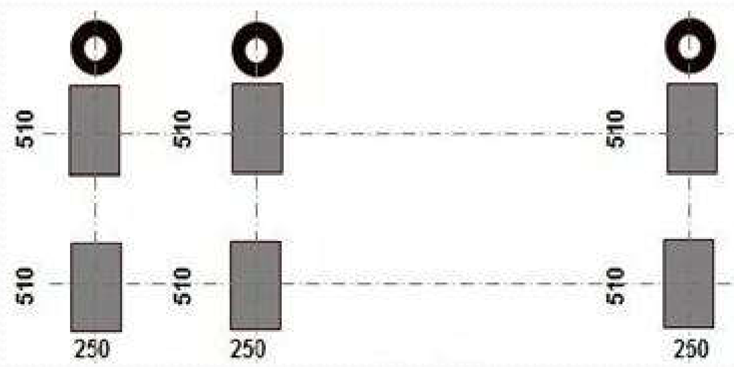
- For the design of the deck overhang -300 mm from the face of the curb or railing, and
- For the design of all other components -600 mm from the edge of the design lane.

Tire Contact Area (AASHTO Art. 3.6.1.2.5)

The tire contact area of a wheel consisting of one or two tires shall be assumed to be a single rectangle, whose width is 510 mm and whose length is 250 mm.

The tire pressure shall be assumed to be uniformly distributed over the contact area.

Axles that do not contribute to the extreme force effect under consideration shall be neglected.



Deck Overhang Load (AASHTO Art. 3.6.1.3.4)

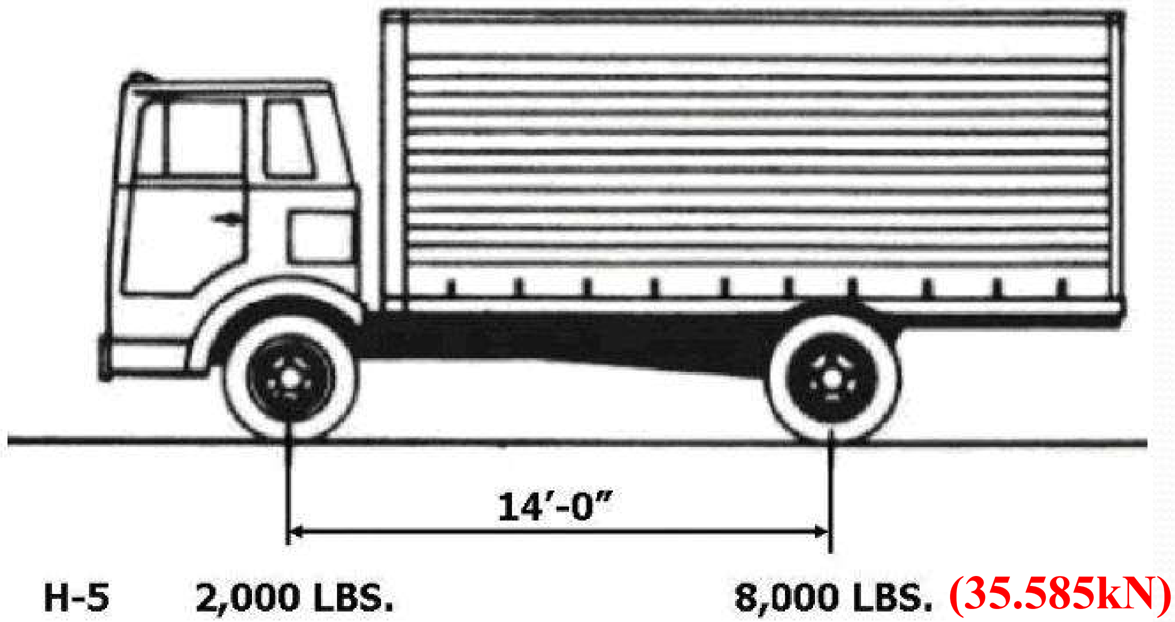
For the design of deck overhangs with a cantilever, not exceeding 1800 mm from the centerline of the exterior girder to the face of a structurally continuous concrete railing, the outside row of wheel loads may be replaced with a uniformly distributed line load of 14.6 N/mm intensity, located 300 mm from the face of the railing.

Fatigue Load (AASHTO Art. 3.6.1. 4)

The fatigue load shall be one design truck or axles thereof specified in Article 3.6.1.2.2, but with a constant spacing of 9000 mm between the 145 kN axles.

The dynamic load allowance specified in Article 3.6.2 shall be applied to the fatigue load.

Pedestrian live load



H5 Service Vehicle

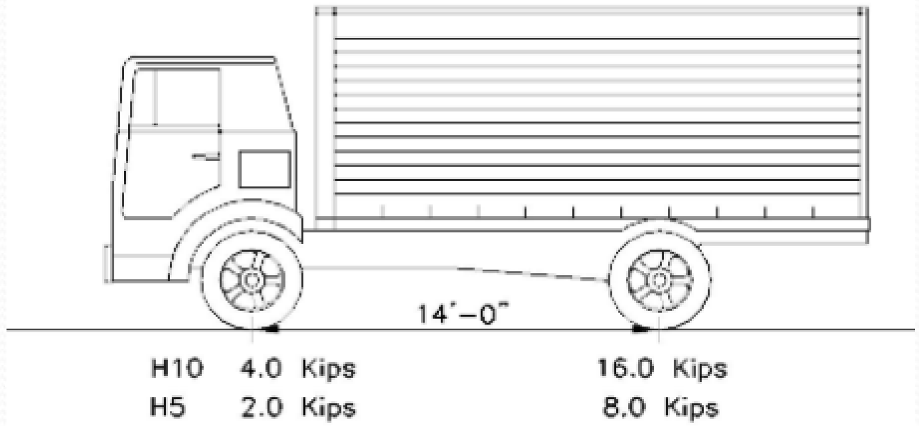
As detailed in *AASHTO LRFD Bridge Design Specifications*, with the walkway on the bridge being only 10 ft wide, the code recommends using an H5 design service vehicle. Further, the AASHTO code states that the service vehicle load is not applied in combination to the pedestrian live load.

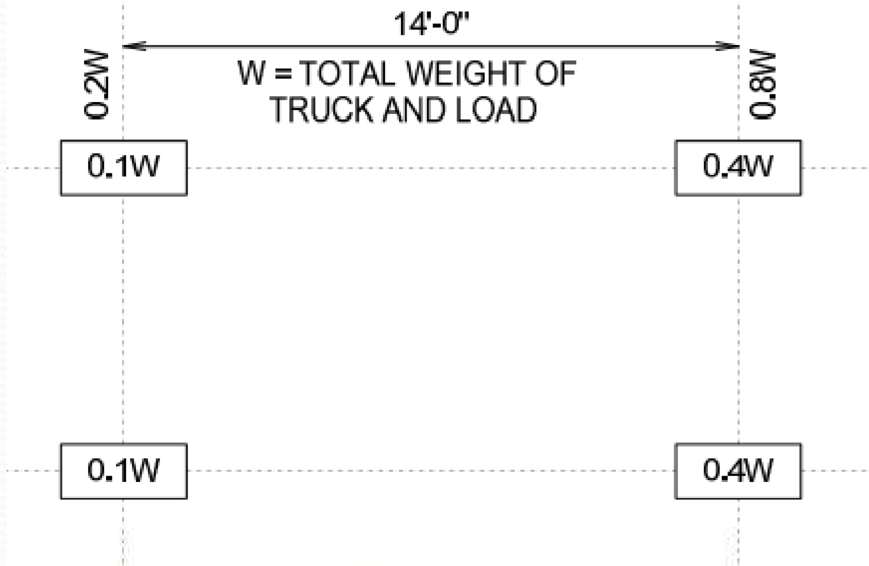
Pedestrian live load

Design Vehicle

Clear Deck With	Design Vehicle
7 to 10 feet	H5
Over 10 feet	H10

1000lbs=1kips= 4.4482kN
14'=4.3m
1'=0.307m





Maintenance Vehicle Configurations.

Pedestrian Bridges

Live Load of 50 psf

2.39kPa

For Clear Bridge Width of :

7 ft ≤ w ≤ 10 ft

Live Load of 150 psf

7.2kPa

For Clear Bridge Width of :

w > 10 ft

Pedestrian Bridges . . .

Clear bridge widths of less than 7 feet need not be designed for maintenance vehicles. [**Article 3.2**]

- The maintenance vehicle live load shall not be placed in combination with the pedestrian live load. [**Article 3.2**]
- Dynamic load allowance is not applied to the maintenance vehicle. [**Article 3.2**]
- Strength I Limit State shall be used for the maintenance vehicle loading. [**Article 3.2, 3.7**]

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State of Wisconsin

EQUESTRIAN LOAD (LL)

Decks intended to carry equestrian loading shall be designed for a patch load of 1.00 kips (4.448kN) over a square area measuring 4.0 inches (10cm) on a side. This loading includes dynamic factors.

APPLICATION OF LOADS

When determining the pattern of pedestrian live loading which maximizes or minimizes the load effect on a given member, the least dimension of the loaded area shall be greater than or equal to 2.0 ft.

Dynamic Load Allowance (IM)

The dynamic load allowance (***IM***) is an increment to be applied to the static wheel load to account for wheel load impact from moving vehicles.

AASHTO section 3.6.2

The static effects of the design **truck or tandem**, other than centrifugal and braking forces, shall be increased by the percentage specified in AASHTO Table 3.6.2.1-1 for dynamic load allowance.

Dynamic Load Allowance (IM)

Dynamic effects due to moving vehicles shall be attributed to two sources:

- Hammering effect is the dynamic response of the wheel assembly
- Dynamic response of the bridge as a whole

Dynamic load allowance need not be applied to:

- Retaining walls not subject to vertical reactions from the superstructure, and
- Foundation components that are entirely below ground level.
- The dynamic load allowance shall not be applied to pedestrian loads or to the design lane load.
- Dynamic load allowance need not be applied to wood components.

Dynamic Load Allowance……

- The factor to be applied to the static load shall be taken as: (**1 + IM/100**).

Dynamic Load Allowance, IM (AASHTO , Table 3.6.2.1-1 **)**

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

- The dynamic load allowance for **culverts** and other buried structures, in %, shall be taken as:

$$IM = 33\,(1.0 - 4.1 * 10^{-4}\, D_E) > 0\%$$

where: D_E = the minimum depth of earth cover above the structure (mm)

Indian IRC

No allowance for impact is to be made for foot bridges.

For road bridges shall be determined from the following equation:

For RC Bridges, $I = \dfrac{4.5}{(6+L)}$

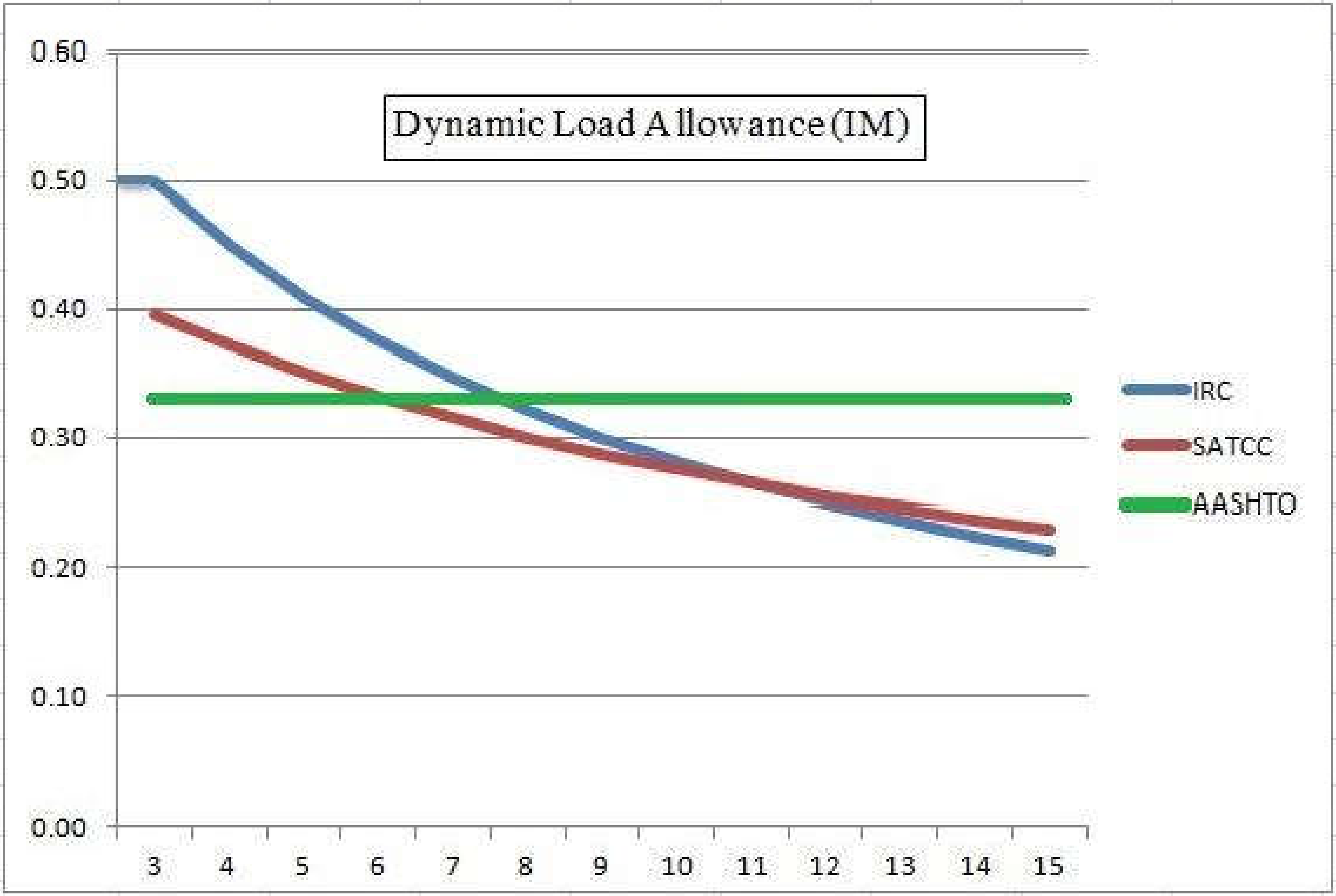
where: L is the span (m) ; $3 \leq L \leq 45$m

SATCC

NA Loading is a formula loading representing normal traffic consisting of the most severe arrangements of legal vehicles that are probable. It is based on South African legal loadings and an allowance for impact based on the Swiss (SIA Norm 160 - 1970) Highway Impact Factor

$$\Phi = 0.05\left(\frac{100 + L_s}{10 + L_s}\right)$$

where L_s is the equivalent span length in metres.



Dynamic Load Allowance (IM)
0.60
0.50
0.40
0.30
0.20
0.10
0.00
3
4
5
6
7
8
9
10
11
12
13
14
15
IRC
SATCC
AASHTO

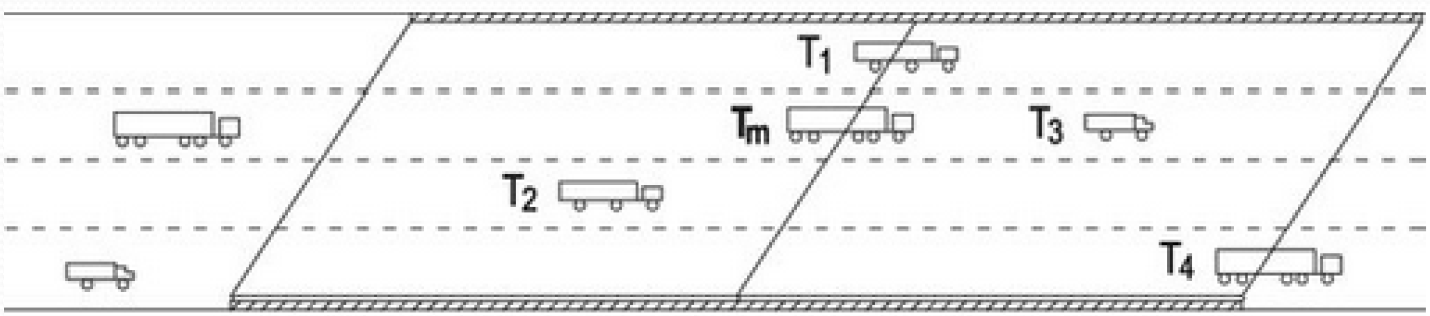
Multiple Presence of Live Load:

- Trucks will be present in adjacent lanes on roadways with multiple design lanes but this is unlikely that all adjacent lanes will be loaded simultaneously. This will be considered by the multiple presence factors.

- When the loading condition includes the pedestrian loads combined with one or more lanes of the vehicular live load, the pedestrian loads shall be taken to be one loaded lane.

AASHTO section 3.6.1.1.2

- To account for the probability of simultaneous lane occupation by the full design live load, reduction in load intensity is used.



Multiple Presence of Live Load:

Multiple Presence Factors (AASHTO, Table 3.6.1.1.2-1)

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

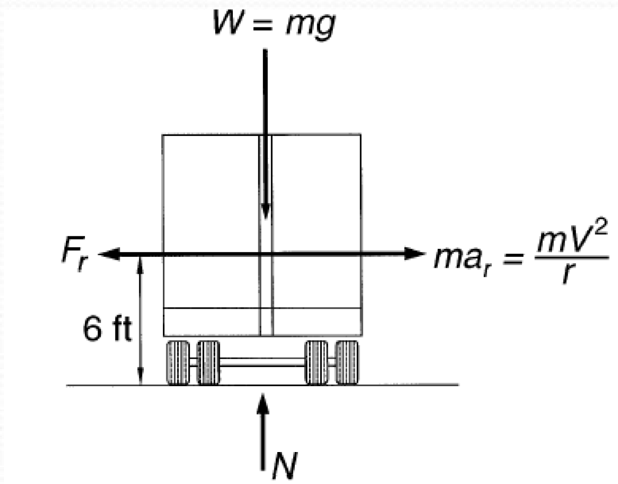
Number of Design Lanes

- Generally, the number of design lanes should be determined by taking the integer part of the ratio W/3600

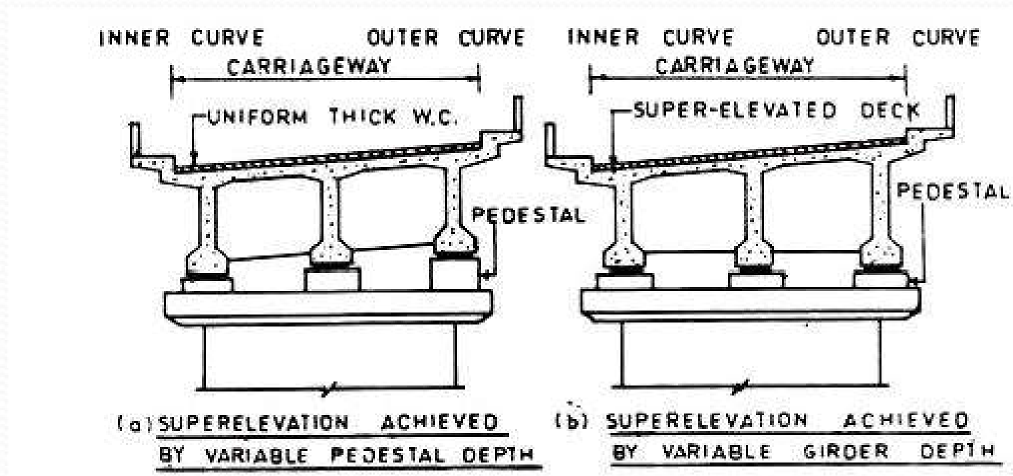
Where: w is the clear roadway width in mm between curbs and/or barriers

Centrifugal Forces *(CE= Vehicular Centrifugal Force):*

- Centrifugal force is due to inertia force of vehicles on curved bridges at speed. Centrifugal forces shall be applied horizontally at a distance **1.8 m** **above** the roadway surface.



Free-body diagram for centrifugal force



Method of providing superelevation in curved bridges

Centrifugal Forces . . .

Centrifugal forces shall be taken as the product of the axle weights of the design truck or tandem and the factor C, taken as:

$$C = \frac{4}{3}\frac{v^2}{g^*R}$$

where: v = highway design speed (m/s)

g = gravitational acceleration: 9.81 (m/s^2)

R = radius of curvature of traffic lane (m)

The multiple presence factors shall apply.

AASHTO Article 3.6.3

Breaking Force ***(BR= Vehicular Braking Force):***

- From AASHTO Commentary 3.6.4 Based on energy principles, and assuming uniform deceleration (retardation), the braking force determined as a fraction "b" of vehicle weight.

  According to AASHTO Article 3.6.4 the braking force shall be taken as the greater of:
  - 25 % of the axle weights of the design truck or design tandem or,
  - 5 % of the design truck plus lane load or 5 % of the design tandem plus lane load

- These forces shall be assumed to act horizontally at a distance of 1800 mm above the roadway surface in either longitudinal direction to cause extreme force effects.

Vehicular Collision *(CT= Vehicular Collision Force):*

- Unless protections are provided a horizontal force of 1800kN applied at 1.2m above the ground should be considered.

AASHTO Article 3.6.5

Pedestrian Loads

- A pedestrian load of **3.6kPa** shall be applied to all sidewalks wider than 0.6m and considered simultaneously with the vehicular design live load.

Water Loads

- **Water Loads** (WA= Water Load and Stream Pressure)
  - **Static Pressure**: Static pressure of water shall be assumed to act perpendicular to the surface that is retaining the water. AASHTO Article 3.7.1

    $$P = \gamma\, g\, h$$

  - **Buoyancy :** Buoyancy shall be considered to be an uplift force, taken as the sum of the vertical components of static pressures, as specified in Article 3.7.1, acting on all components below design water level.

Stream Pressure

- **Longitudinal:** The longitudinal drag force shall be taken as the product of longitudinal stream pressure and the projected surface exposed thereto.

$$p = 5.14 * 10^{-4}\, C_D V^2$$

Where: p = pressure of flowing water (MPa)

C_D = drag coefficient for piers

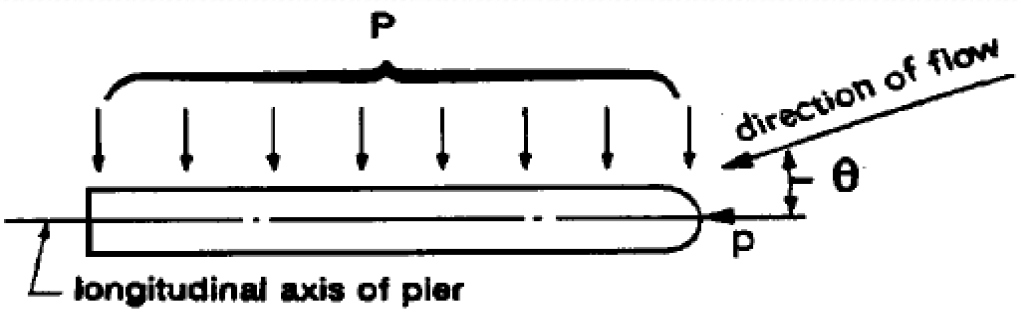
V = design velocity in m/s of water for the design flood

Table: Drag Coefficient

Type	C_D
Semicircular-nosed pier	0.7
Square-ended pier	1.4
Debris lodged against the pier	1.4
Wedged-nosed pier with nose angle 90° or less	0.8

- **Lateral:** The lateral, uniformly distributed pressure on substructure due to water flowing at an angle, θ, to the longitudinal axis of the pier.

$$P_L = 5.14 \times 10^{-4} C_L V^2$$



Lateral Drag Coefficient

Angle, θ,	C_L
0°	0.0
1°	0.5
10°	0.7
20°	0.9
≥30°	1.0

Wind Loads

- **Wind Pressure on Structures, (WS):** For small and medium sized concrete bridges below 50m length the wind load on structures shall be neglected.
- In the absence of more precise data, design wind pressure, P_D in kPa, shall be determined as:

AASHTO section 3.8.1.2.1

$$P_D = P_B\left[\frac{V_{DZ}}{V_B}\right]^2$$

Where P_B = base wind pressure

VDZ = design velocity of wind at design elevation, Z (Km/hr)

VB= Base wind velocity (Km/hr)

Wind Pressure on Vehicles: WL

(AASHTO Art. 3.8.1.3)

When vehicles are present, the design wind pressure shall be applied to both structure and vehicles. Wind pressure on vehicles shall be represented by an interruptible, moving force of 1.46 N/mm acting normal to, and 1800 mm above, the roadway and shall be transmitted to the structure.

Table 3.8.1.3-1 Wind Components on Live Load.

Skew Angle	Normal Component	Parallel Component
Degrees	N/mm	N/mm
0	1.46	0.00
15	1.28	0.18
30	1.20	0.35
45	0.96	0.47
60	0.50	0.55

Aeroelastic Instability: WL
(AASHTO Art. 3.8.3)

Aeroelastic force effects shall be taken into account in the design of bridges and structural components apt to be wind-sensitive. For the purpose of this Article, all bridges, and structural components thereof with a span length to width or depth ratio exceeding 30.0 shall be deemed to be wind-sensitive.

The vibration of cables due to the interaction of wind and rain shall also be considered.

Earthquake Effects (EQ= Earthquake)

- Earthquake loads are given by the product of the elastic seismic response coefficient C_{sm} and the equivalent weight of the superstructure.

Minimum Analysis Requirements for Seismic Effects

Seismic Zone	Single-Span Bridges	Multispan Bridges					
		Other Bridges		Essential Bridges		Critical Bridges	
		Regular	Irregular	Regular	Irregular	Regular	Irregular
1-3	No Seismic Analysis	*	*	*	*	*	*
4	Seismic Analysis	SM/UL	SM	SM/UL	MM	MM	MM

* = no seismic analysis required (Zone 1-3)

Where:

UL = uniform load elastic method

SM = single-mode elastic method

MM = multimode elastic method

Earthquake Effects (EQ= Earthquake)

- Except as specified below, bridges satisfying the requirements of Table 4.7.4.3.1-2 may be taken as “regular” bridges. Bridges not satisfying the requirements of Table 4.7.4.3.1-2 shall be taken as “irregular” bridges.

- Curved bridges comprised of multiple simple-spans shall be considered to be “irregular” if the subtended angle in plan is greater than 20 degrees. Such bridges shall be analyzed by either the multimode elastic method or the time-history method.

Table 4.7.4.3.1-2—Regular Bridge Requirements

Parameter	Value				
Number of Spans	2	3	4	5	6
Maximum subtended angle for a curved bridge	90°	90°	90°	90°	90°
Maximum span length ratio from span to span	3	2	2	1.5	1.5
Maximum bent/pier stiffness ratio from span to span, excluding abutments	—	4	4	3	2

Earth Pressure

- **Earth Pressure** (EH = Horizontal Earth Pressure; ES = Earth Surcharge; LS = Live Load Surcharge; DD = Down drag)

Earth pressure shall be considered as a function of the:

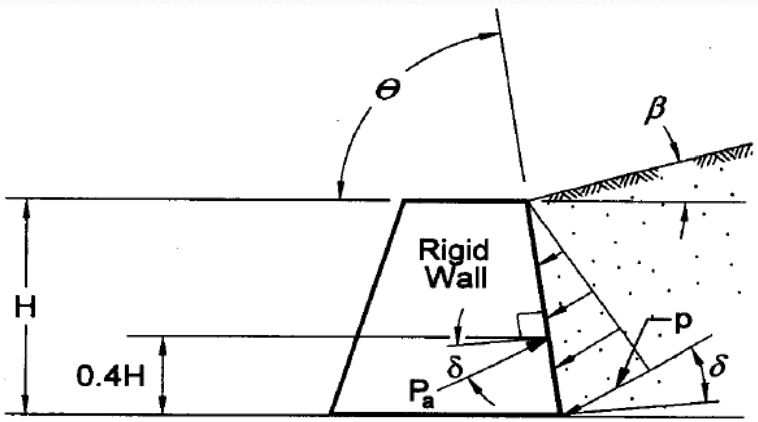
- Type and density of earth,
- Water content,
- Soil creep characteristics,
- Degree of compaction
- Location of groundwater table,
- Earth-structure interaction,
- Amount of surcharge, and
- Earthquake effects.

EH = Horizontal Earth Pressure

Active and Passive Earth Pressures

- Coulomb theory is recommended by AASHTO for masonry and RC abutment since this theory holds better for the actual situation.

k_h=coefficient of lateral earth pressure taken as k_o



$$k_a = \frac{\sin^2(\theta + \varphi')}{\Psi * \sin^2\theta \sin(\theta - \delta)}$$

Where:

$$\Psi = \left[1 + \sqrt{\frac{\sin(\varphi' + \delta)\sin(\varphi' - \beta)}{\sin(\theta - \delta)\sin(\theta + \beta)}}\right]^2$$

Where: δ = friction angle between fill and wall

β = angle of fill to the horizontal

θ = angle of backfill of wall to the vertical

φ = effective angle of internal friction (°)

ES = Earth Surcharge; LS = Live Load Surcharge

Where a uniform surcharge is present, a constant horizontal earth pressure, Δ_p (MPa), shall be added to the basic earth pressure. This constant earth pressure shall be taken as:

$$\Delta_p = ks\ q_s$$

Where: k_s = coefficient of earth pressure due to surcharge
q_s = uniform surcharge applied to the upper surface of the active earth wedge (MPa)

live load surcharge shall be applied where vehicular load is expected to act on the surface of the backfill within a distance equal to the wall height behind the back face of the wall.

The increase in horizontal pressure due to live load surcharge shall be estimated as:

$$\Delta_p = k{*}\gamma_s{*}g{*}h_{eq}$$

where: Δ_p = constant horizontal earth pressure due to uniform surcharge (MPa)
γ_s = density of soil (kg /m^3)
k = coefficient of earth pressure
h_{eq} = equivalent height of soil for the design truck (mm)

- The “Wall Height” shall be taken as the distance between the surface of the backfill and the bottom of the footing.

Wall Height (mm)	h_{eq} (mm)
≤1500	1700
3000	1200
6000	760
≥9000	610

Down Drag (DD):

- When soil surrounding piles settle, it applies a downward force. In this case, the force should be considered.

Design Objectives

The objectives in a bridge design are:
safety, serviceability, economy, constructability and *aesthetics.*

- ***Safety*** – the primary responsibility of the Engineer is to ensure public safety in the design by ensuring adequate structural safety.
- ***Serviceability*** – consists of satisfying requirements of deformation, durability, inspect ability, maintainability and ride ability.

Minimum requirements are provided for *clearances, environmental protection ,geological studies, rideability, durability, inspectibility and maintainability.*

Design Philosophy

- In Engineering design the general principle is that the resistance of a cross section has to exceed the effects come from the applied loads.
- When a particular loading condition reaches and just exceeds the resistance capacity of the provided section failure is the result. Such a condition is referred to as a Limit State.
- **Load and Resistance Factor Design (LRFD)-** A reliability-based design methodology in which force effects caused by factored loads are not permitted to exceed the factored resistance of the components.

Resistance ≥Effect of Loads

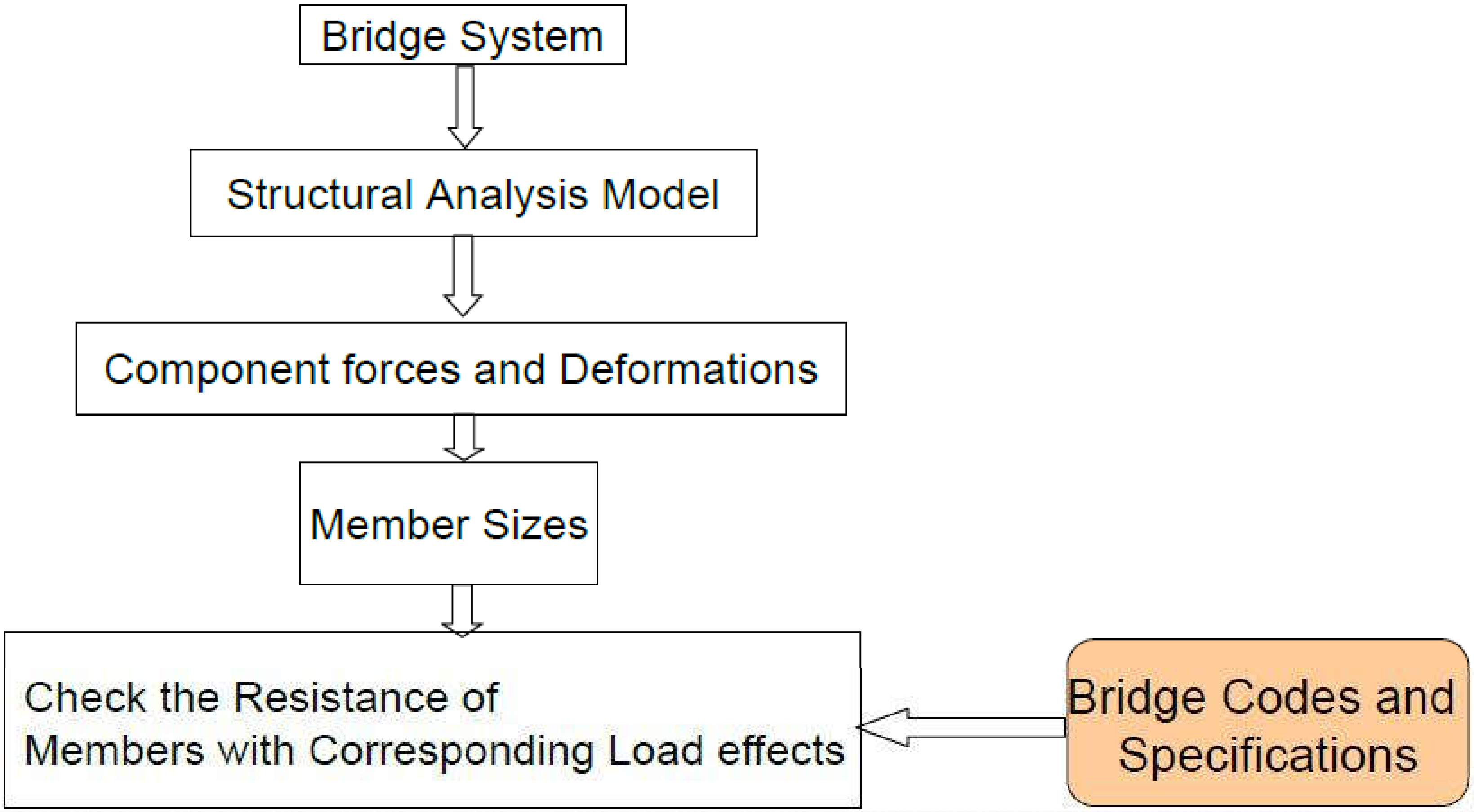
$$\boxed{\Phi * Rn \geq \sum \gamma i * Qi}$$

Design Philosophy ... Cont'd

The resistance factor Φ for a particular limit state must account for the uncertainties in

- Material properties
- Equations that predict strength
- Workmanship
- Quality control
- Consequence of a failure

Process of Design



Limit States

Structural components shall be proportioned to satisfy the requirements at all appropriate service, fatigue, strength, and extreme event limit states.

Service Limit State:

- Deals with restrictions on stress, deformation, and crack width under regular service conditions.
- Intended to ensure that the bridge performs acceptably during its design life.

Strength Limit State :

- Intended to ensure that strength and stability are provided to resist statistically significant load combinations that a bridge will experience during its design life.
- Extensive distress and structural damage may occur at strength limit state conditions, but overall structural integrity is expected to be maintained.

Strength Limit State

Violation of the strength limit state occurs when safety of the structure is endangered through:

- Unlimited deformation
- Overturning
- Instability

Extensive distress and structural damage may occur but overall structural integrity is expected to be maintained.

Service Limit State

The service limit state shall be taken as restrictions on stress deformations and crack width, under regular service conditions. The design level of these actions is chosen so that they:

- Do not make the bridge unfit for use
- Do not cause public concern
- Do not significantly reduce the service life of the bridge

Fatigue and Fracture Limit State

The fatigue limit state shall be taken as restrictions on stress range as a result of a single design truck occurring at the number of expected stress range cycles.

The fatigue limit state is intended to limit crack growth under repetitive loads to prevent fracture during the design life of the bridge.

Limit States

Fatigue Limit State :

- Deals with restrictions on stress range under regular service conditions reflecting the number of expected cycles.

- Fatigue limit states are used to limit stress in steel reinforcements to control concrete crack growth under repetitive truck loading in order to prevent early fracture failure before the design service life of a bridge.

Extreme Event Limit State :

- Intended to ensure structural survival of a bridge during an earthquake, vehicle collision, ice flow, or foundation scour.

Load Factors and Load Combinations

- In LRFD method, load factors are applied to the loads and resistance factors to the internal resistances or capacities of sections.
  - ***Load Factor-*** A statistically-based multiplier applied to force effects accounting primarily for the variability of loads, the lack of accuracy in analysis, and the probability of simultaneous occurrence of different loads, but also related to the statistics of the resistance through the calibration process.
  - ***Load Modifier-*** A factor accounting for ductility, redundancy, and the operational classification of the bridge.

Load Factors and Load Combinations

- Moreover, the load combinations and load factors are considered accordingly.

$$Q = \Sigma \eta_i \gamma_i Q_i$$

Where:

η_i= load modifier
Q_i = force effects from loads
γ_i = load factors specified
Rn= is resistance

: load modifier specified in AASHTO Article 1.3.2

: load factors specified in AASHTO Tables 3.4.1-1 and 3.4.1-2

Load Combination / Limit State	DC DD DW EH EV ES	LL IM CE BR PL LS EL	WA	WS	WL	FR	TU CR SH	TG	SE	Use one of these at a time	
										EQ	CT
STRENGTH 1 (Unless noted)	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	0.50	1.0	1.00	0.50/1.20	γ_{TG}	γ_{SE}	-	-
EXTREME EVENT I	γ_p	γ_{EQ}	1.00	-	-	1.00	-	-	-	1.00	-
SERVICE I	1.00	1.00	1.00	0.30	1.0	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 and CE ONLY	-	0.75	-	-	-	-	-	-	-	-	-

Strength Limit State

Strength I: This strength limit state is the basic load combination relating to normal vehicular use of the bridge without wind.

Strength II: This strength limit state is the load combination relating to the use of the bridge by owner-specified special design vehicles, evaluation permit vehicles, or both without wind.

Strength III: This strength limit state is the load combination relating to the bridge exposed to wind velocity exceeding 55 mph (90 km/h).

Strength IV: This strength limit state is the load combination relating to very high dead/live load force effect ratios.

Strength V: This strength limit state is the load combination relating to normal vehicular use of the bridge with wind of 55-mph (90-km/h) velocity.

Extreme Event Limit State

The extreme event limit state refers to the structural survival of a bridge during a major earthquake or flood or when collided by a vessel, vehicle, or ice

Extreme Event I: This extreme event limit state is the load combination relating to earthquake.

Extreme Event II: This extreme event limit state is the load combination relating to ice load, collision by vessels and vehicles, and to certain hydraulic events with reduced live load.

Service Limit States

The service limit state refers to restrictions on stresses, deflections, and crack widths of bridge components that occur under regular service conditions.

Service I: refers to the load combination relating to the normal operational use of the bridge with 55-mph (90- km/h) wind, and with all loads taken at their nominal values.

Service II: refers to the load combination relating only to steel structures and is intended to control yielding and slip of slip-critical connections due to vehicular live load.

Service III: refers to the load combination for longitudinal analysis relating to tension in prestressed concrete superstructures with the objective of crack control and to principal tension in the webs of segmental concrete girders.

Service IV: This service limit state refers to the load combination relating only to tension in prestressed concrete substructures with the objective of crack control.

Fatigue and Fracture Limit State

The fatigue and fracture limit state refers to a set of restrictions on stress range caused by a design truck.

The restrictions depend on the number of stress-range excursions expected to occur during the design life of the bridge

Load Modifiers

Load Modifiers, η_i $= \eta_D \eta_R \eta_I$:

η_D = a factor relating to ductility,

η_R = a factor relating to redundancy

η_I = a factor relating to operational importance

Ductility, redundancy, and operational importance are significant aspects affecting the margin of safety of bridges.

Table 3.4.1-2—Load Factors for Permanent Loads, γ_p

Type of Load, Foundation Type, and Method Used to Calculate Downdrag		Load Factor	
		Maximum	Minimum
DC: Component and Attachments		1.25	0.90
DC: Strength IV only		1.50	0.90
DD: Downdrag	Piles, α Tomlinson Method	1.4	0.25
	Piles, λ Method	1.05	0.30
	Drilled shafts, O'Neill and Reese (1999) Method	1.25	0.35
DW: Wearing Surfaces and Utilities		1.50	0.65
EH: Horizontal Earth Pressure			
• Active		1.50	0.90
• At-Rest		1.35	0.90
• *AEP* for anchored walls		1.35	N/A
EL: Locked-in Construction Stresses		1.00	1.00
EV: Vertical Earth Pressure			
• Overall Stability		1.00	N/A
• Retaining Walls and Abutments		1.35	1.00
• 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 and Structural Plate Culverts with Deep Corrugations		1.50	0.90
ES: Earth Surcharge		1.50	0.75

Load comb. SA

Type of action			Nominal action F_k		Notation	Clause No	Limit state	$\gamma_{fL} = \gamma_{f1} \times \gamma_{f2}$ to be considered in combinations: 1	2	3
actions	Principal actions	Permanent and long term	Dead loads: concrete		g_c	2.2.2	ULS SLS	1.2 1.0	1.05 1.0	1.2 1.0
			Dead loads: steel		g_s	2.2.2	ULS SLS	1.1 1.0	1.0 1.0	1.1 1.0
			Superimposed dead loads		g_d	2.2.2	ULS SLS	1.2 1.0	1.05 1.0	1.2 1.0
			Reduced load factor for dead and superimposed dead load where this has a more severe effect		g_R	2.2.2.2 2.3.2.2	ULS SLS	1.0 1.0	1.0 1.0	1.0 1.0
			Vertical earth loading on culverts	Method (i)	g_n	2.3.3.3	ULS SLS	1.5 1.1	1.3 1.1	1.5 1.1
				Methods (ii) & (iii)	g_n	2.3.3.3	ULS SLS	1.4 1.0	1.2 1.0	1.4 1.0
			Earth pressure due to retained fill	Approximate theory	f_{ep}	2.4.3	ULS SLS	1.5 1.1	1.3 1.1	1.5 1.1
				More accurate theory	f_{ep}	2.4.3	ULS SLS	1.4 1.0	1.2 1.0	1.4 1.0
			As above but causing relieving effect		f_{ep}	2.4.3.1	ULS SLS	See Clause 2.4.3.1		
			Water pressure of retained or excluded water		f_w	2.5.2	ULS SLS	1.2 1.0	1.05 1.0	1.2 1.0
			As above but causing relieving effect		f_w	2.5.2	ULS SLS	1.0 1.0	1.0 1.0	1.0 1.0
		Transient and variable	Primary live: Vehicle traffic loading and surcharge	NA Load	Q_a	2.6.3.3	ULS SLS	1.5 1.0	1.3 1.0	-
				NB Load	Q_b	2.6.4.3	ULS SLS	1.2 1.0	1.1 1.0	-
				NC + b NA on separate carriageways	Q_c	2.6.5.3	ULS SLS	1.2 1.0	1.1 1.0	-

Ductility Factor: η_D

Ductility is important to the safety of the bridge.

If ductility is present overloaded portion of the structure can redistribute the load to other portions that have reserve strength.

- This redistribution is dependent on the ability of the overloaded component and its connections to develop inelastic deformations without failure.
- Brittle behavior is to be avoided, because it implies a sudden loss of load carrying capacity when the elastic limit is exceeded.

Ductility

For the strength limit state:

$\eta_D \geq 1.05$ for non-ductile components and connections

$= 1.00$ for conventional designs and details complying with AASHTO

≥ 0.95 for components and connections for which additional ductility-enhancing measures have been specified beyond those required by AASHTO

For all other limit states:

$\eta_D = 1.00.$

Redundancy Factor: η_R

- A statically indeterminate structure is redundant, that is, it has more restraints than necessary to satisfy conditions of equilibrium.
- Redundancy in a bridge system will increase its margin of safety

Redundancy

For the strength limit state:

$\eta_R \geq 1.05$ for non redundant members

$= 1.00$ for conventional levels of redundancy, foundation elements where ϕ already accounts for redundancy as specified in ASSHTO Article 10.5

≥ 0.95 for exceptional levels of redundancy beyond girder continuity and a torsionally-closed cross-section.

For all other limit states:

$\eta_R = 1.00$

Operational Importance Factor: η_i

- Bridges can be considered of operational importance if they are on the shortest path between residential areas and a hospital or a school or provide access for police, fire, and rescue vehicles to homes, businesses, industrial plants, etc.
- a non important bridge could be on a secondary road leading to a remote recreation area, that is not open year around.
- In the event of an earthquake, it is important that all lifelines, such as bridges remain open.

Operational Importance

For the strength limit state:

$$\begin{aligned} \eta_I &\geq 1.05 && \text{for critical or essential bridges} \\ &= 1.00 && \text{for typical bridges} \\ &\geq 0.95 && \text{for relatively less important bridges.} \end{aligned}$$

For all other limit states:

$$\eta_I = 1.00$$

Ultimate Design Load

- The ultimate design load acting on a member will be the summation of the relevant characteristic load combinations multiplied by their respective partial safety factors.

$$Q = \Sigma \eta_i \gamma_i Q_i$$

Where:

η_i= load modifier
Q_i = force effects from loads
γ_i = load factors specified
Rn= is resistance

Analysis

Static Analysis

Slab Bridges

- Equivalent Strip Method is used for the analysis

Girder Bridges

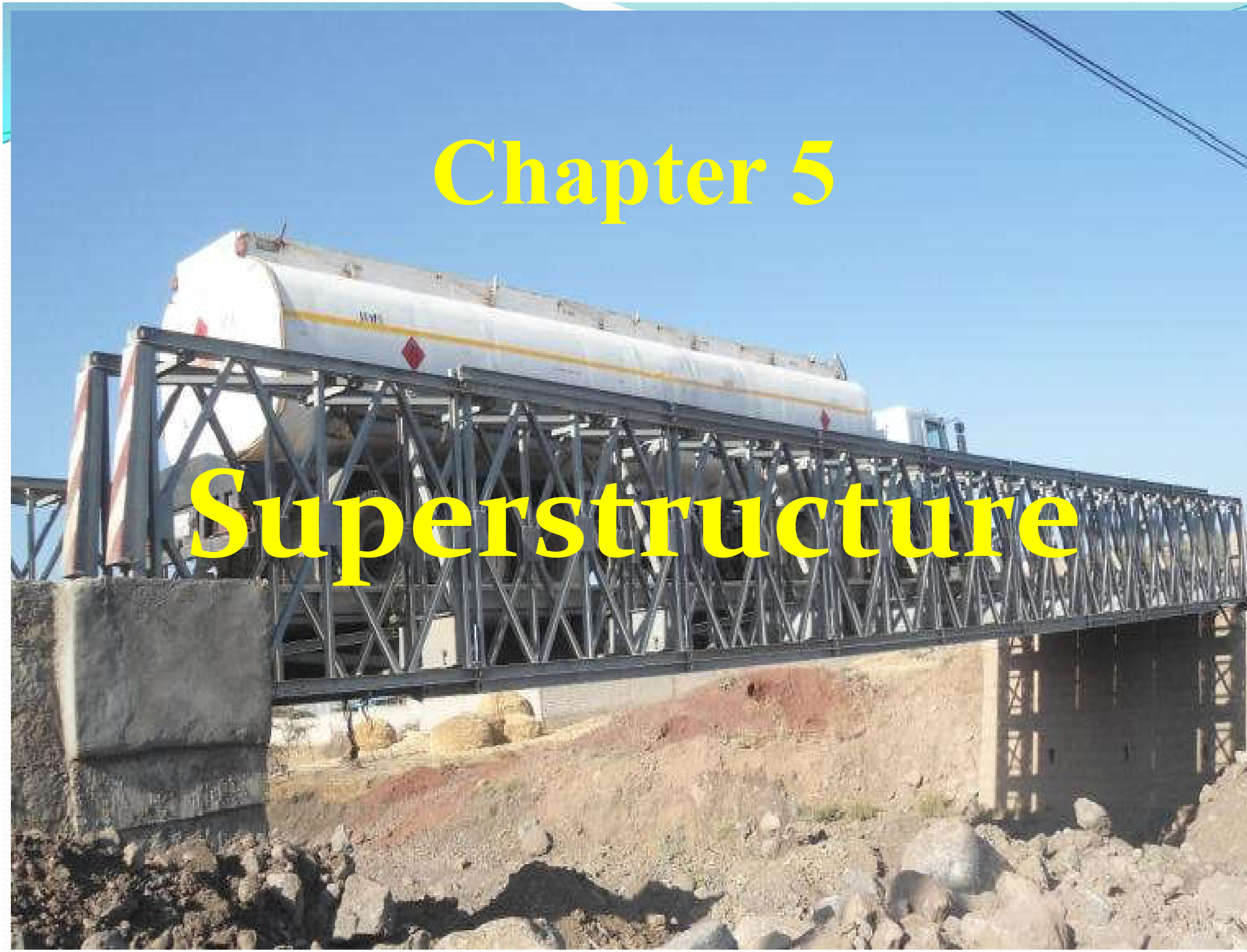
- Variations of loads along the transversal and longitudinal directions are used

Dynamic Analysis

Seismic Design per **ES- EN 1998:2015 (Design of Structures for EQ resistance)**, in conjunction with AASHTO, ERA seismic design criteria.

Chapter 5

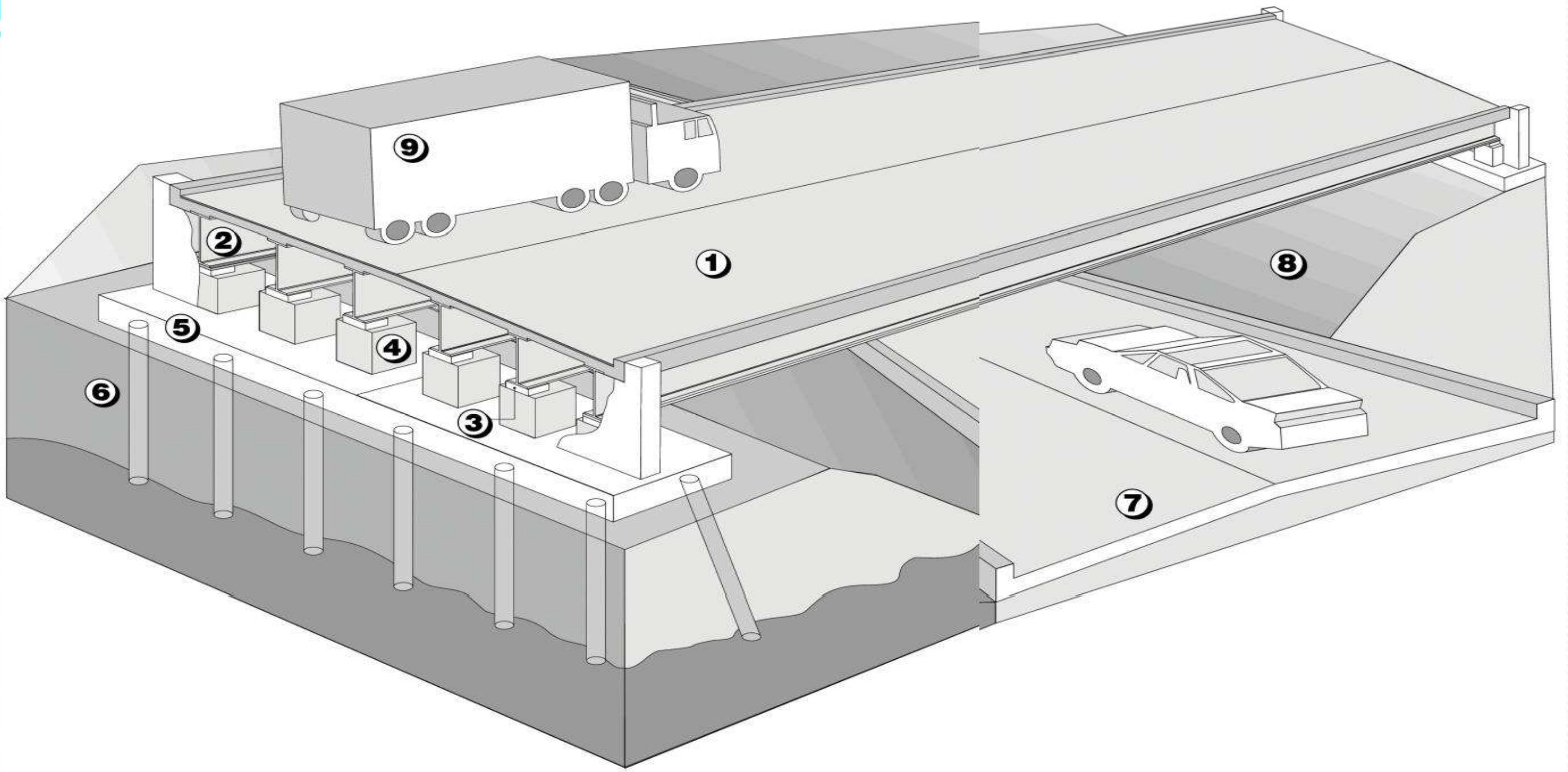
Superstructure



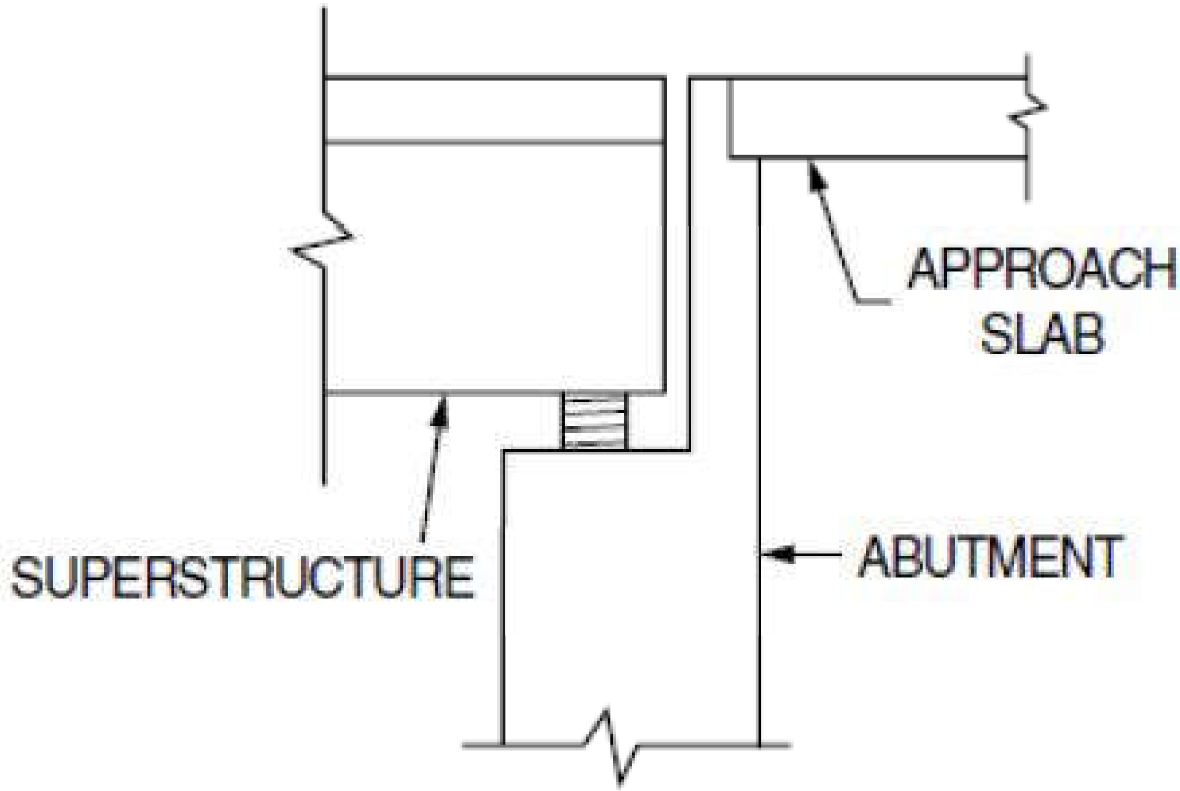
Superstructure

The superstructure of a bridge is made up of the portion of the bridge built on top of the substructure and supports the bridge deck. Several materials and structural configurations can be used to make up the superstructure of a bridge.

A bridge superstructure is an integrated body of various members of reinforced concrete, prestressed concrete, steel, composite, diaphragms, trusses, arches, etc. Determination of' forces in theses components is essential for design purposes.

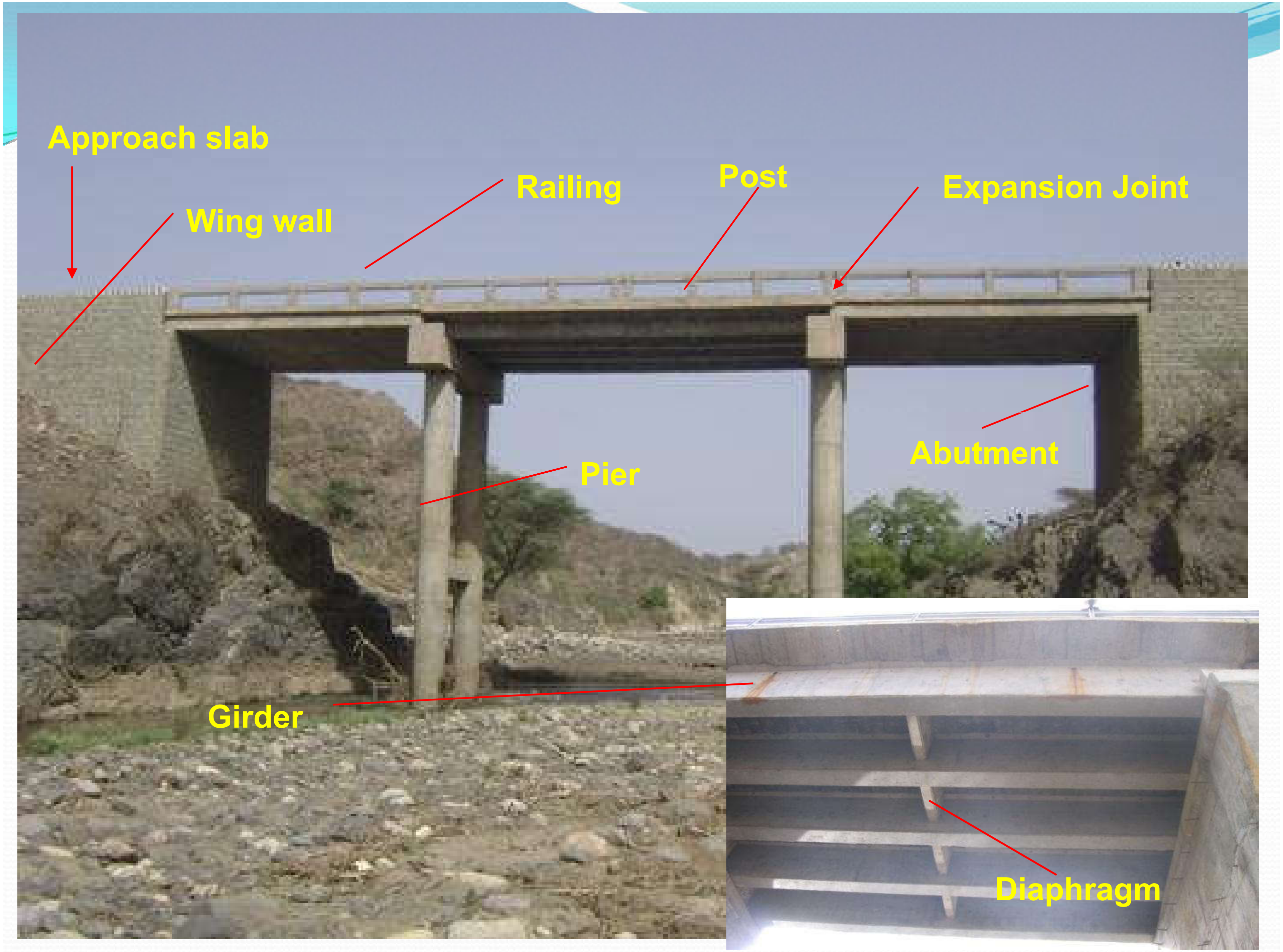


1-Deck and overpass   2- Stringer(longitudinal beams)   3-Bearing
4- Pedestal   5-Footing   6- Piles
7-Underpass   8- Embankment   9- Live load



Approach: Section of roadway immediately before and after the structure.

Approach slab: used to prevent settlement of the approach pavement.


Approach slab
Railing
Post
Expansion Joint
Wing wall
Abutment
Pier
Girder
Diaphragm

Reinforced Concrete Bridges

RC bridges possess several advantages over steel bridges

- adaptability of concrete wide variety of structural shapes
- Low maintenance cost
- Long life and better resistance to temporary overloads and dynamic loads than steel bridges.
- Cast-in-place reinforced concrete structures are continuous and monolithic, which translate into easy construction, low cost and good seismic resistance.
- They can also be given the desired aesthetic appearance.

Reinforced Concrete Bridges:

Disadvantages

- large dead weight (which require larger foundation)
- difficulty to widen
- Concrete is susceptible to cracking
- longer construction time
- Long curing time (Concrete attains specified compressive strength in 28-days after casting and curing.)
- requires formwork and false work
- Demands Strict Quality Control (Concrete demands strict quality control and skilled labor during mixing, placing and curing of concrete.)

Advantages of Continuous RC Bridges:

- Less number of bearings than simply supported bridge since one line of bearings is used over the piers
- Reduced width of pier, thus less obstruction to flow and as such possibility of less scour.
- Due to reduction in the width of pier,less amount of material is required
- Due to reduction in the width of pier, less number of expansion joints due to which both the initial cost and maintenance cost become less. The rigidity quality over the bridge is thus improved.

Advantages of Continuous RC Bridges . . .

- Lesser depth of girder, hence economical supports
- Better architectural appearance
- Lesser vibration and deflection

Disadvantages

- Not suitable on yielding foundations
- Differential settlement may cause undesirable stresses.

Steel Bridges

Steel bridge construction consists of rolled steel beams, plate girders or trusses with reinforced concrete deck or steel plate deck-beam bridges.

Steel has got several advantages

- It is a high quality, homogeneous, isotropic material that is perfectly elastic to its yield point.
- It has high tensile and compressive strengths.

Steel Bridges

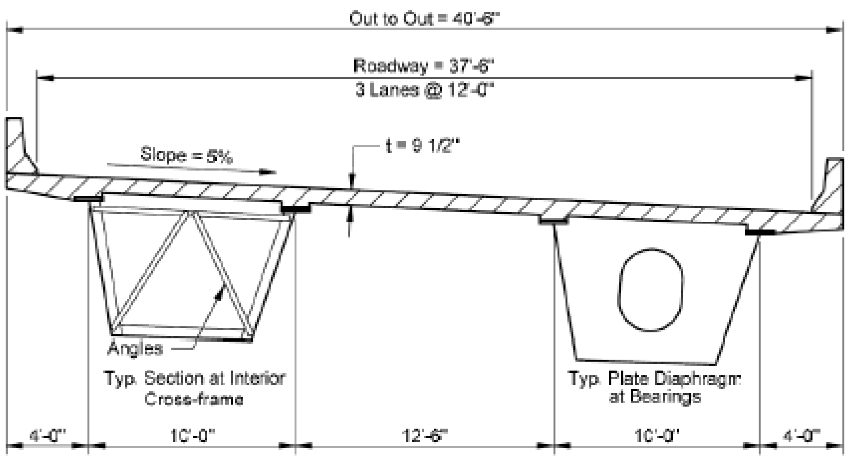
Advantages ....

- Faster construction time
- They can be erected with ease and this minimizing construction costs.
- Steel superstructures are usually lighter than concrete superstructures which translate into reduced substructures costs

Disadvantages :

- Corrosion of steel
- high maintenance cost
- Corrosion can reduce cross section of structural members and weaken the superstructure also.

Steel Bridges



Steel Bridges

- Secondary Members:

Bracing between primary members designed to resist cross-sectional deformation of the superstructure frame and help distribute part of the vertical load between stringers.

Refer: CENG 4123-Design of Steel Structures

Arch Bridges

- Arches are generally characterized by the development of inclined rather than vertical reactions under vertical loads.
- Cross-sections are designed for thrust, moment and shear, with magnitudes depending on the location of the pressure line.
- The most suitable for crossing valley, with the arch foundations located on dry rock slopes.
- The erection problem varies, being easiest for the cantilever arch
- A compression structure
- Aesthetically, the most successful of all bridge types

Analysis and Design of RC Slab Bridges

RC Slab Bridges

- Longitudinally reinforced slab bridges have the simplest superstructure configuration and the neatest appearance.
- They generally require more reinforcing steel and structural concrete than do girder-type bridges of the same span.
- However, the design details and formworks are easier and less expensive.
- are most commonly used to span short spans up to 12 meters.

- It is the most common forms of bridges in Ethiopia, and economical for spans:
  - 4m to 15 m [ERA, manual] (and if >15m they can be ribbed)
  - 6-12m [AACRA]
  - 10-12m [AASHTO]

Optional Criteria for Span-to-Depth Ratios

- Unless otherwise specified herein, if an Owner chooses to invoke controls on span-to-depth ratios, the limits in AASHTO, Table 2.5.2.6.3-1, in which S is the slab span length and *L* is the span length, both in ft., may be considered in the absence of other criteria. Where used, the limits in Table 2.5.2.6.3-1 shall be taken to apply to overall depth unless noted.

Table 2.5.2.6.3-1 Traditional Minimum Depths for Constant Depth Superstructures.

Superstructure		Minimum Depth (Including Deck) When variable depth members are used, values may be adjusted to account for changes in relative stiffness of positive and negative moment sections	
Material	Type	Simple Spans	Continuous Spans
Reinforced Concrete	Slabs with main reinforcement parallel to traffic	$\frac{1.2(S+3000)}{30}$	$\frac{S+3000}{30} \geq 165\text{ mm}$
	T-Beams	$0.070L$	$0.065L$
	Box Beams	$0.060L$	$0.055L$
	Pedestrian Structure Beams	$0.035L$	$0.033L$
Prestressed Concrete	Slabs	$0.030L \geq 165$ mm	$0.027L \geq 165$ mm
	CIP Box Beams	$0.045L$	$0.040L$
	Precast I-Beams	$0.045L$	$0.040L$
	Pedestrian Structure Beams	$0.033L$	$0.030L$
	Adjacent Box Beams	$0.030L$	$0.025L$
Steel	Overall Depth of Composite I-Beam	$0.040L$	$0.032L$
	Depth of I-Beam Portion of Composite I-Beam	$0.033L$	$0.027L$
	Trusses	$0.100L$	$0.100L$

SLAB BRIDGES

Depth Determination:

- According to AASHTO, Table 2.5.2.6.3.1, minimum recommended depth for slab with main reinforcement parallel to the traffic is :

$$D = \frac{1.2(S + 3000)}{30} \qquad \textbf{(mm)}$$

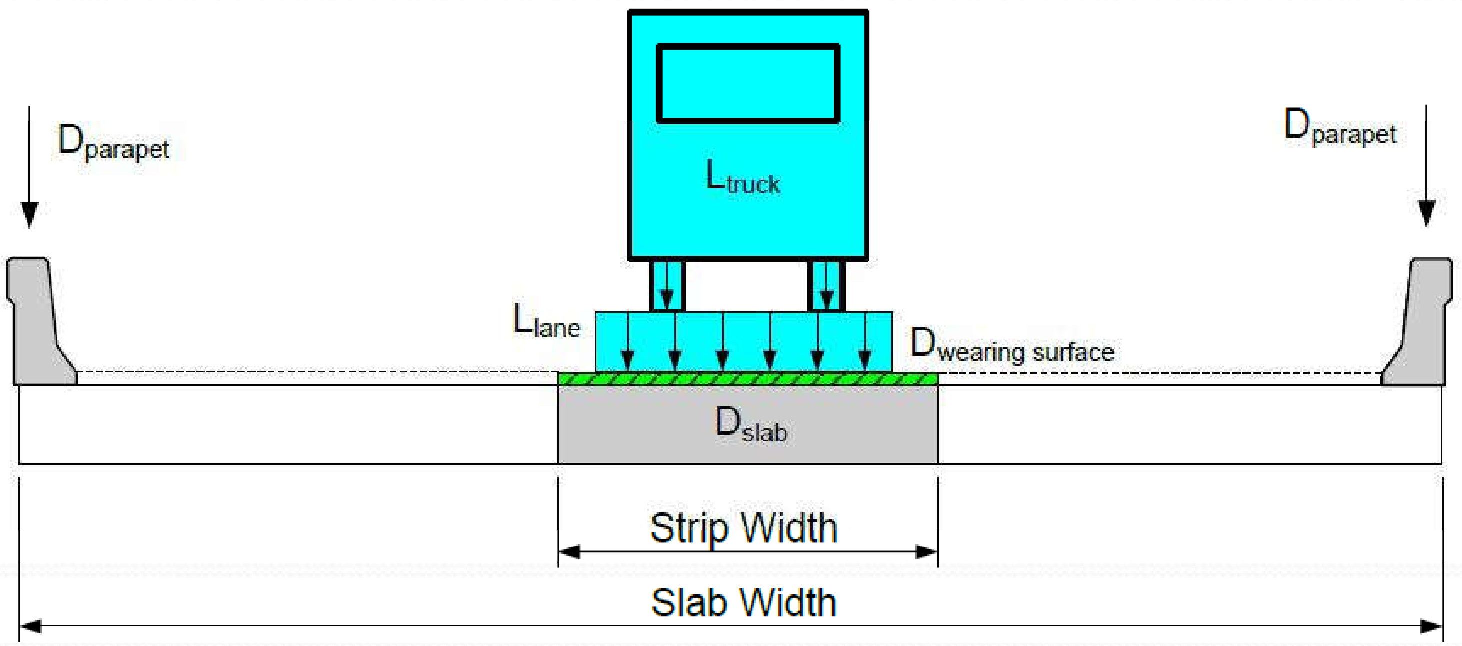
where:

D : slab thickness

S : c/c spacing of the bridge (mm)

According to ERA Bridge Design Manual 2013, article 5.4.1.1, the depth of a concrete deck, excluding any provision for grinding, grooving, and sacrificial surface, should not be less than 185 mm and minimum cover should not be less than 35mm.

Load distributions:



According to AASHTO sec. 4.6.2.3, the equivalent width (*E*) of longitudinal strips per lane for both shear and moment with one lane, i.e., two lines of wheels, loaded shall be determined

Load distributions …..

Interior Strip width

$$E = 250 + 0.42\sqrt{L_1 W_1}$$ Single lane loaded

$$E = 2100 + 0.12\sqrt{L_1 W_1} \le \frac{W}{N_L}$$ Multiple lanes loaded

Where: E = equivalent width (mm)

L_1 = modified span length taken ≤ of the actual span or 18,000 (mm)

W_1 = modified edge-to-edge width of bridge taken to be ≤ of the actual width or 18,000 mm for multilane loading, or 9,000 mm for single-lane loading (mm)

W = physical edge-to-edge width of bridge (mm)

N_L = number of design lanes as specified

The general equation for loads applied to the interior strip width is as follows:

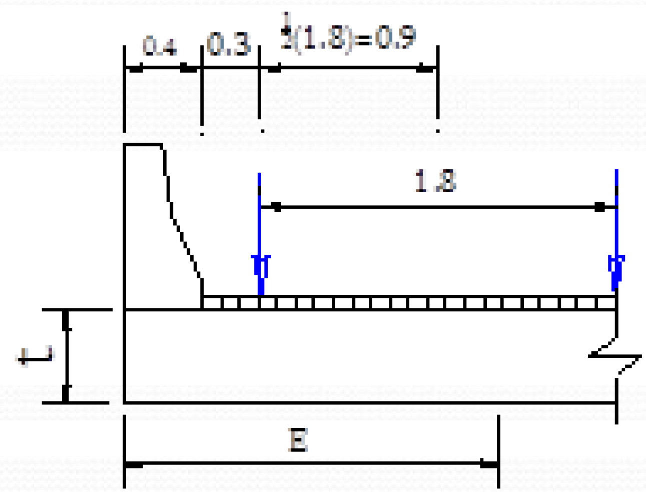
$$\text{Total Load} = D_{slab} + D_{wearing\ surface} + \left[(2\,D_{parapet})\left(\frac{\text{Strip Width}}{\text{Slab Width}}\right)\right] + (L_{truck} + L_{lane})$$

WisDOT, BDM-17.2.7

Edge Strip width

Edge strip is limited to half lane width; use multiple presence factors 1.2 and half design lane load (for a two-lane bridge, because the possibility of occurrence of two trucks at a time is less).

Thus, live loads due to truck and tandem are divided by 2 as the width of the edge strip is less than 2.1m (wheel are placed 300mm from curb edge and wheel spacing of 1800mm) plus curb width. Thus the effect of the live load is reduced by half.



$$E_e = \min\begin{cases} \left(C_w + 300 + \dfrac{E_i}{2}\right) \\ 1800\,(mm) \end{cases}$$

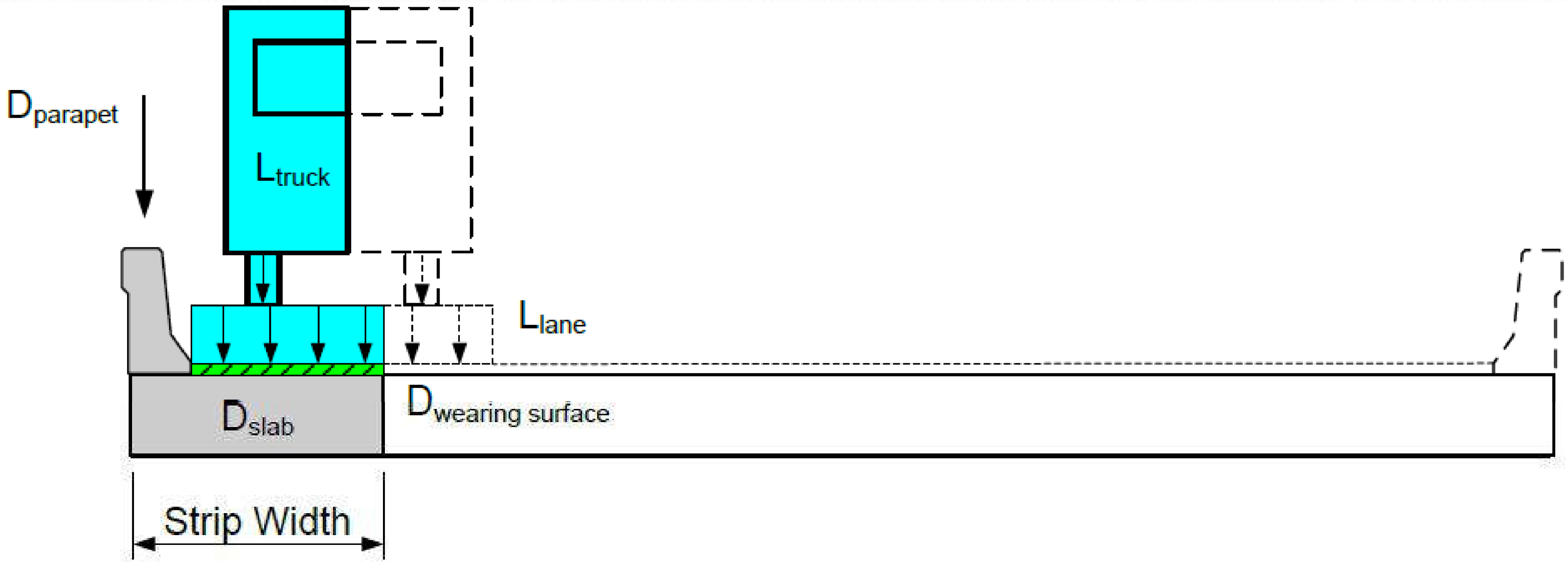
Where:

E_e : Edge strip width

C_w : Curb width

E_i : interior strip width

Edge Strip width . . .



Thus, live loads due to truck and tandem are divided by 2 as the strip width is less than the width of the truck and only half of the live load is with in the edge strip width.

The general equation for loads applied to the exterior strip width is as follows:

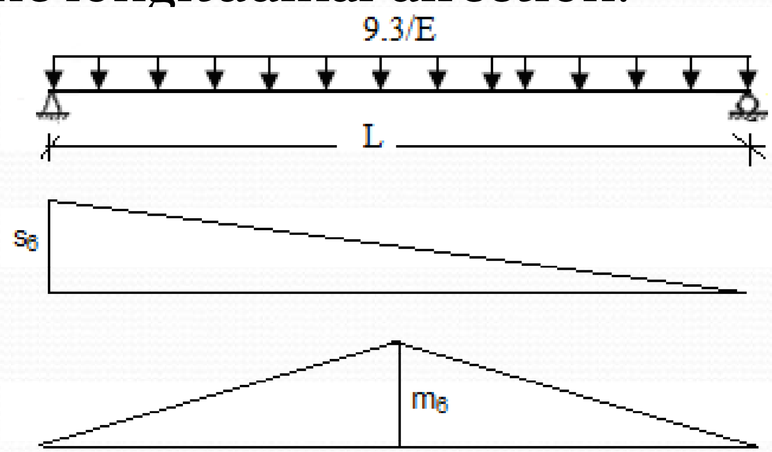
$$\text{Total Load} = D_{slab} + \left(D_{wearing\ surface} + D_{parapet}\right)_{directly\ over\ strip} + \left(L_{truck} + L_{lane}\right)_{directly\ over\ strip}$$

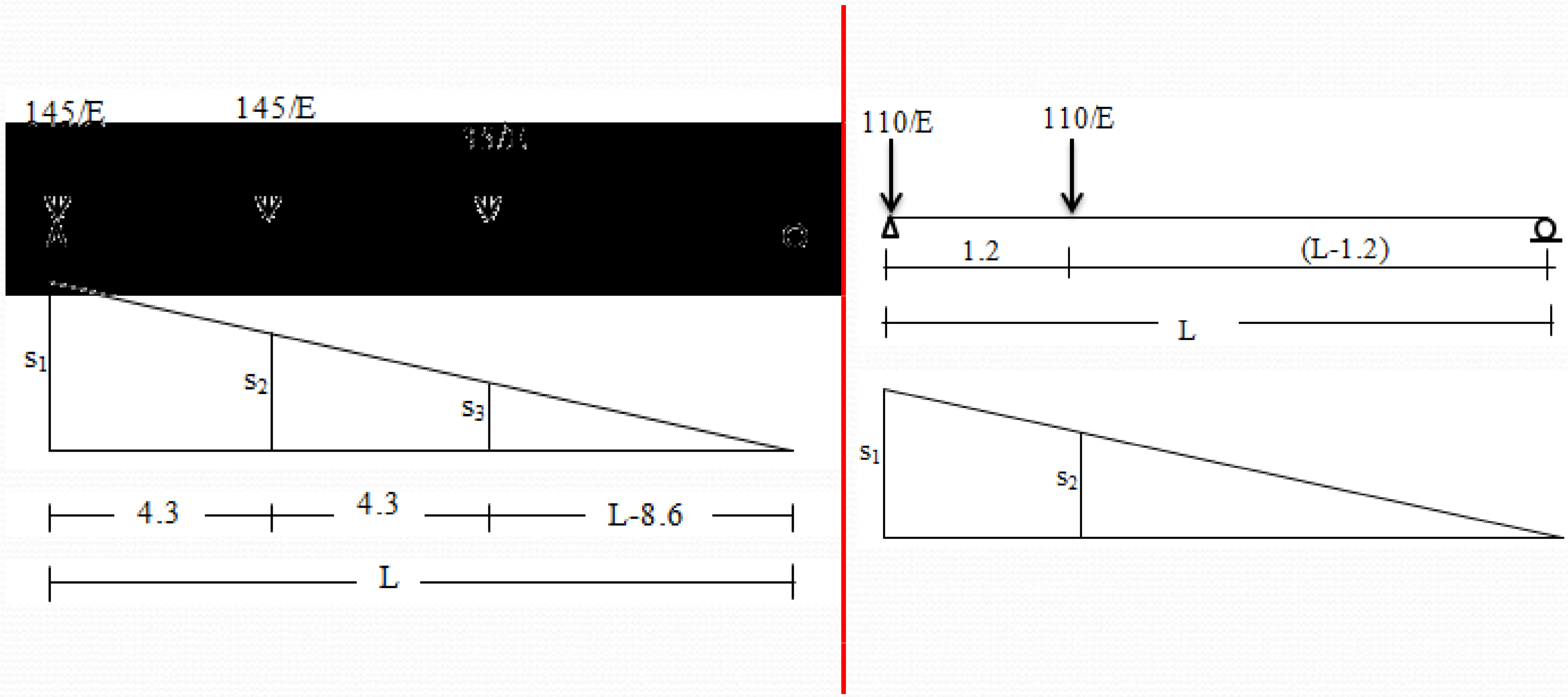
Equivalent Concentrated and Distributed Loads

The equivalent concentrated and distributed loads per meter width of both interior and edge strips are obtained by dividing the design loads to the corresponding strip width and applying a dynamic impact factor. For the calculation of live load force effects, influence line is used and the maximum effect will be selected for the design.

Lane Load:

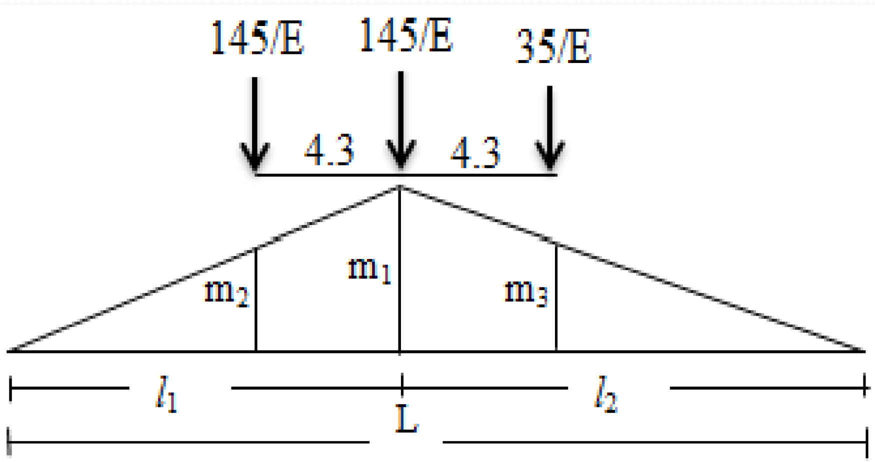
The design lane load shall consist of a load of 9.3kN/m uniformly distributed in the longitudinal direction.





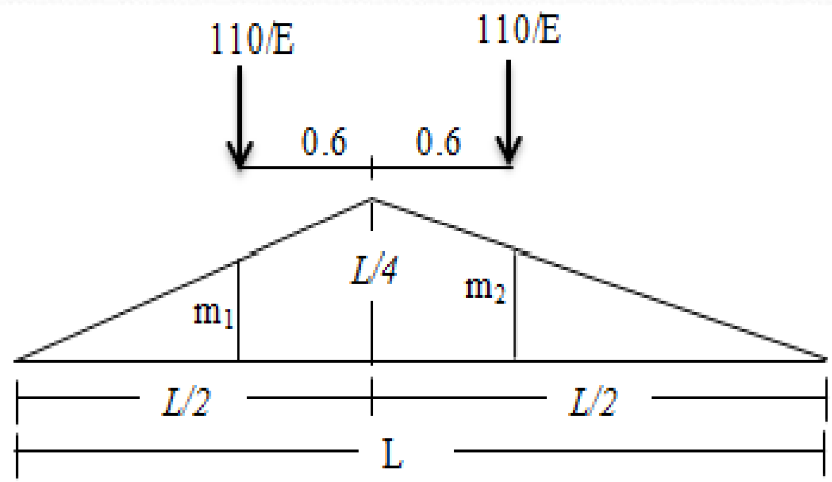
IL for maximum shear force due to truck and tandem loads

Live load force effects



$$m_1 = \frac{l_1 l_2}{L} \qquad m_2 = \frac{l_2 \langle l_1 - 4.3 \rangle}{L}$$

$$m_3 = \frac{l_1 \langle l_2 - 4.3 \rangle}{L}$$



$$m_1 = m_2 = \frac{(L - 1.2)}{4}$$

IL for maximum bending moment due to truck and tandem loads

Maximum Live Load Moments for Interior Strips

$$M_{tr} = \frac{145}{E}(m_1 + m_2) + \frac{35}{E} m_3$$

$$M_{\tan} = \frac{110}{E}(m_1 + m_2)$$

$$M_{ln} = \frac{9.3}{E} \frac{(L^2)}{8}$$

$$(M_{LL+IM})_{\text{int}} = 1.33 max\ (M_{tr},\ M_{\tan}) + M_{\ln}$$

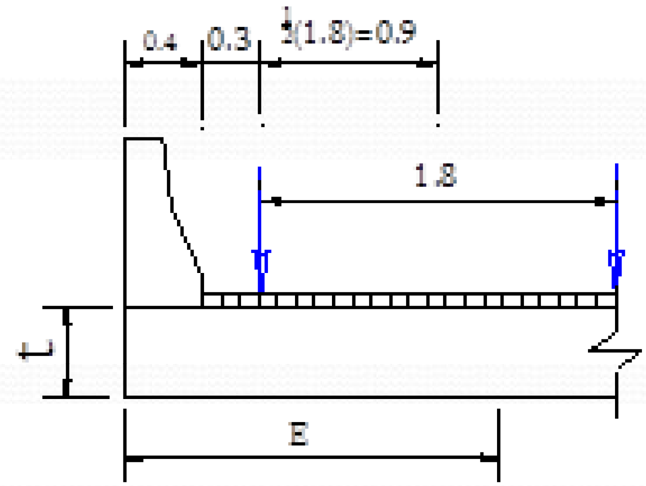
where E : interior strip width

Maximum Live Load Moments for edge Strips

$$M_{tr} = \frac{m}{2}\left(\frac{145}{E_e}(m_1 + m_2) + \frac{35}{E_e} m_3\right)$$

$$M_{tan} = \frac{m}{2}\left(\frac{110}{E_e}(m_1 + m_2)\right)$$

$$M_{ln} = \frac{m}{2}\left(\frac{9.3}{E_e}\frac{(L^2)}{8}\right)$$



$$(M_{LL+IM})_{edge} = 1.33 max\left(M_{tr}, M_{tan}\right) + M_{\ln}$$

where Ee : edge strip width

m : multiple presence factor (=1.2)

Thus, the design moment is taking by considering the maximum effects due to live and dead load moments.

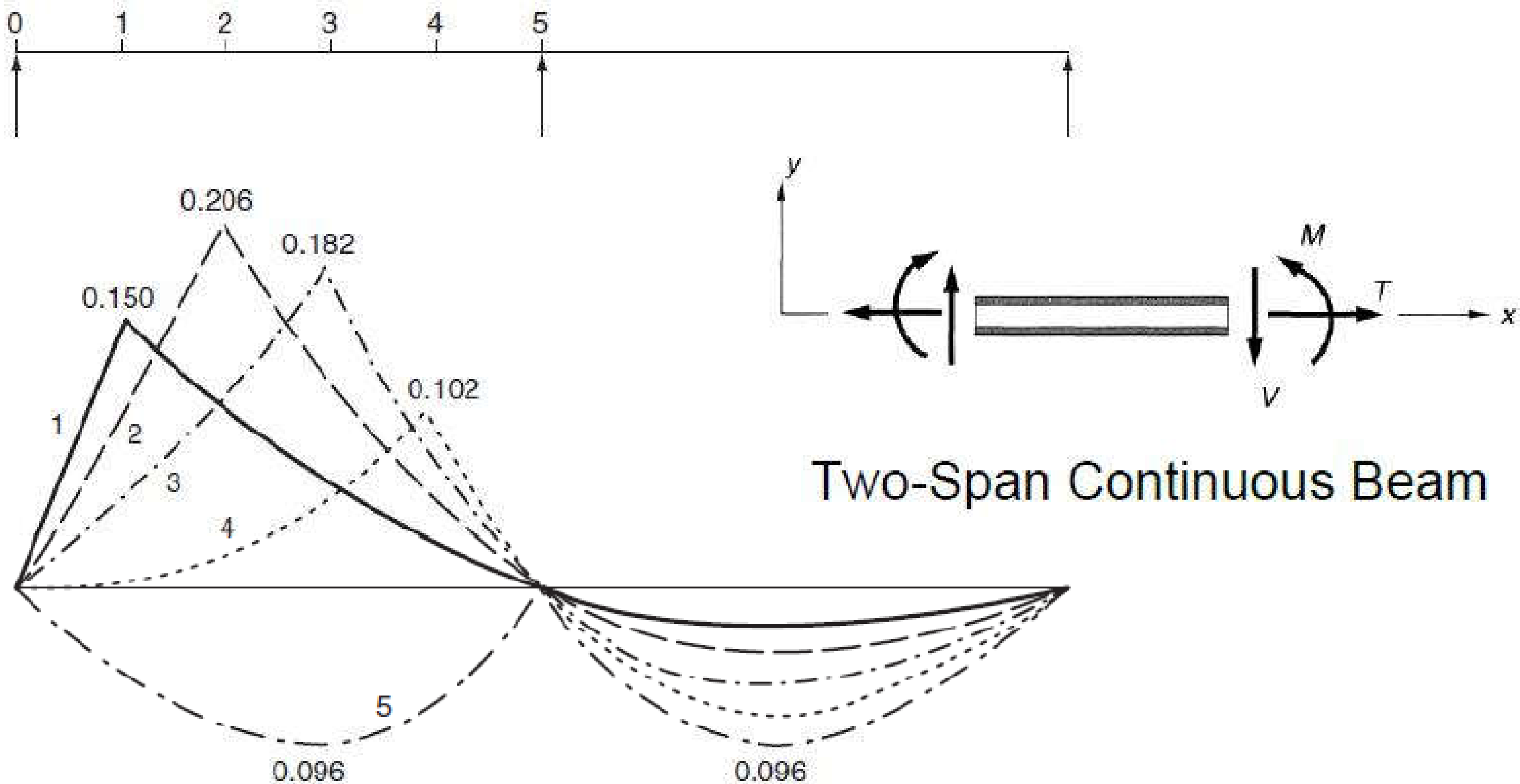
Maximum Live Load Moments for interior and edge strips

The maximum Live Load Moments for both interior and edge strips can be computed using influence line concepts and the influence line coefficients (maximum ordinates)

Influence Functions (Influence Lines)

1. Vehicular loads move on bridges
2. Hence the load effects vary as the vehicle traverses the bridge.
3. Maximum load effects have to be found by placing the vehicular loads in the most critical manner on bridges
4. Influence functions / Influence lines provide the foundation for determining the critical positions of vehicular loads

Influence Line Diagram for B.M.



Two-Span Continuous Beam

Design Moment

The design moment is computed by combining the effects of dead loads and live loads and applying the corresponding load combinations and load factors specified in AASHTO, Table 3.4.1.1.

$$M_{sd} = \eta(1.25M_{DL} + 1.5M_{DW} + 1.75M_{LL+IM})$$

For strength limit state η=1.05- for critical or essential bridges.

where:

M_{sd} : design moment

M_{DL} : dead load moment

M_{DW} : dead load moment from wearing surfaces

M_{LL+IM} : moment from live load and impact and given by

$$= 1.33 max \left(M_{tr}, M_{tan} \right) + M_{ln}$$

M_{tr} : moment from Truck load

M_{tan} : moment from Tandem load

M_{ln} : moment from lane load

Design

- Check the adequacy of the section

- Main Reinforcement

The required amount of reinforcement for both interior and edge strips are computed using the general empirical equation:

$$\rho = \left(1 - \sqrt{1 - \frac{2M_u}{0.9bd^2\varphi f'_c}}\right)\frac{\varphi f'_c}{f_y}$$

and $\rho_{min} = \dfrac{0.03 f_c'}{f_y}$

The factor φ accounts the reduction in concrete compressive strength due to slow loading or the weakening of the concrete =0.85.

Refer: Design of Reinforced Concrete Structures

Shear Reinforcement

Slab bridges designed for moment in conformance with Article 4.6.2.3 may be considered satisfactory for shear. (AASHTO Article 5.14.4.1)

Distribution Reinforcement

According to AASHTO, article 5.14.4.1, if the main reinforcement is parallel to the traffic, the amount of bottom transverse reinforcement may be taken as a percentage of the main reinforcement required for positive moment.

$$p_e = min \ [50, \frac{1750}{\sqrt{L_1}}]$$

Shrinkage & Temperature Reinforcement

As indicated in AASHTO, section 5.10.8, reinforcement for shrinkage and temperature reinforcement shall be provided near surfaces of concrete exposed to daily temperature changes. The specified amount of the steel should be distributed equally on both sides.

$$A_{st} \geq \frac{0.75 A_g}{f_y}$$

Serviceablity Limit Requirements

Actions to be considered at the service limit state shall be cracking, deformations, and concrete stresses, as specified in Articles 5.7.3.4, 5.7.3.6, and 5.9.4 respectively.

- **Durability**

  For durability, adequate cover shall be used (for bottom of cast in place slab the cover is 35mm).

- **Adequacy of Reinforcement Bars**

$$A_s = \frac{M_p}{f_s j d_p}$$

Assume; j =0.875 and fs=0.6fy

For determination of Mp, the load factors to be are taken as unity.

Serviceablity . ..

iii) Control of Cracking

The cracking stress shall be taken as the modulus of rupture specified in AASHTO, Article 5.4.2.6.

Cracking may occur in the tension zone for RC members due to the low tensile strength of concrete. The cracks may be controlled by distributing steel reinforcements over the maximum tension zone in order limit the maximum allowable crack widths at the surface of the concrete for given types of environment.

The tensile stress in the mild steel reinforcement (*fs*) *at the service limit state doesn't exceed* f_{sa}.

$$f_{sa} = \frac{Z}{(dc * A)^{\frac{1}{3}}} \leq 0.6 f_y$$

Z=crack width parameter=23,000N/mm

$$Z = \begin{cases} 30,000 & \text{moderate exposure conditions} \\ 23,000 & \text{severe exposure conditions} \\ 17,500 & \text{buried structures} \end{cases}$$

Serviceablity . ..

Control of Cracking . . .

For rectangular cross section,

$$f_{cten} = \frac{6M_{us}}{bD^2}$$ where f_{cten}= *tensile strength of the concrete*

If $f_{cten} > 0.8f_r$, *the section has cracked (AASHTO, Article 5.7.3.4 and 5.4.2.6)*

Where: f_r = *modulus of rupture*=$0.63\sqrt{f_c'}$

Type of exposure	**Dry environment (Mild)**	**Humid environment (eg. Laundries) (Moderate)**	**Sea water and/ or aggressive chemical environment (Severe)**
Characteristic crack widths, W_k (mm)	0.4	0.2	0.1

Refer: **"Design of Reinforced Concrete Structures"**

Serviceablity . ..

Control of Cracking . . .

There are four fundamental ways in which surface crack widths may be reduced:

- Reduce the stress in the reinforcement (σ_s) which will hence reduce ε_{sm}
- Reduce the bar diameters (Φ) which will reduce bar spacing & have the effect of reducing the crack spacing (S_m)
- Increase the effective reinforcement ratio (ρ_r)
- Use high bond rather than plain bars

Serviceablity . ..

iv) Deformations and Camber

Deflection and camber calculations shall consider dead load, live load, erection loads, concrete creep and shrinkage.

Immediate (Instantaneous) deflections may be computed taking the moment of inertia as either the effective moment of inertia I_{eff}, or the gross moment of inertia I_g.

Unless a more exact deformation calculation is made, the long-term deflection due to creep and shrinkage may be taken as the immediate deflection multiplied by the following factor (AASHTO, Article 5.7.3.6.2).

4, if the instantaneous deflection is based on I_g.

$\boxed{3.0 - 1.2\left(\frac{A'_s}{A_s}\right) \geq 1.6}$ if the instantaneous deflection is based on I_e.

Serviceablity . ..

iv) Deformations ……

The deflection of a structure should not adversely affect the proper functioning or appearance of a structure. The immediate elastic deflection caused by service loads may be calculated using usual elastic theory equations for deflections.

Long term deflections

The deflection of reinforced concrete beams increases with time due to creep and shrinkage.

Additional deflections two or three times as large as the immediate deflection may result externally.

Deflection Limit:

For simple span bridges =L/800

For overhanging part, cantilevers = L/300

Fatigue Limit State

Investigation of Fatigue Limit State (AASHTO, Section 5.5.3)

The fatigue load shall be one design truck or axles thereof specified in Article 3.6.1.2.2, but with a constant spacing of 9000mm between the 145 000-N axles. The dynamic load allowance specified in Article 3.6.2 shall be applied to the fatigue load.

U=0.75(LL+IM) ; F.S for LL is 0.75

a) Tensile live load stresses

$$f_{s,max} = n\frac{M_{llf}(D-x_2)}{I_{cr}} \leq 0.4f_y$$

M_{llf} is the maximum moment per meter width for fatigue.

Fatigue Limit State . .. .

b) Reinforcing Bars

The stresses range in straight reinforcement bars resulting from fatigue load combination shall not exceed (AASHTO, Section 5.5.3.2).

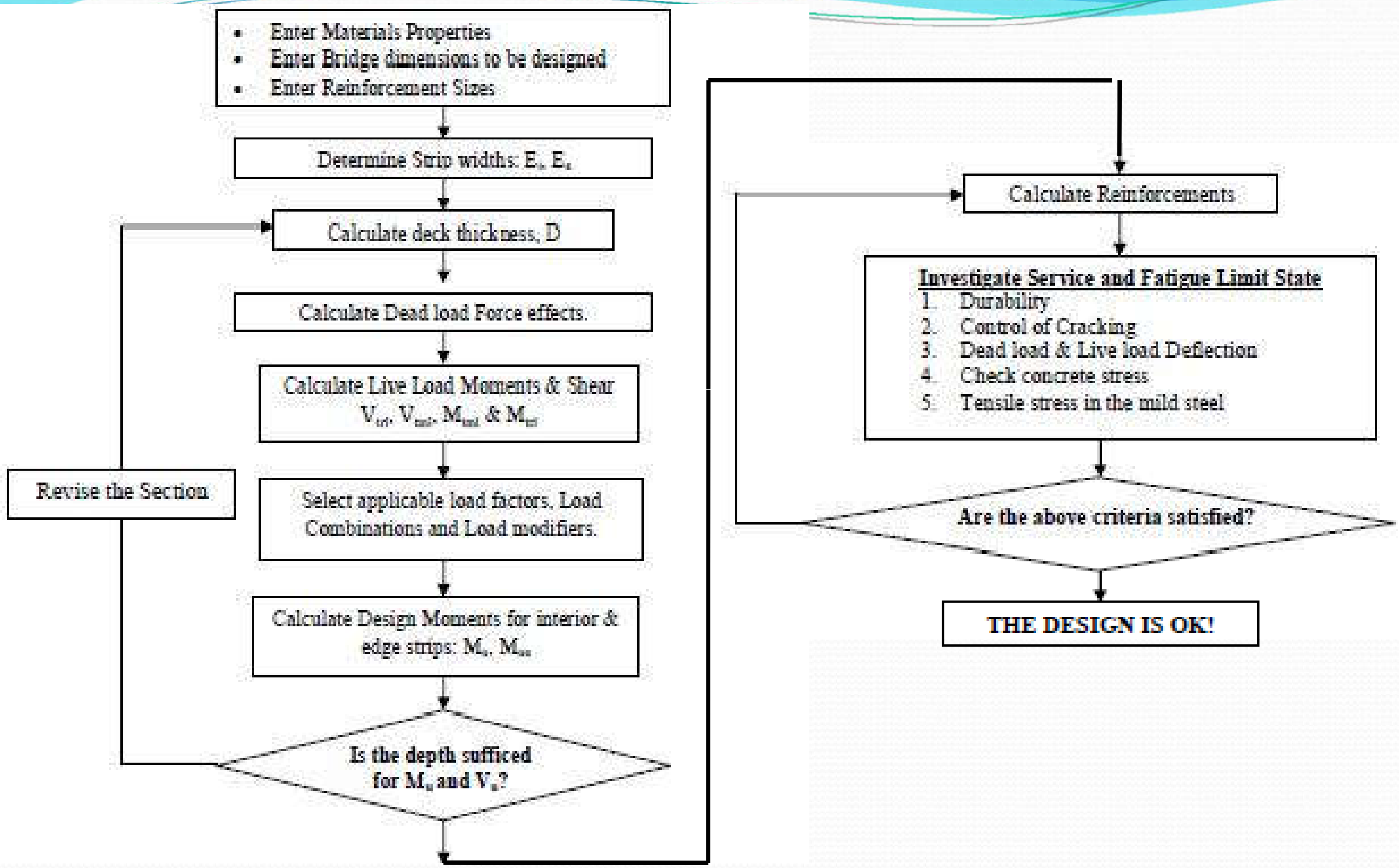
If $f_{s,max} < f_f$, then there is no problem of fatigue. Otherwise increase the area of reinforcing bars.

Where:

f_f *is the stress range.*

f_{min} *is the minimum live load stress resulting from fatigue load, combined with the more severe stress from permanent loads. For* simply supported slab bridge f_{min} *is zero.*

$$f_f = 166 - 0.33 f_{min}$$



Flow chart for the design of RC slab bridges

Design Example

RC SLAB BRIDGE DESIGN

Design Data and Specifications

i) Material Properties

Steel strength, f_y =	400 MPa
Concrete strength, f'_c =	28 MPa
Concrete density, γ_c =	2400 kg/m^3
Bituminous density, γ_b =	2250 kg/m^3
The modulus of elasticity of steel, E_s =	200 GPa

ii) Bridge Span and Support Dimensions

Clear span of the bridge, C_s =	12 m
Road way /clear carriage width, R_w =	7.32 m
Additional curb width including bottom width of the concrete barrier/ posts, Cw=	0.80 m
Curb depth, C_d =	0.25 m
Bearing shelf width, W_{rs} =	0.5 m
Concrete barriers are used	
Thickness of Asphalt Layer =	100 mm
Concrete cover for the slab =	35 mm

*** Design Method: LRFD**

Specifications:

-AASHTO LRFD Bridge Design Specifications, 4th ed. 2007

-Ethiopian Roads Authority, ERA Bridge Design Manual, 2013

Analysis and Design of T-Girder Bridges


Cross Section of a T-Girder Bridge

T-Girder Bridges

- The T-Girder construction consists of a transversely reinforced slab deck which spans across to the longitudinal support girders.
- The slab is structural1y continuous across the top. The slab serves dual purpose of supporting the live load on the bridge and acting as the top flange of the longitudinal beams.
- T-Girder require a more-complicated formwork.
- T- Girders are used for bridges spanning from about **10 -25 meters.**
- The girder stem thickness usually varies from **35 to 55 cm.**
- Optimum lateral spacing of longitudinal girders is typically between **1.8 and 3.0 m** (equally spaced beams)
- They shall be used for span lengths
  - 10 – 25 m [ERA]
  - 10 – 20 m [Design of RC Bridge]
  - 10-25m [AASHTO]
  - 12-18m [Bridge Engineering Handbook]

Diaphragms

- Diaphragms are provided transversely between the beams over the supports and depending on the span, at mid-span and other intermediate locations.

- The purpose of providing diaphragms is to ensure lateral distribution of live loads to various adjacent stringers, the magnitude of the share of each stringer depends on the stiffness of the diaphragms relative to the stringers and on the method of connectivity.
- *Diaphragms shall be used at span ends. Intermediate diaphragms shall be used where required in the judgment of the Engineer.*

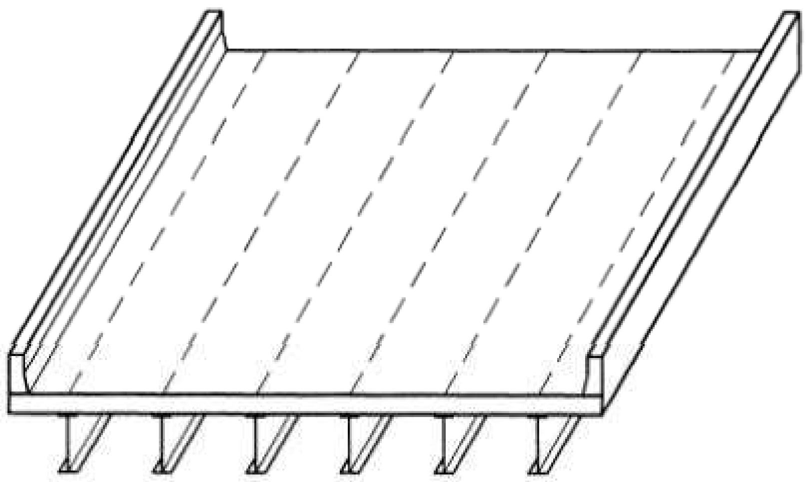
Diaphragms

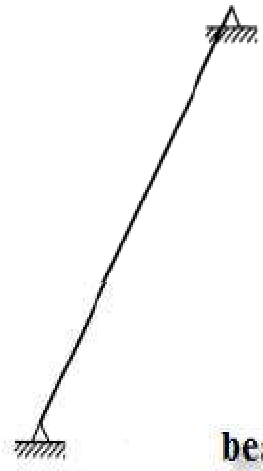
Analysis of T-Girder Bridges

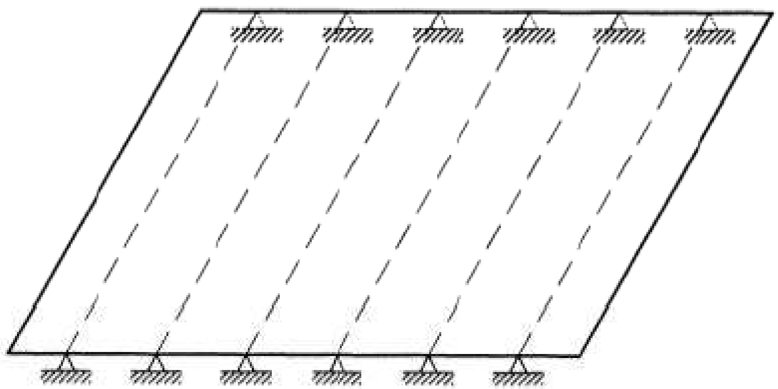
- Design of T- girder bridges consists of
  - deck slab analysis and design, and
  - T-girder analysis and design.

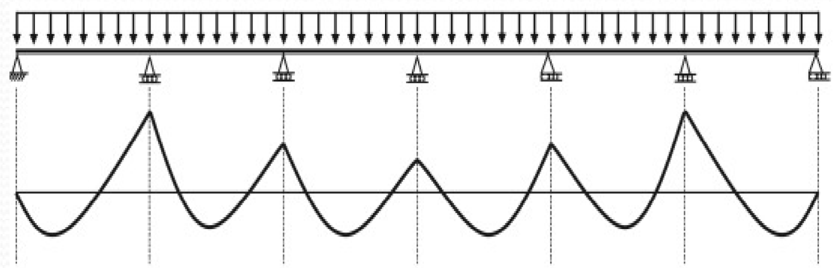
- Structural analysis of the deck slab involves taking a continuous strip perpendicular to the girders (AASHTO Art.9.6.1) and analyzing by moment distribution or using design aid given by AASHTO.
- An approximate method of analysis in which the deck is subdivided into strips perpendicular to the supporting components shall be considered acceptable for decks (AASHTO Art. **4.6.2.1**)

Analysis of T-Girder Bridges . . .


Slab-Girder Bridge


beam line model


flate-plate model (2D)



Design of Deck Slab of Girder Bridges

Thickness of Top Flange (ERA Bridge Design Manual 2013, Article 5.4.1.1 and AASHTO, Article 5.14.1.5.1a)

The thickness of top flanges serving as deck slabs shall be:

- the depth of a concrete deck, excluding any provision for grinding, grooving, and sacrificial surface, should not be less than 185 mm
- Not less than the clear span between fillets, haunches, or webs divided by 20, unless transverse ribs at a spacing equal to the clear span are used or transverse prestressing is provided.

Slab thickness (AASHTO, Table 2.5.2.6.3.1):

$$t_s = \frac{(s+3000)}{30}$$

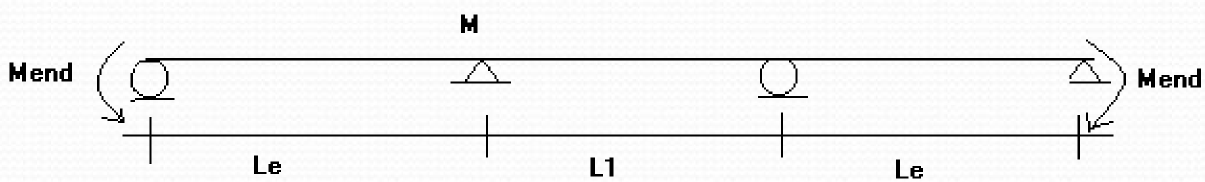
S= Girder spacing (mm)

Design of Deck Slab of Girder Bridges . . .

An approximate method of analysis (AASHTO, article **4.6.2.1**) in which the deck is subdivided into strips perpendicular to the supporting components shall be considered acceptable for decks.

- **Influence line segment coefficient**

The Influence line segment coefficient table is intended



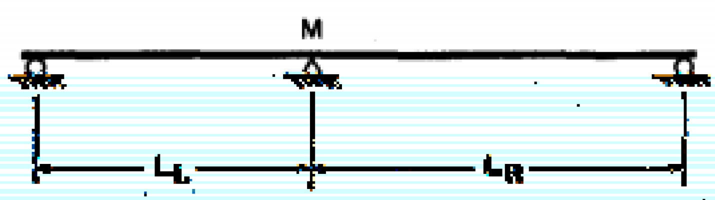
$$M = \left(\sum W \, . \, coef\right) L_R^2 + \sum M_E \, coef$$

Influence Segment Coefficients

TABLE 1 INFLUENCE SEGMENT COEFFICIENTS FOR 2 SPANS - - 1-ST INTERIOR SUPPORT

[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
[illegible]									
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
LAST 70 OF FIRST SPAN	10	0.011971	0.014511	0.017339	0.020458	0.023876	0.027596	0.031623	0.035962
LAST 70 OF FIRST SPAN	15	0.009751	0.011821	0.014124	0.016666	0.019450	0.022480	0.025761	0.029295
FIRST 40 OF FIRST SPAN	00	0.006124	0.007424	0.008871	0.010467	0.012216	0.014119	0.016180	0.018399
LAST 60 OF FIRST SPAN	05	0.012306	0.014918	0.017824	0.021031	0.024545	0.028369	0.032510	0.036970
LAST 60 OF FIRST SPAN	10	0.010173	0.012332	0.014735	0.017386	0.020290	0.023452	0.026875	0.030562
LAST 60 OF FIRST SPAN	15	0.008271	0.010027	0.011980	0.014135	0.016497	0.019067	0.021850	0.024848
FIRST 50 OF FIRST SPAN	00	0.009102	0.011034	0.013183	0.015555	0.018154	0.020982	0.024044	0.027343
LAST 50 OF FIRST SPAN	05	0.009724	0.011789	0.014085	0.016619	0.019396	0.022418	0.025690	0.029214
LAST 50 OF FIRST SPAN	10	0.007947	0.009634	0.011511	0.013582	0.015851	0.018320	0.020994	0.023874
LAST 50 OF FIRST SPAN	15	0.006362	0.007712	0.009215	0.010873	0.012689	0.014667	0.016807	0.019113
LAST 30 OF LAST SPAN	00	0.013022	0.012639	0.012278	0.011937	0.011614	0.011309	0.011019	0.010743
FIRST 70 OF LAST SPAN	05	0.052652	0.051103	0.049643	0.048264	0.046960	0.045724	0.044552	0.043438
FIRST 70 OF LAST SPAN	10	0.043590	0.042308	0.041099	0.039958	0.038878	0.037855	0.036884	0.035962
FIRST 70 OF LAST SPAN	15	0.035510	0.034465	0.033481	0.032551	0.031671	0.030837	0.030047	0.029295
LAST 40 OF LAST SPAN	00	0.022303	0.021647	0.021028	0.020444	0.019891	0.019368	0.018871	0.018400
FIRST 60 OF LAST SPAN	05	0.044812	0.043494	0.042251	0.041078	0.039967	0.038916	0.037918	0.036970
FIRST 60 OF LAST SPAN	10	0.037045	0.035955	0.034928	0.033958	0.033040	0.032171	0.031346	0.030562
FIRST 60 OF LAST SPAN	15	0.030119	0.029233	0.028398	0.027609	0.026863	0.026156	0.025485	0.024848
LAST 50 OF LAST SPAN	00	0.033143	0.032169	0.031250	0.030381	0.029560	0.028782	0.028044	0.027343
FIRST 50 OF LAST SPAN	05	0.035411	0.034370	0.033388	0.032460	0.031583	0.030752	0.029963	0.029214
FIRST 50 OF LAST SPAN	10	0.028939	0.028088	0.027285	0.026527	0.025810	0.025131	0.024487	0.023875
FIRST 50 OF LAST SPAN	15	0.023167	0.022486	0.021843	0.021237	0.020663	0.020119	0.019603	0.019113
UNIT MOMENT ON LEFT END		-0.196969	-0.205882	-0.214285	-0.222222	-0.229729	-0.236842	-0.243589	-0.250000
UNIT MOMENT ON RIGHT END		-0.303030	-0.294117	-0.285714	-0.277777	-0.270270	-0.263157	-0.256410	-0.250000
- - - APPLIED LOADS - - -									
UNIT DEAD LOAD ON 1-ST SPAN		-0.020804	-0.025220	-0.030133	-0.035555	-0.041494	-0.047960	-0.054959	-0.062500
UNIT DEAD LOAD ON 2-ND		-0.075757	-0.073529	-0.071428	-0.069444	-0.067567	-0.065789	-0.064102	-0.062500
UNIT DEAD LOAD ON BOTH SPANS		-0.096562	-0.098749	-0.101562	-0.104999	-0.109062	-0.113749	-0.119062	-0.125000

*For example, in the third line of coefficients the 70 and 10 indicate 70% and 10% of the first span.



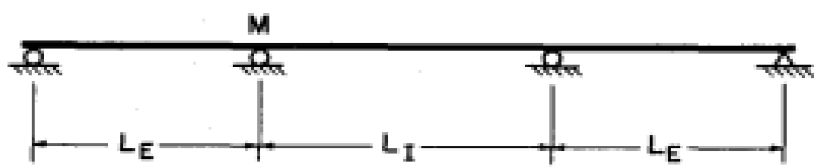
$$M = \left\{ \sum w \cdot \mathrm{coef} \right\} \cdot L_R^2 + \sum M_E \cdot \mathrm{coef}$$

Influence Segment Coefficients . . .

TABLE II INFLUENCE SEGMENT COEFFICIENTS FOR 3 SPANS - - 1-ST INTERIOR SUPPORT

NUMBERS REFER TO PER CENT OF SPAN — - - RATIO OF EXTERIOR SPAN LENGTH TO INTERIOR SPAN LENGTH - -

LOADING DESCRIPTION	REVERSE CURVE	0.650	0.700	0.750	0.800	0.850	0.900	0.950	1.000
- - SYMMETRIC PRESTRESS - -									
END 30 OF END SPANS (SYM)	00	0.002744	0.003350	0.004028	0.004783	0.005615	0.006526	0.007519	0.008594
INNER 70 OF END SPANS	05	0.011096	0.013544	0.016289	0.019339	0.022703	0.026388	0.030402	0.034750
INNER 70 OF END SPANS	10	0.009187	0.011213	0.013485	0.016011	0.018796	0.021847	0.025170	0.028769
INNER 70 OF END SPANS	15	0.007484	0.009134	0.010985	0.013043	0.015311	0.017797	0.020504	0.023436
END 40 OF END SPANS	00	0.004700	0.005737	0.006899	0.008191	0.009616	0.011177	0.012878	0.014719
INNER 60 OF END SPANS	05	0.009444	0.011527	0.013863	0.016459	0.019322	0.022459	0.025875	0.029576
INNER 60 OF END SPANS	10	0.007807	0.009529	0.011460	0.013606	0.015973	0.018566	0.021390	0.024449
INNER 60 OF END SPANS	15	0.006347	0.007748	0.009318	0.011062	0.012987	0.015095	0.017391	0.019878
END 50 OF END SPANS	00	0.006985	0.008526	0.010253	0.012173	0.014291	0.016611	0.019137	0.021875
INNER 50 OF END SPANS	05	0.007463	0.009109	0.010955	0.013006	0.015269	0.017747	0.020447	0.023371
INNER 50 OF END SPANS	10	0.006099	0.007444	0.008953	0.010629	0.012478	0.014504	0.016710	0.019099
INNER 50 OF END SPANS	15	0.004882	0.005959	0.007167	0.008509	0.009989	0.011611	0.013377	0.015290
CENTER SPAN	05	0.049709	0.048579	0.047499	0.046467	0.045478	0.044531	0.043622	0.042749
CENTER SPAN	10	0.041860	0.040909	0.039999	0.039130	0.038297	0.037499	0.036734	0.035999
CENTER SPAN	15	0.034593	0.033806	0.033055	0.032337	0.031649	0.030989	0.030357	0.029750
UNIT MOMENTS ON THE ENDS		-0.151162	-0.159090	-0.166666	-0.173913	-0.180851	-0.187499	-0.193877	-0.200000
- ANTI-SYMMETRIC PRESTRESS -									
END 30 OF END SP. (ANTI-	00	0.005131	0.006141	0.007252	0.008462	0.009774	0.011188	0.012705	0.014324
INNER 70 OF END SP. -SYM)	05	0.020746	0.024832	0.029320	0.034215	0.039520	0.045237	0.051369	0.057917
INNER 70 OF END SP.	10	0.017175	0.020558	0.024274	0.028327	0.032719	0.037452	0.042528	0.047949
INNER 70 OF END SP.	15	0.013990	0.016745	0.019772	0.023073	0.026650	0.030506	0.034641	0.039056
END 40 OF END SP.	00	0.008787	0.010518	0.012419	0.014493	0.016740	0.019162	0.021759	0.024533
INNER 60 OF END SP.	05	0.017657	0.021134	0.024954	0.029121	0.033636	0.038501	0.043720	0.049293
INNER 60 OF END SP.	10	0.014596	0.017471	0.020629	0.024073	0.027806	0.031828	0.036142	0.040749
INNER 60 OF END SP.	15	0.011867	0.014204	0.016772	0.019572	0.022607	0.025877	0.029385	0.033131
END 50 OF END SP.	00	0.013059	0.015631	0.018457	0.021538	0.024877	0.028476	0.032336	0.036458
INNER 50 OF END SP.	05	0.013953	0.016701	0.019719	0.023012	0.026580	0.030425	0.034548	0.038953
INNER 50 OF END SP.	10	0.011402	0.013648	0.016115	0.018806	0.021721	0.024864	0.028234	0.031833
INNER 50 OF END SP.	15	0.009130	0.010928	0.012903	0.015058	0.017392	0.019908	0.022607	0.025488
UNIT MOMENTS ON THE ENDS		-0.282608	-0.291666	-0.300000	-0.307692	-0.314814	-0.321428	-0.327586	-0.333333
- - - APPLIED LOADS - - -									
UNIT DEAD LOAD ON 1-ST SPAN		-0.022908	-0.027608	-0.032812	-0.038528	-0.044764	-0.051528	-0.058827	-0.066666
UNIT DEAD LOAD ON 2-ND		-0.058139	-0.056818	-0.055555	-0.054347	-0.053191	-0.052083	-0.051020	-0.050000
UNIT DEAD LOAD ON 3-RD		0.006941	0.008120	0.009375	0.010702	0.012098	0.013560	0.015084	0.016666
UNIT DEAD LOAD ON ALL SPANS		-0.074106	-0.076306	-0.078993	-0.082173	-0.085857	-0.090052	-0.094764	-0.100000
UNIT D.L. ON SPANS 1 AND 2		-0.081048	-0.084427	-0.088368	-0.092876	-0.097956	-0.103612	-0.109848	-0.116666
UNIT D.L. ON SPANS 1 AND 3		-0.015966	-0.019488	-0.023437	-0.027826	-0.032666	-0.037968	-0.043743	-0.050000



$$M = \left[\sum w \cdot \text{coef.} \right] \cdot L_I^2 + \sum M_E \cdot \text{coef.}$$

AASHTO Art. 4.6.2.1.3 - Width of Equivalent Interior Strips

- The width of equivalent interior transverse strip over which the wheel loads can be considered distributed longitudinally in concrete decks is given as [Table Art. 4.6.2.1.3-1]

Type Of Deck	Direction of Primary Strip Relative to Traffic	Width of Primary Strip (mm)
Concrete:		
• Cast-in-place	Overhang	$1140 + 0.833X$
	Either Parallel or Perpendicular	$+M$: $660 + 0.55S$ $-M$: $1220 + 0.25S$
• Cast-in-place with stay-in-place concrete formwork	Either Parallel or Perpendicular	$+M$: $660 + 0.55S$ $-M$: $1220 + 0.25S$
• Precast, post-tensioned	Either Parallel or Perpendicular	$+M$: $660 + 0.55S$ $-M$: $1220 + 0.25S$

$+M$ = *positive moment*

$-M$ = *negative moment*

X = *distance from load to point of support (mm)*

S: spacing of supporting components(mm) (T- beams)

AASHTO Art. 4.6.2.1.3—Width of Equivalent Interior Strips . . .

Type Of Deck	Direction of Primary Strip Relative to Traffic	Width of Primary Strip (mm)
Steel:		
• Open grid	Main Bars	$0.007P + 4.0S_b$
• Filled or partially filled grid	Main Bars	Article 4.6.2.1.8 applies
• Unfilled, composite grids	Main Bars	Article 4.6.2.1.8 applies
Wood:		
• Prefabricated glulam ○ Noninterconnected	Parallel Perpendicular	$2.0h + 760$ $2.0h + 1020$
○ Interconnected	Parallel Perpendicular	$2280 + 0.07L$ $4.0h + 760$
• Stress-laminated	Parallel Perpendicular	$0.066S + 2740$ $0.84S + 610$
• Spike-laminated ○ Continuous decks or interconnected panels	Parallel Perpendicular	$2.0h + 760$ $4.0h + 1020$
○ Noninterconnected panels	Parallel Perpendicular	$2.0h + 760$ $2.0h + 1020$

h = depth of deck (mm)
L = span length of deck (mm)
P = axle load (N)
S_b = spacing of grid bars (mm)

Calculation of Force Effects (AASHTO Art. *4.6.2.1.6*)

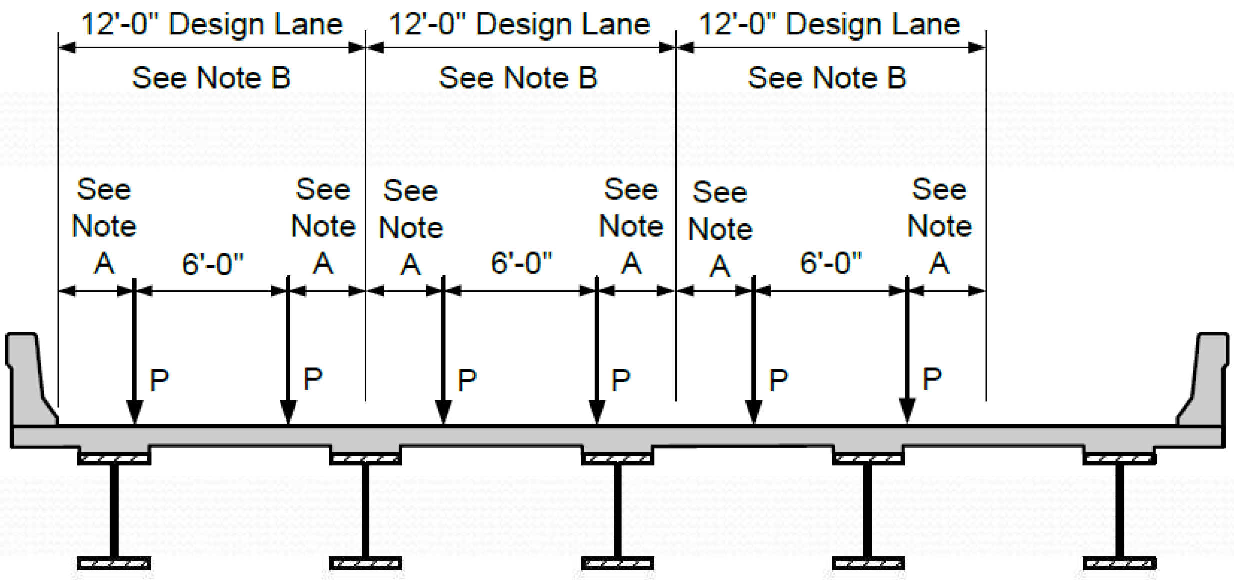
The strips shall be treated as continuous beams or simply supported beams, as appropriate. Span length shall be taken as the center-to-center distance between the supporting components. For the purpose of determining force effects in the strip, the supporting components shall be assumed to be infinitely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

The strips should be analyzed by classical beam theory.

Transverse Configuration for a Design Truck or Design Tandem

In the transverse direction, the design truck and design tandem should be located in such a way that the effect being considered is maximized.



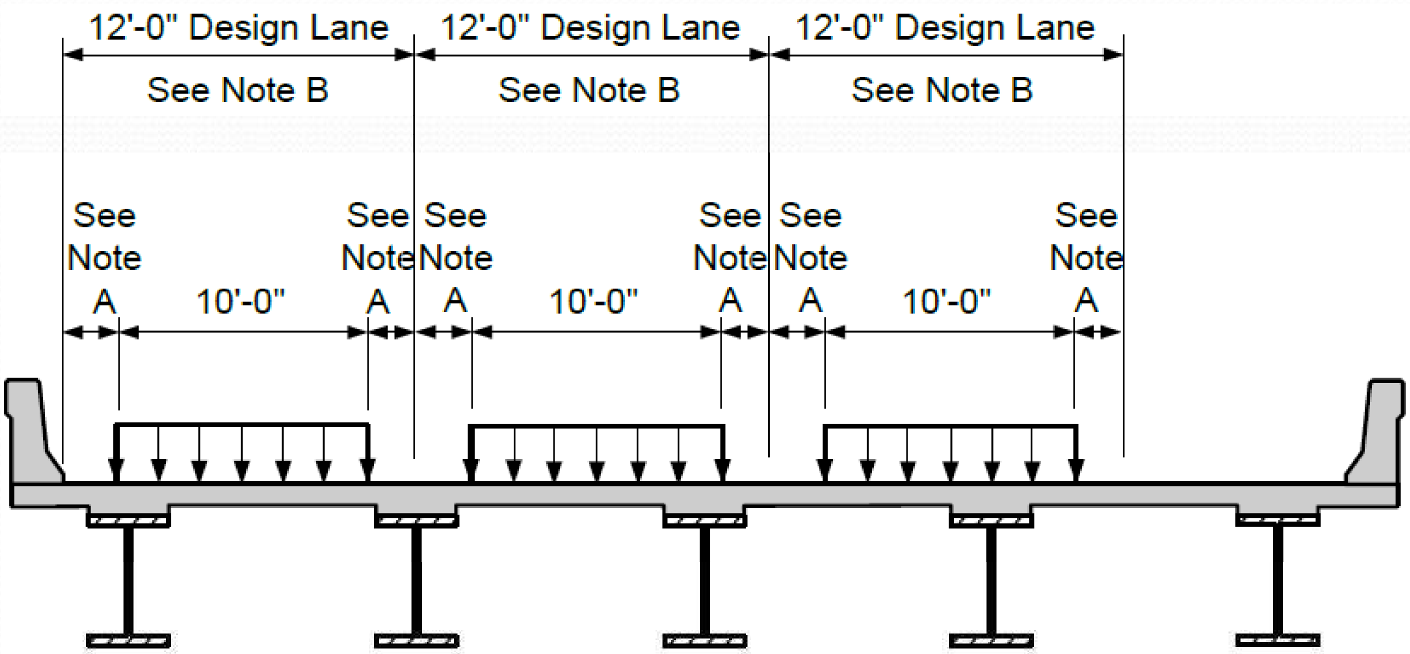
P = Wheel Load

Note A: Position wheel loads within the design lane such that the effect being considered is maximized; minimum = 2'-0".

Note B: Position design lanes across the roadway such that the effect being considered is maximized.

Transverse Configuration for Lane Load

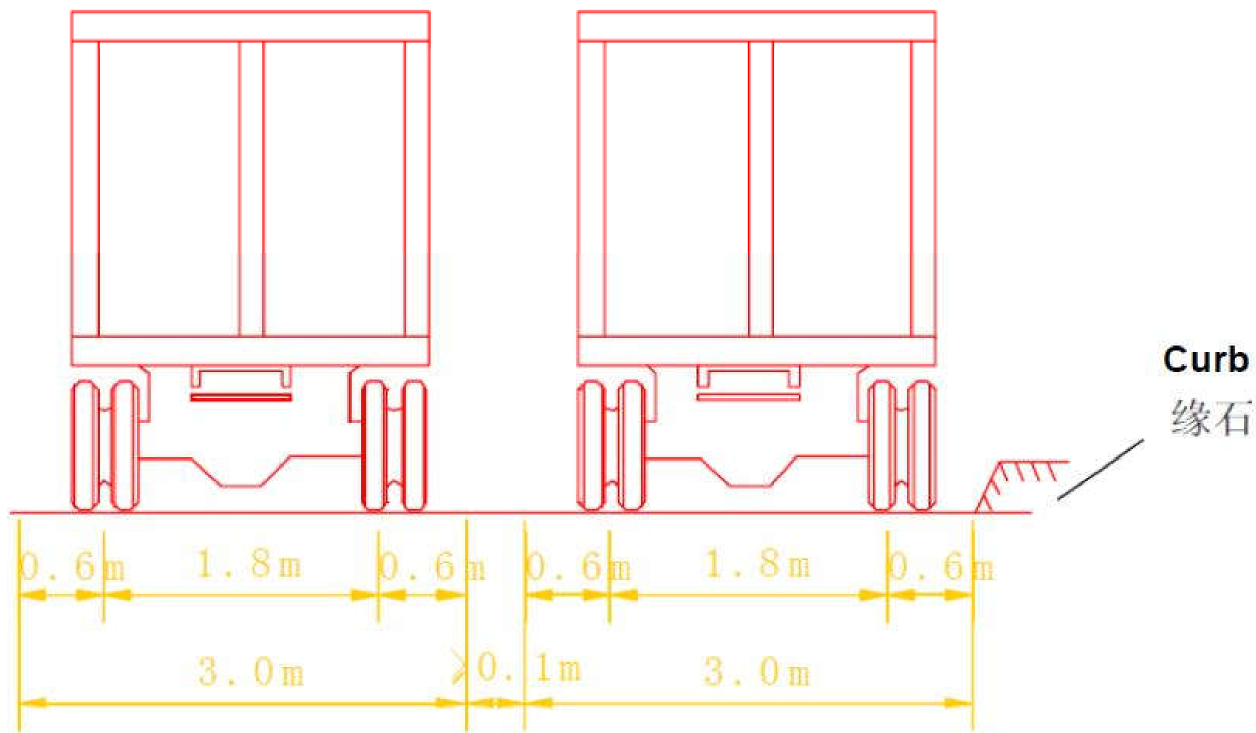
Similarly, the design lane is distributed uniformly over the 10-foot loaded width. Since the design lane is 0.64 kips per linear foot in the longitudinal direction and it acts over a 10-foot width, the design lane load is equivalent to 64 psf.



Note A: Position 10'-0" lane loads within the 12'-0" design lane such that the effect being considered is maximized; minimum = 0'.

Note B: Position 12'-0" design lanes across the roadway such that the effect being considered is maximized.

Transverse Configuration for Live Load . . .



China's City Highway Loads

Critical Placement of Live loads

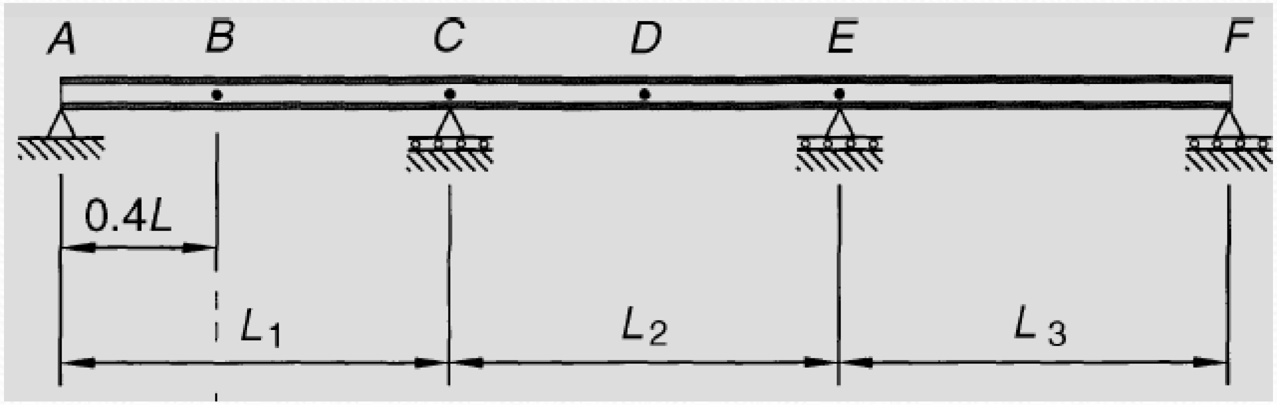


Table 5.1 Span point notation

(Design Of Highway Bridges an LRFD approach , 2nd Edition, Richard M.Barker et. al , 2007)

Span point notation	Alternative Span Point Notation	Span	Percentage	Explanation	Critical Action (Typical)
100	1.00	1	0	Left end of the first span	Shear
104	1.40	1	40	Forty percent of the way across the first span	Positive moment
110	1.10	1	100	Right end of the first span immediately left of the first interior support	Shear, negative moment
200	2.00	2	0	Left end of the second span immediately right of the first interior support	Shear, negative moment
205	2.50	2	50	Middle of the second span	Positive moment

Müller Breslau Principle for Qualitative Influence Lines

In 1886, Heinrich Müller Breslau proposed a technique to draw influence lines quickly. The Müller Breslau Principle states that the ordinate value of an influence line for any function on any structure is proportional to the ordinates of the deflected shape that is obtained by removing the restraint corresponding to the function from the structure and introducing a force that causes a unit displacement in the positive direction.

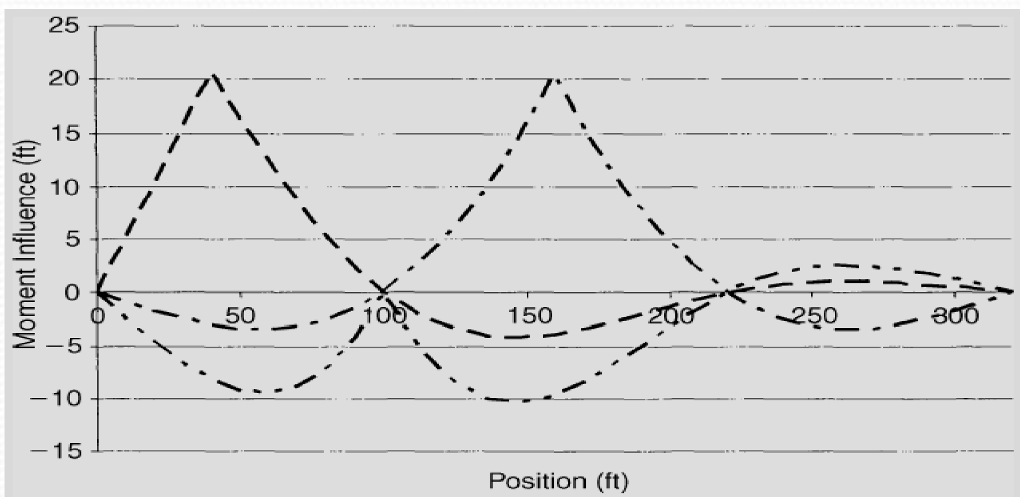
Critical Placement of Live loads

Table 5.2: Influence ordinates and areas (three-span continuous beam)
Richard M.Barker et. al , 2007

Location	Position	*M*(104)	*M*(200)	*M*(205)	*V*(100)	*V*(104)	*V*(110)	*V*(200)	*V*(205)
100	0	0.00	0.00	0.00	1.00	0.00	0.00	0.00	0.00
101	10	5.03	−2.43	−0.88	0.88	−0.12	−0.12	0.03	0.03
102	20	10.11	−4.71	−1.71	0.75	−0.25	−0.25	0.05	0.05
103	30	15.32	−6.70	−2.43	0.63	−0.37	−0.37	0.07	0.07
104	40	20.7	−8.25	−3.00	0.52	−0.48/0.52	−0.48	0.09	0.09
105	50	16.32	−9.21	−3.35	0.41	0.41	−0.59	0.10	0.10
106	60	12.23	−9.43	−3.43	0.31	0.31	−0.69	0.10	0.10
107	70	8.49	−8.77	−3.19	0.21	0.21	−0.79	0.09	0.09
108	80	5.17	−7.07	−2.57	0.13	0.13	−0.87	0.08	0.08
109	90	2.32	−4.20	−1.53	0.06	0.06	−0.94	0.04	0.04
110 or 200	100	0.00	0.00	0.00	0.00	0.00	−1.00	1.00	0.00
201	112	−2.04	−5.09	2.53	−0.05	−0.05	−0.05	0.93	−0.07
202	124	−3.33	−8.33	5.83	−0.08	−0.08	−0.84	0.08	−0.16
203	136	−4.00	−9.99	9.90	−0.10	−0.10	−0.10	0.73	−0.27
204	148	−4.14	−10.34	14.74	−0.10	−0.10	−0.10	0.62	−0.38
205	160	−3.89	−9.64	20.4	−0.10	−0.10	−0.10	0.50	−0.5/0.50
206	172	−3.27	−8.18	14.74	−0.08	−0.08	−0.08	0.38	0.38
207	184	−2.48	−6.21	9.90	−0.06	−0.06	−0.06	0.27	0.27
208	196	−1.60	−4.01	5.83	−0.04	−0.04	−0.04	0.16	0.16
209	208	−0.74	−1.85	2.53	−0.02	−0.02	−0.02	0.07	0.07

Critical Placement of Live loads

Location	Position	*M*(104)	*M*(200)	*M*(205)	*V*(100)	*V*(104)	*V*(110)	*V*(200)	*V*(205)
210 or 300	220	0.00	0.00	0.00	1.00	0.00	0.00	0.00	0.00
301	230	0.46	1.15	−1.53	0.01	0.01	0.01	−0.04	−0.04
302	240	0.77	1.93	−2.57	0.02	0.02	0.02	−0.08	−0.08
303	250	0.96	2.39	−3.19	0.02	0.02	0.02	−0.09	−0.09
304	260	1.03	2.57	−3.43	0.03	0.03	0.03	−0.10	−0.10
305	270	1.00	2.51	−3.35	0.03	0.03	0.03	−0.10	−0.10
306	280	0.90	2.25	−3.00	0.02	0.02	0.02	−0.09	−0.09
307	290	0.73	1.83	−2.44	0.02	0.02	0.02	−0.07	−0.07
308	300	0.51	1.29	−1.71	0.01	0.01	0.01	−0.05	−0.05
309	310	0.27	0.66	−0.88	0.01	0.01	0.01	−0.03	−0.03
310	320	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
Total positive area		1023	165.7	1036.3	45.6	15.4	1.7	66.4	20.1
Total negative area		−305.5	−1371.4	−442.0	−7.60	−17.4	−63.7	−6.4	−20.1
Net area		717.4	−1205.7	594.3	38.0	−2.0	−62.0	60.0	0.00



Example 5.7

Use the trapezoidal rule to determine the positive, negative, and net areas of the influence function for M_{104} in Table 5.2.

$$A_{\text{Span1}} = [0/2 + 5.03 + 10.11 + 15.32 + 20.7 + 16.32 + 12.23 + 8.49 + 5.17 + 2.32 + 0/2](10) = 956.9$$

$$A_{\text{Span2}} = [0/2 + (-2.04) + (-3.33) + (-4.00) + (-4.14) + (-3.89) + (-3.27) + (-2.48) + (-1.60) + (-0.74) + 0/2](10) = -305.8$$

$$A_{\text{Span3}} = [0/2 + 0.46 + 0.77 + 0.96 + 1.03 + 1.00 + 0.90 + 0.73 + 0.51 + 0.27 + 0/2](10) = 66.3$$

These areas are added to give the positive, negative, and net areas:

$$A^{+} = 956.9 + 66.3 = 1023.2 \text{ ft}^2$$

$$A^{-} = -305.8 \text{ ft}^2$$

$$A^{\text{Net}} = 717.4 \text{ ft}^2$$

Table 5.4
Normalized influence functions (three span, span ratio = 1.2)[a]

Location	M(104)	M(200)	M(205)	V(100)	V(104)	V(110)	V(200)	V(205)
100	0.00000	0.00000	0.00000	1.00000	0.00000	0.00000	0.00000	0.00000
101	0.05028	−0.02431	−0.00884	0.87569	−0.12431	−0.12431	0.02578	0.02578
102	0.10114	−0.04714	−0.01714	0.75286	−0.24714	−0.24714	0.05000	0.05000
103	0.15319	−0.06703	−0.02437	0.63297	−0.36703	−0.36703	0.07109	0.07109
104	0.20700	−0.08250	−0.03000	0.51750	−0.4825/0.51750	−0.48250	0.08750	0.08750
105	0.16317	−0.09208	−0.03348	0.40792	0.40792	−0.59208	0.09766	0.09766
106	0.12229	−0.09429	−0.03429	0.30571	0.30571	−0.69429	0.10000	0.10000
107	0.08494	−0.08766	−0.03187	0.21234	0.21234	−0.78766	0.09297	0.09297
108	0.05171	−0.07071	−0.02571	0.12929	0.12929	−0.87071	0.07500	0.07500
109	0.02321	−0.04199	−0.01527	0.05801	0.05801	−0.94199	0.04453	0.04453
110 or 200	0.00000	0.00000	0.00000	0.00000	0.00000	−1.00000/0.0	0.0/1.00000	0.00000
201	−0.02037	−0.05091	0.02529	−0.05091	−0.05091	−0.05091	0.92700	−0.07300
202	−0.03333	−0.08331	0.05829	−0.08331	−0.08331	−0.08331	0.83600	−0.16400
203	−0.03996	−0.09990	0.09900	−0.09990	−0.09990	−0.09990	0.73150	−0.26850
204	−0.04135	−0.10337	0.14743	−0.10337	−0.10337	−0.10337	0.61800	−0.38200
205	−0.03857	−0.09643	0.20357	−0.09643	−0.09643	−0.09643	0.50000	−0.50/0.50
206	−0.03271	−0.08177	0.14743	−0.08177	−0.08177	−0.08177	0.38200	0.38200
207	−0.02484	−0.06210	0.09900	−0.06210	−0.06210	−0.06210	0.26850	0.26850
208	−0.01605	−0.04011	0.05829	−0.04011	−0.04011	−0.04011	0.16400	0.16400
209	−0.00741	−0.01851	0.02529	−0.01851	−0.01851	−0.01851	0.07300	0.07300
210 or 300	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000
301	0.00458	0.01145	−0.01527	0.01145	0.01145	0.01145	−0.04453	−0.04453
302	0.00771	0.01929	−0.02571	0.01929	0.01929	0.01929	−0.07500	−0.07500
303	0.00956	0.02391	−0.03188	0.02391	0.02391	0.02391	−0.09297	−0.09297
304	0.01029	0.02571	−0.03429	0.02571	0.02571	0.02571	−0.10000	−0.10000

(continued)

Table 5.4
(Continued)

Location	*M*(104)	*M*(200)	*M*(205)	*V*(100)	*V*(104)	*V*(110)	*V*(200)	*V*(205)
305	0.01004	0.02511	−0.03348	0.02511	0.02511	0.02511	−0.09766	−0.09766
306	0.00900	0.02250	−0.03000	0.02250	0.02250	0.02250	−0.08750	−0.08750
307	0.00731	0.01828	−0.02437	0.01828	0.01828	0.01828	−0.07109	−0.07109
308	0.00514	0.01286	−0.01714	0.01286	0.01286	0.01286	−0.05000	−0.05000
309	0.00265	0.00663	−0.00884	0.00663	0.00663	0.00663	−0.02578	−0.02578
310	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000
Pos. Area span 1	0.09545	0.00000	0.00000	0.43862	0.13720	0.0000	0.06510	0.06510
Neg. Area Span 1	0.00000	−0.06138	−0.02232	0.00000	−0.09797	−0.56138	0.00000	0.00000
Pos Area Span 2	0.00000	0.00000	0.10286	0.00000	0.00000	0.00000	0.60000	0.13650
Neg. Area Span 2	−0.03086	−0.07714	0.00000	−0.07714	−0.07714	−0.07714	0.00000	−0.13650
Pos. Area Span 3	0.00670	0.01674	0.00000	0.01674	0.01674	0.01674	0.00000	0.00000
Neg. Area Span 3	0.00000	0.00000	−0.02232	0.00000	0.00000	0.00000	−0.06510	−0.06510
Total Pos. Area	0.10214	0.01674	0.10286	0.45536	0.15394	0.01674	0.66510	0.20160
Total Neg. Area	−0.03086	−0.13853	−0.04464	−0.07714	−0.17512	−0.63853	−0.06510	−0.20160
Net Area	0.07129	−0.12179	0.05821	0.37821	−0.02117	−0.62179	0.60000	0.00000

[a] *Usage:*

Multiply influence ordinates for moment by length of span 1.

Multiply areas for moment by length of (span 1)^2.

Multiply areas for shear by length of span 1.

Notes:

Area *M(205)+ for span 2 is 0.1036, 0.1052, and 0.1029 for trapezoidal, Simpson's and exact integration, respectively.*

Areas *V(205)+ and V(205)− for span 2 were computed by Simpson's integration.*

Table 5.7 Action envelopes for three-span continuous beam 100, 120, 100 ft (30480, 36 576 and 30480 mm)[a]

Location, ft	V+, kips 1.33 × Truck + Lane	V+, kips 1.33 × Tandem + Lane	V+, kips Critical	V−, kips 1.33 × Truck + Lane	V−, kips 1.33 × Tandem + Lane	V−, kips Critical	M+, ft kips 1.33 × Truck + Lane	M+, ft kips 1.33 × Tandem + Lane	M+, ft kips Critical	M−, ft kips 1.33 × Truck + Lane	M−, ft kips 1.33 × Tandem + Lane	M−, ft kips 0.9 × 1.33 Train 0.9 × Lane	M−, ft kips Critical
0	**113.9**	94.0	113.9	**−14.5**	−11.7	−14.5	**0.0**	0.0	0.0	**0.0**	0.0	0.0	0.0
10	**96.2**	79.8	96.2	**−14.9**	−12.1	−14.9	**989.9**	825.7	989.9	**−144.6**	−117.2	−137.3	−144.6
20	**79.6**	66.4	79.6	**−20.2**	−21.3	−21.3	**1687.2**	1424.8	1687.2	**−289.3**	−234.5	−274.6	−289.3
30	**64.1**	54.1	64.1	**−32.8**	−31.2	−32.8	**2107.1**	1803.8	2107.1	**−433.9**	−351.7	−411.8	−433.9
40	**49.9**	42.8	49.9	**−46.9**	−41.7	−46.9	**2301.5**	1973.0	2301.5	**−578.5**	−468.9	−549.1	−578.5
50	**37.2**	32.6	37.2	**−61.3**	−52.5	−61.3	**2273.0**	1954.7	2273.0	**−723.2**	−586.2	−686.4	−723.2
60	**26.0**	23.7	26.0	**−75.7**	−63.5	−75.7	**2051.0**	1766.6	2051.0	**−867.8**	−703.4	−823.7	−867.8
70	**16.4**	16.0	16.4	**−90.2**	−74.6	−90.2	**1618.4**	1416.4	1618.4	**−1012.4**	−820.6	−961.0	−1012.4
80	**8.8**	9.5	9.5	**−104.3**	−85.6	−104.3	**1012.1**	928.9	1012.1	**−1157.1**	−937.9	−1098.3	−1157.1
90	**3.6**	4.3	4.3	**−117.9**	−96.3	−117.9	**385.4**	436.0	436.0	−1404.8	−1158.2	**−1535.3**	−1535.3
100	**3.4**	2.8	3.4	**−130.8**	−106.5	−130.8	**341.5**	276.3	341.5	−1835.3	−1561.3	**−2428.4**	−2428.4
100	**132.4**	108.2	132.4	**−13.3**	−10.8	−13.3	**341.5**	276.3	341.5	−1835.3	−1561.3	**−2428.4**	−2428.4
112	**116.8**	95.8	116.8	**−13.6**	−11.0	−13.6	**419.8**	464.5	464.5	−1221.4	−1012.8	**−1398.8**	−1398.8
124	**100.4**	82.8	100.4	**−14.9**	−15.2	−15.2	**1101.9**	990.7	1101.9	**−934.3**	−756.2	−846.6	−934.3
136	**83.8**	69.7	83.8	**−25.3**	−23.7	−25.3	**1750.4**	1516.6	1750.4	−816.4	−668.6	**−819.4**	−819.4
148	**67.6**	56.9	67.6	**−37.8**	−33.6	−37.8	**2151.1**	1847.3	2151.1	−706.5	−589.2	**−819.4**	−819.4
160	**52.1**	44.8	52.1	**−52.1**	−44.8	−52.1	**2271.0**	1954.7	2271.0	−596.7	−509.8	**−819.4**	−819.4
172	**37.8**	33.6	37.8	**−67.6**	−56.9	−67.6	**2151.1**	1847.3	2151.1	−706.5	−589.2	**−819.4**	−819.4
184	**25.3**	23.7	25.3	**−83.8**	−69.7	−83.8	**1750.4**	1516.6	1750.4	−816.4	−668.6	**−819.5**	−819.5
196	**14.5**	15.2	15.2	**−100.4**	−82.8	−100.4	**1101.9**	990.7	1101.9	**−934.4**	−756.2	−846.6	−934.4
208	**13.6**	11.0	13.6	**−116.8**	−95.8	−116.8	**419.8**	464.5	464.5	−1221.4	−1012.9	**−1398.8**	−1398.8
220	**13.3**	10.7	13.3	**−132.4**	−108.2	−132.4	**341.4**	276.3	341.4	−1835.3	−1561.3	**−2428.4**	−2428.4
220	**130.8**	106.5	130.8	**−3.4**	−2.8	−3.4	**341.4**	276.3	341.4	−1835.3	−1561.3	**−2428.4**	−2428.4
230	**117.9**	96.3	117.9	**−3.7**	−4.3	−4.3	**358.7**	436.0	436.0	−1404.8	−1158.2	**−1535.3**	−1535.3
240	**104.3**	85.6	104.3	**−8.8**	−9.5	−9.5	**1012.1**	928.9	1012.1	**−1157.1**	−937.9	−1098.3	−1157.1
250	**90.2**	74.6	90.2	**−16.4**	−16.0	−16.4	**1618.4**	1416.4	1618.4	**−1012.5**	−820.6	−961.0	−1012.5
260	**75.7**	63.5	75.7	**−26.0**	−23.7	−26.0	**2051.1**	1766.6	2051.1	**−867.8**	−703.4	−823.7	−867.8
270	**61.3**	52.5	61.3	**−37.2**	−32.6	−37.2	**2273.1**	1954.8	2273.1	**−723.2**	−586.2	−686.4	−723.2
280	**46.9**	41.7	46.9	**−49.9**	−42.8	−49.9	**2301.5**	1973.1	2301.5	**−578.6**	−468.9	−549.2	−578.6
290	**32.8**	31.2	32.8	**−64.1**	−54.1	−64.1	**2107.1**	1803.8	2107.1	**−433.9**	−351.7	−411.9	−433.9
300	**20.2**	21.3	21.3	**−79.6**	−66.4	−79.6	**1687.2**	1424.8	1687.2	**−289.3**	−234.5	−274.6	−289.3
310	**14.9**	12.1	14.9	**−96.2**	−79.8	−96.2	**989.9**	825.7	989.9	**−144.6**	−117.2	−137.3	−144.6
320	**14.5**	11.7	14.5	**−113.9**	−94.0	−113.9	**0.0**	0.0	0.0	**0.0**	0.0	0.0	0.0

Action Envelope — Live-Load Actions (Critical Values in Bold)

[a] *The truck, tandem, and train vehicle actions are multiplied by the dynamic load allowance of 1.33 prior to combining with the* lane load.

Design of top flange

Design for Flexure

To design the top flange, follow similar procedure with that of RC sections.

* Negative moment should be taken at the face of the girder.

Shear Reinforcement

Slab bridges designed for moment in conformance with Article 4.6.2.3 may be considered satisfactory for shear. (AASHTO Article 5.14.4.1)

Check Serviceability Limit Requirements

Distribution Reinforcement

According to AASHTO, article 5.14.4.1, if the main reinforcement is perpendicular to the traffic, the amount of bottom transverse reinforcement may be taken as a percentage of the main reinforcement and a minimum spacing of 250mm.

$$P_e = \min[67, 3840/\sqrt{Se}]$$

P_e = Percentage of distribution reinforcement

S_e = Clear spacing of girders = G_s-b

Shrinkage & Temperature Reinforcement

As indicated in AASHTO, section 5.10.8, reinforcement for shrinkage and temperature reinforcement shall be provided near surfaces of concrete exposed to daily temperature changes. The specified amount of the steel should be distributed equally on both sides.

$$A_{st} \geq \frac{0.75 A_g}{f_y}$$

Design of Longitudinal Girders

- According to AASHTO, Table 2.5.2.6.3.1, minimum structural depth (including deck) for simple span T- and Box beams is given as follows :

$$D_w = a * L$$

where:

D_w : structural depth for simple span T- and Box beams

L : c/c spacing of the bridge

a = 0.07 for T-girder and 0.06 for Box-girder bridges

For continuous spans:

a=0.065 for T-girder and 0.055 for Box-girder bridges

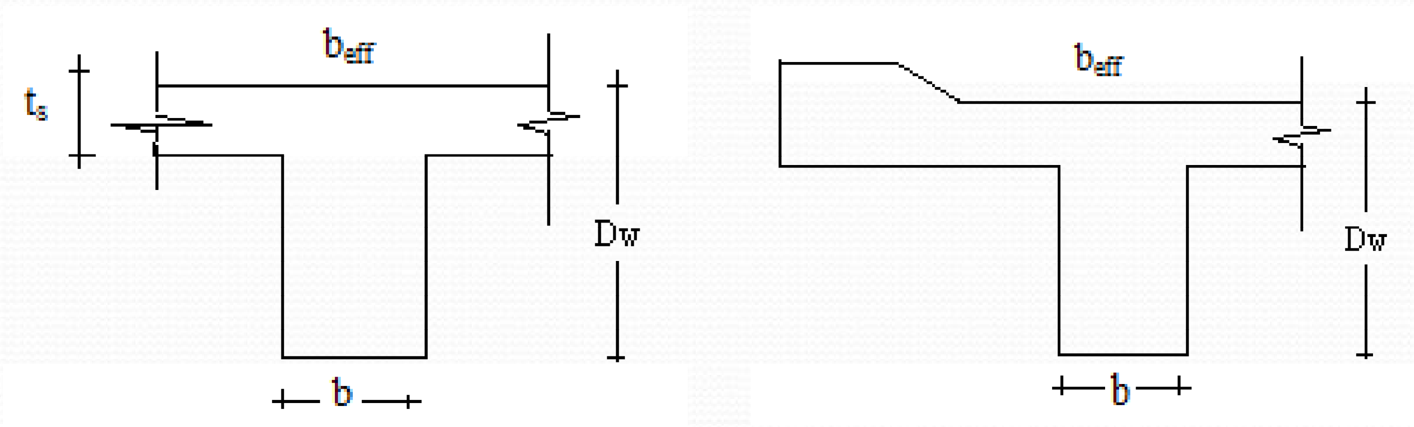
Design of Longitudinal Girders . . .

Web thickness

- The minimum web thickness shall be determined by requirements for shear, torsion, concrete cover and adequate field placement and consolidation of concrete. As per AASHTO, article C5.14.1.5.1C, the minimum web thickness, b_w (mm)is given by

$$b_w = \begin{cases} 200 \text{ mm for webs without prestressing ducts} \\ 300 \text{ mm for webs with only longitudinal or vertical ducts} \\ 380 \text{ mm for webs with both longitudinal and vertical ducts} \end{cases}$$

Design of Longitudinal Girders …



Interior Girder Exterior Girder

Cross sections of T- girder

Effective flange width:

As the longitudinal compressive stress varies across the flange width of same level, it is convenient in design to make use of an effective flange width, (may be smaller than the actual flange width) which is considered to be uniformly stressed.

Design of Longitudinal Girders …

Effective Flange Width for interior and exterior beams *(AASHTO Article, 4.6.2.6)*

For interior beams, the effective flange width may be taken as the least of:

- One-quarter of the effective span length;
- 12 times the average depth of the slab, plus the greater of web thickness or one-half the width of the top flange of the girder; or
- The average spacing of adjacent beams

$$b_i = \min \begin{cases} \dfrac{L_{eff}}{4} \\ 12t_s + b_w \\ s \end{cases}$$

Design of Longitudinal Girders …

Effective Flange Width for interior and exterior beams *(AASHTO Article, 4.6.2.6)*

For exterior beams, the effective flange width may be taken as one-half the effective width of the adjacent interior beam, plus the least of:

- One-eighth of the effective span length;
- 6 times the average depth of the slab, plus the greater of one-half the web thickness or one quarter of the width of the top flange of the basic girder; or
- The width of the overhang

$$b_e - \frac{b_i}{2} = \min \begin{cases} \dfrac{L_{eff}}{8} \\ 6t_s + \dfrac{b_w}{2} \\ \text{width of overhang} \end{cases}$$

Load Distribution Factors for the Girders:

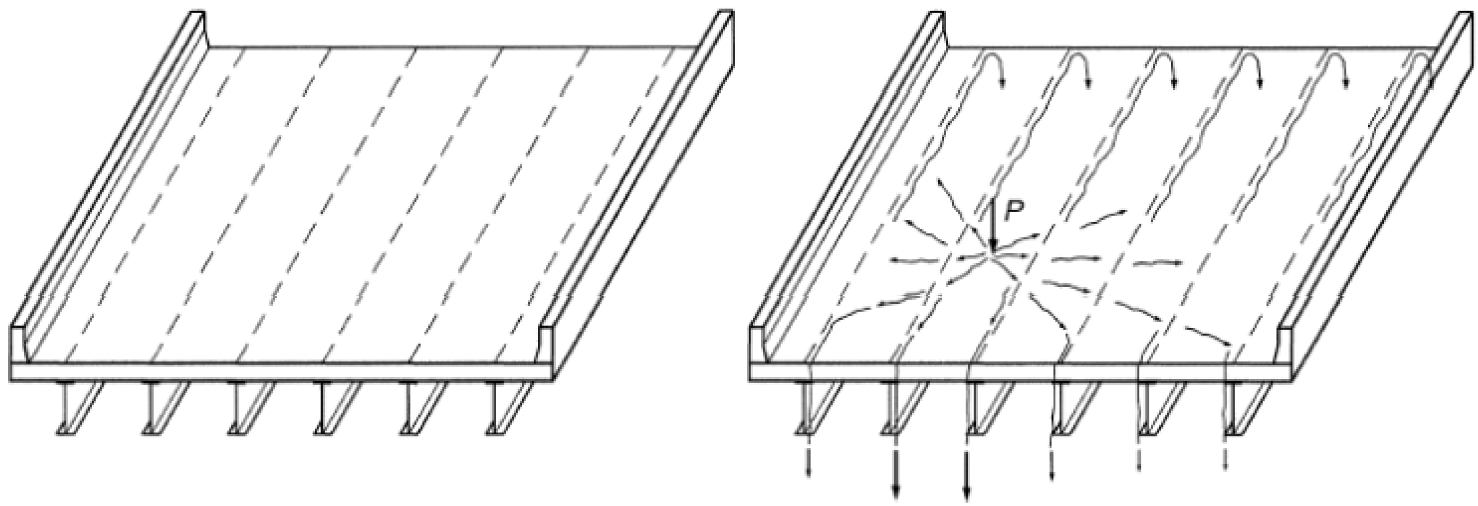
For moment:

- The live load flexural moment for interior and exterior beams with concrete decks shall be determined by applying the lane fractions

For shear:

- The live load shear for interior and exterior beams shall be determined by applying the lane fractions

Load Distribution Factors for the Girders:



The load carried by each girder is a function of the relative stiffness of the components that comprise the slab girder system

Load Distribution Factors for the Girders:

Distribution of Live Loads Per Lane for Moment in Interior and Exterior Longitudinal Beams. (AASHTO, from Tables 4.6.2.2.2b-1 to 4.6.2.2.2e-1)

Distribution of Live Loads Per Lane for shear in Interior and Exterior Longitudinal Beams. (AASHTO, from Tables 4.6.2.2.3a-1 to 4.6.2.2.3c-1)

If the ranges of applicability are not satisfied, then conservative assumptions must be made based on sound engineering judgment.

Table: Distribution of Live Load per Lane for Moment in Interior Beam

Type of Beams	Applicable Cross-section from Figure 13-2	Distribution Factors	Range of Applicability
Concrete Deck on Wood Beams	l	One Design Lane Loaded: S/3700 Two or More Design Lanes Loaded: S/3000	$S \leq 1800$
Concrete Deck, Filled Grid, or Partially Filled Grid 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: $0.06+\left(\frac{S}{4300}\right)^{0.4}\left(\frac{S}{L}\right)^{0.3}\left(\frac{K_g}{Lt_s^3}\right)^{0.1}$ Two or More Design Lanes Loaded: $0.075+\left(\frac{S}{4300}\right)^{0.6}\left(\frac{S}{L}\right)^{0.2}\left(\frac{K_g}{Lt_s^3}\right)^{0.1}$	$1100\leq S\leq 4900$ $110 \leq t_s \leq 300$ $6000\leq L\leq 73000$ $N_b \geq 4$
		Use lesser of the values obtained from the equation above with $N_b = 3$ or the lever rule	$N_b = 3$
Multicell Concrete Box Beam	d	One Design Lane Loaded: $\left(1.75+\frac{S}{1100}\right)\left(\frac{300}{L}\right)^{0.35}\left(\frac{1}{N_c}\right)^{0.45}$ Two or More Design Lanes Loaded: $\left(\frac{13}{N_c}\right)^{0.3}\left(\frac{S}{430}\right)\left(\frac{1}{L}\right)^{0.25}$	$2100\leq S\leq 4000$ $18\ 000 \leq L \leq 73000$ $N_c \geq 3$ If $N_c>8$ use $N_c=8$
Steel Grids on Steel Beams	a	One Design Lane Loaded: S/2300 If $t_g < 100$ mm S/3050 If $t_g \geq 100$ mm Two or More Design Lanes Loaded: S/2400 If $t_g < 100$ mm S/3050 If $t_g \geq 100$ mm	$S \leq 1800$ mm $S \leq 3200$ mm

Load Distribution Factors . . .

Table. Distribution of Live Loads per Lane for Moment in Exterior Longitudinal Beams

Type of Superstructure	Applicable Cross-section from Figure 13-2	One Design Lane Loaded	Two or More Design Lanes Loaded	Range of Applicability
Wood Deck on Wood or Steel Beam	a, l	Lever Rule	Lever Rule	N/A
Concrete Deck on Wood Beams	L	Lever Rule	Lever Rule	N/A
Concrete Deck, filled Grid, or Partially Filled Grid 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	Lever Rule	$g = e\, g_{interior}$ $e = 0.77 + \frac{d_e}{2800}$	$-300 \le d_e \le 1700$
			Use lesser of the values obtained from the equation above with $N_b = 3$ or the lever rule	$N_b = 3$

Load Distribution Factors . . .

Table -Distribution of Live Load per Lane for Shear in Interior Beams

Type of Superstructure	Applicable Cross-section from Figure 13-2	One Design Lane Loaded	Two or More Design Lanes Loaded	Range of Applicability
Concrete Deck on Wood Beams	l	Lever Rule	Lever Rule	N/A
Concrete Deck, Filled Grid, or Partially Filled Grid 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	$0.36+\frac{S}{7600}$	$0.2+\frac{S}{3600}-\left(\frac{S}{10700}\right)^{2.0}$	$1100 \le S \le 4900$ $6000 \le L \le 73000$ $110 \le t_s \le 300$ $4x10^9 \le K_g \le 3x10^{12}$ $N_b \ge 4$
		Lever Rule	Lever Rule	$N_b = 3$
Multi-cell Concrete Box Beams, Box Sections	d	$\left(\frac{S}{2900}\right)^{0.6}\left(\frac{d}{L}\right)^{0.1}$	$\left(\frac{S}{2200}\right)^{0.9}\left(\frac{d}{L}\right)^{0.1}$	$1800 \le S \le 4900$ $6000 \le L \le 73000$ $890 \le d \le 2800$ $N_c \ge 3$

Load Distribution Factors . . .

Table: Distribution of Live Load Per Lane for Shear in Exterior Beams

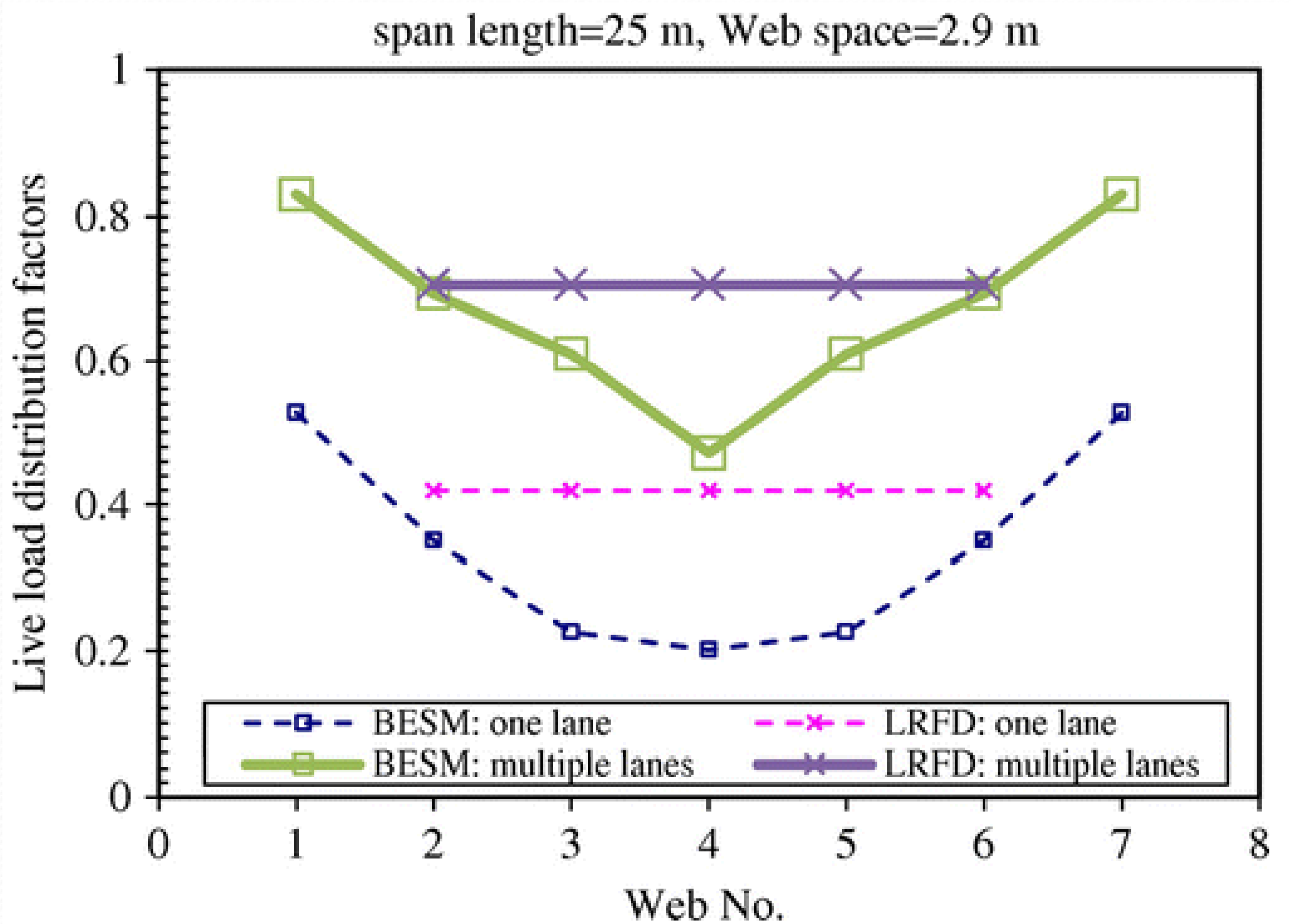
Type of Superstructure	Applicable Cross-section from Figure 13-2	One Design Lane Loaded	Two or More Design Lanes Loaded	Range of Applicability
Wood Deck on Wood or steel Beams	a, 1	Lever Rule	Lever Rule	N/A
Concrete Deck, Filled Grid, or Partially Filled Grid on Steel or Concrete Beams; Concrete T-Beams, T- and Double T-Beams	a, e, k and also i, j if sufficiently connected to act as a unit	Lever Rule	$g = e\, g_{interior}$ $\theta = 0.6 + \frac{d_e}{3000}$	$-300 \le d_e \le 1700$
			Lever Rule	$N_b = 3$
Multi-cell Concrete Box Beams, Box Sections	d	Lever Rule	$g = e\, g_{interior}$ $\theta = 0.64 + \frac{d_e}{3800}$	$-600 \le d_e \le 1500$
Steel Grid Deck on Steel Beams	a	Lever Rule	Lever Rule	N/A

Where: S = spacing between girders (mm)

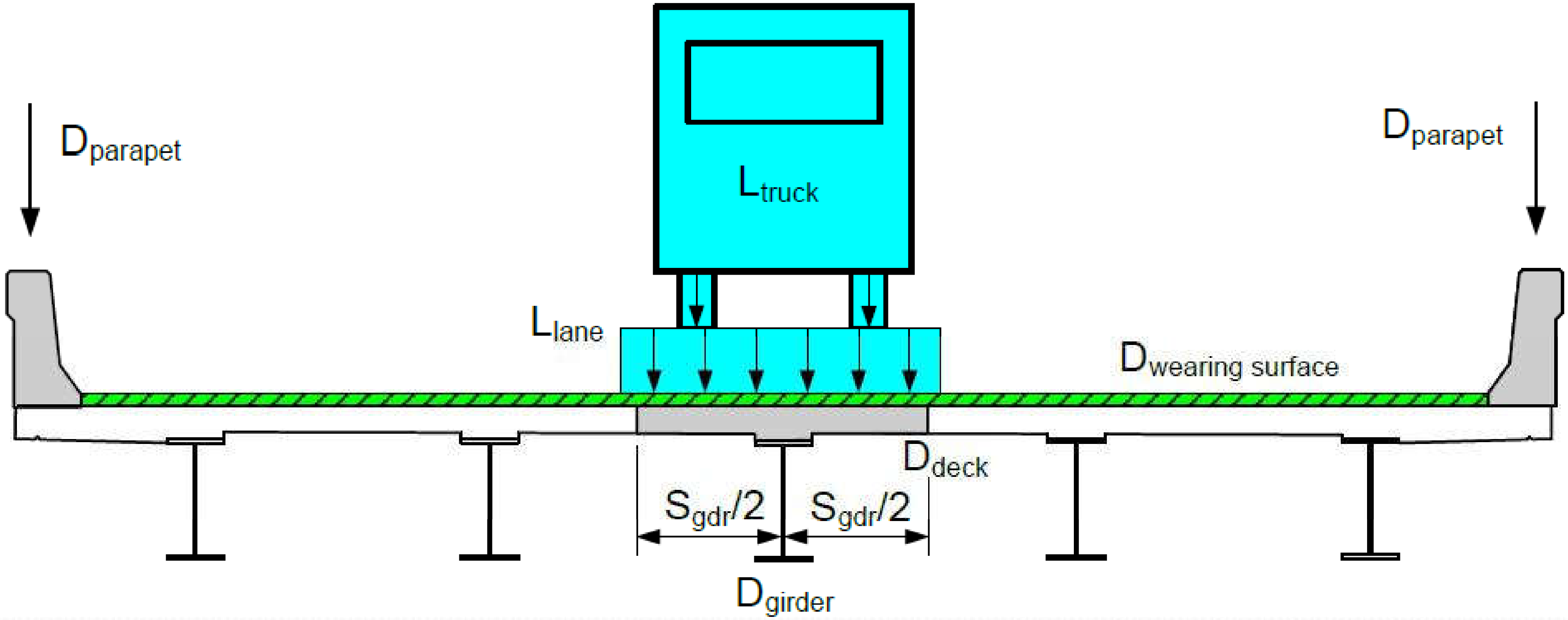
L = Length of Girder (mm)

t_s = thickness of slab (mm)

Load Distribution Factors . . .



Distribution of Loads to Interior Girder for Girder Structure

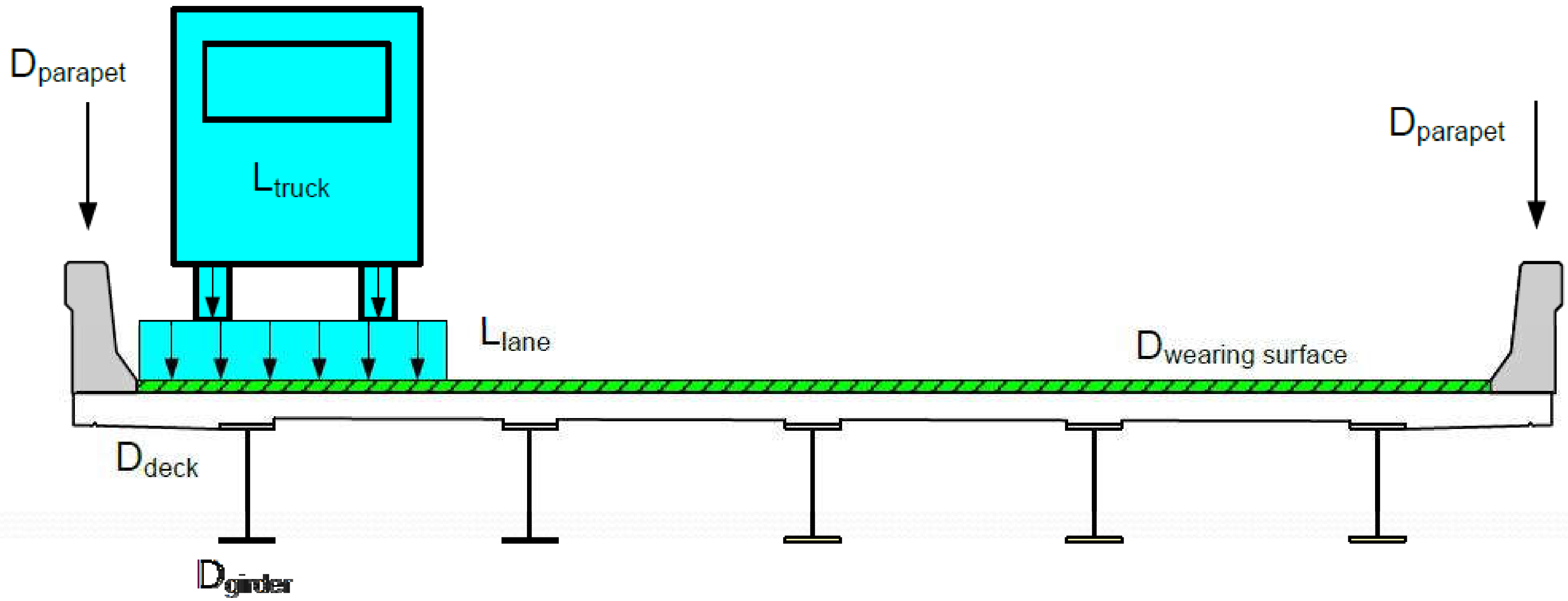


The general equation for loads applied to the interior girder is as follows:

$$\text{Total Load} = D_{girder} + D_{deck} + \left(\frac{D_{wearing\ surface} + 2D_{parapet}}{\text{No. of Girders}}\right) + [(DF_{int})(L_{truck} + L_{lane})]$$

WisDOT Bridge Manual, Jan 2019
State of Wisconsin

Distribution of Loads to Exterior Girder for Girder Structure



The general equation for loads applied to the exterior girder is as follows:

$$\text{Total Load} = D_{girder} + D_{deck} + \left(\frac{D_{wearing\ surface} + 2D_{parapet}}{\text{No. of Girders}}\right) + \left[(DF_{ext})(L_{truck} + L_{lane})\right]$$

WisDOT Bridge Manual, Jan 2019
State of Wisconsin

Design Moment

The design moment is computed by combining the effects of dead loads and live loads and applying the corresponding load combinations and load factors specified in AASHTO, Table 3.4.1.1.

$$M_{sd} = \eta(1.25M_{DL} + 1.5M_{DW} + 1.75M_{LL+IM})$$

For strength limit state η=1.05- for critical or essential bridges.

where:

M_{sd} : design moment

M_{DL} : dead load moment

M_{DW} : dead load moment from wearing surfaces

M_{LL+IM} : moment from live load and impact and given by

$$= 1.33 max\ (M_{tr}, M_{tan}) + M_{ln}$$

M_{tr} : moment from Truck load

M_{tan} : moment from Tandem load

M_{ln} : moment from lane load

Design for flexure

To design the longitudinal girders, follow similar procedure with that of RC sections

Three distinct type of flexural behavior of T-Sections:

1. When the T-Section is subjected to negative BM, tension is produced on the flange portion resulting in a rectangular section of Width b_w.
2. When the T-Section is subjected to positive BM and the equivalent stressed block lies within the flange (if the calculated depth to the NA is less than the slab thickness h_f), the section can be analyzed as rectangular beam with an effective width b_e.
3. When $Y > h_f$, the section can be analyzed as a T-beam.

Design for Shear

Nominal shear resistance

The section is check for maximum shear and thus shear reinforcements are designed.

$$Vn = \min. \begin{cases} Vc+Vs \\ 0.25^{*}f'_c b_v d_v = \quad 2.305 \text{ MPa} \end{cases}$$

$$V_c = 0.083\beta\sqrt{f'_c}\, b_v d_v$$

$$Vs = A_v f_y d/S$$

where: Vn is the nominal shear strength

Vc = shear strength provided by the shear reinforcement

bv = effective web width

dv = effective shear depth

S = spacing of shear reinforcement

Spacings

Determine V_U (the design shear force) at a distance dv from face of support.

$$S \le \frac{A_v f_y d_v \cot\theta}{V_s} \le \frac{A_v f_y}{0.083 b_v \sqrt{f'_c}}$$

$$for \quad V_u < 0.1 f'_c b_v d_v, \qquad S < 0.8 d_v < 600mm$$

$$for \quad V_u \ge 0.1 f'_c b_v d_v, \qquad S < 0.4 d_v < 300mm$$

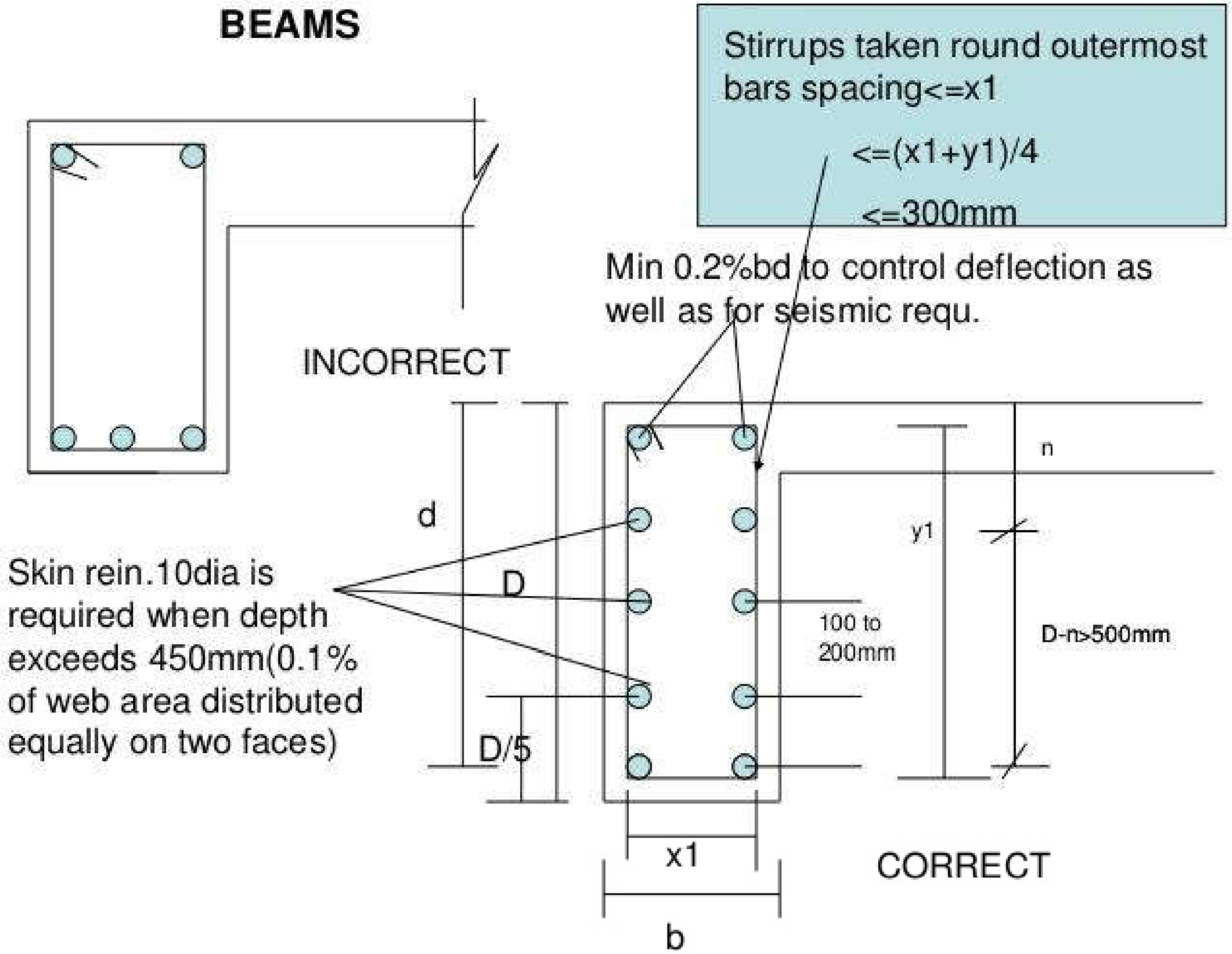
$$\varepsilon_x = \frac{M_D / d_v + 0.5 V_D \cot\theta}{E_s A_s} \le 0.002$$

$$dv = \max \begin{cases} d-a/2 \\ 0.9d \\ 0.72D \end{cases}$$

$a = A_s f_y / 0.8 f'_c b_{eff}$

$Vs = (Vu/\varphi) - Vc, \ \phi = 0.9$

SHEAR AND TORSION REIN. IN BEAMS



Design for Shear . . .

Procedures to compute β

- Assume θ
- Compute ε_x, $\varepsilon_x = \dfrac{M_D / d_v + 0.5V_D \cot\theta}{E_s A_s} \le 0.002$
- Calculate V_n/f'_c (≤ 0.25)
- Obtain θ1 and β from Table 5.8.3.4.2-1
- If θ= θ1 are the same, Use the value of β which was obtained from Table 5.8.3.4.2-1.
- Using β, compute V_c $V_c = 0.083\beta\sqrt{f'_c}\,b_v d_v$
- Otherwise repeat the procedure.

Design for Shear . . .

Table 5.8.3.4.2-1 Values of θ and β for Sections with Transverse Reinforcement.

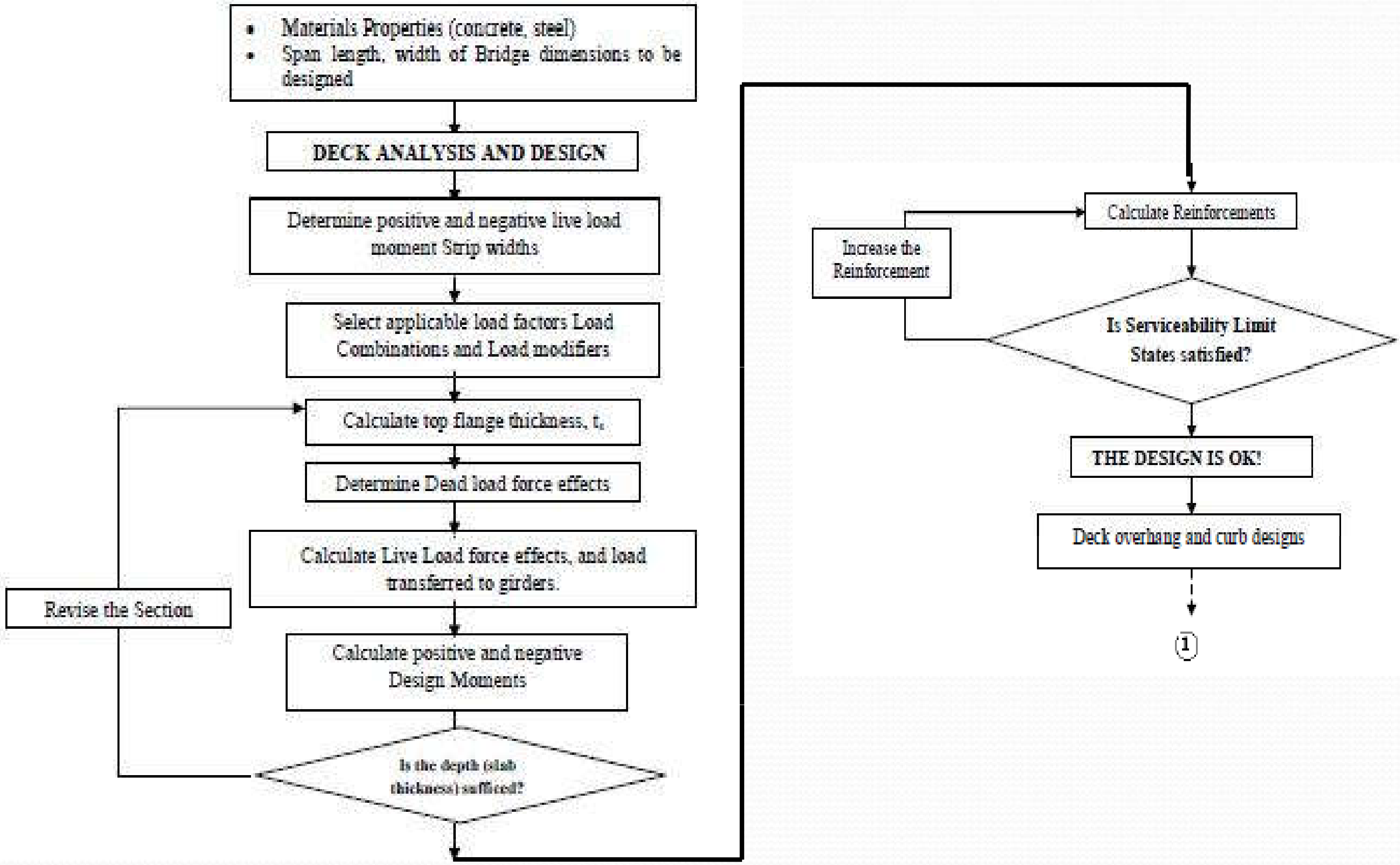
$\frac{v_u}{f'_c}$	$\varepsilon_x \times 1{,}000$								
	≤−0.20	≤−0.10	≤−0.05	≤0	≤0.125	≤0.25	≤0.50	≤0.75	≤1.00
≤0.075	22.3 6.32	20.4 4.75	21.0 4.10	21.8 3.75	24.3 3.24	26.6 2.94	30.5 2.59	33.7 2.38	36.4 2.23
≤0.100	18.1 3.79	20.4 3.38	21.4 3.24	22.5 3.14	24.9 2.91	27.1 2.75	30.8 2.50	34.0 2.32	36.7 2.18
≤0.125	19.9 3.18	21.9 2.99	22.8 2.94	23.7 2.87	25.9 2.74	27.9 2.62	31.4 2.42	34.4 2.26	37.0 2.13
≤0.150	21.6 2.88	23.3 2.79	24.2 2.78	25.0 2.72	26.9 2.60	28.8 2.52	32.1 2.36	34.9 2.21	37.3 2.08
≤0.175	23.2 2.73	24.7 2.66	25.5 2.65	26.2 2.60	28.0 2.52	29.7 2.44	32.7 2.28	35.2 2.14	36.8 1.96
≤0.200	24.7 2.63	26.1 2.59	26.7 2.52	27.4 2.51	29.0 2.43	30.6 2.37	32.8 2.14	34.5 1.94	36.1 1.79
≤0.225	26.1 2.53	27.3 2.45	27.9 2.42	28.5 2.40	30.0 2.34	30.8 2.14	32.3 1.86	34.0 1.73	35.7 1.64
≤0.250	27.5 2.39	28.6 2.39	29.1 2.33	29.7 2.33	30.6 2.12	31.3 1.93	32.8 1.70	34.3 1.58	35.8 1.50

More details on the procedures used in deriving the tabulated values of θ and β are given in Collins and Mitchell (*1991*).

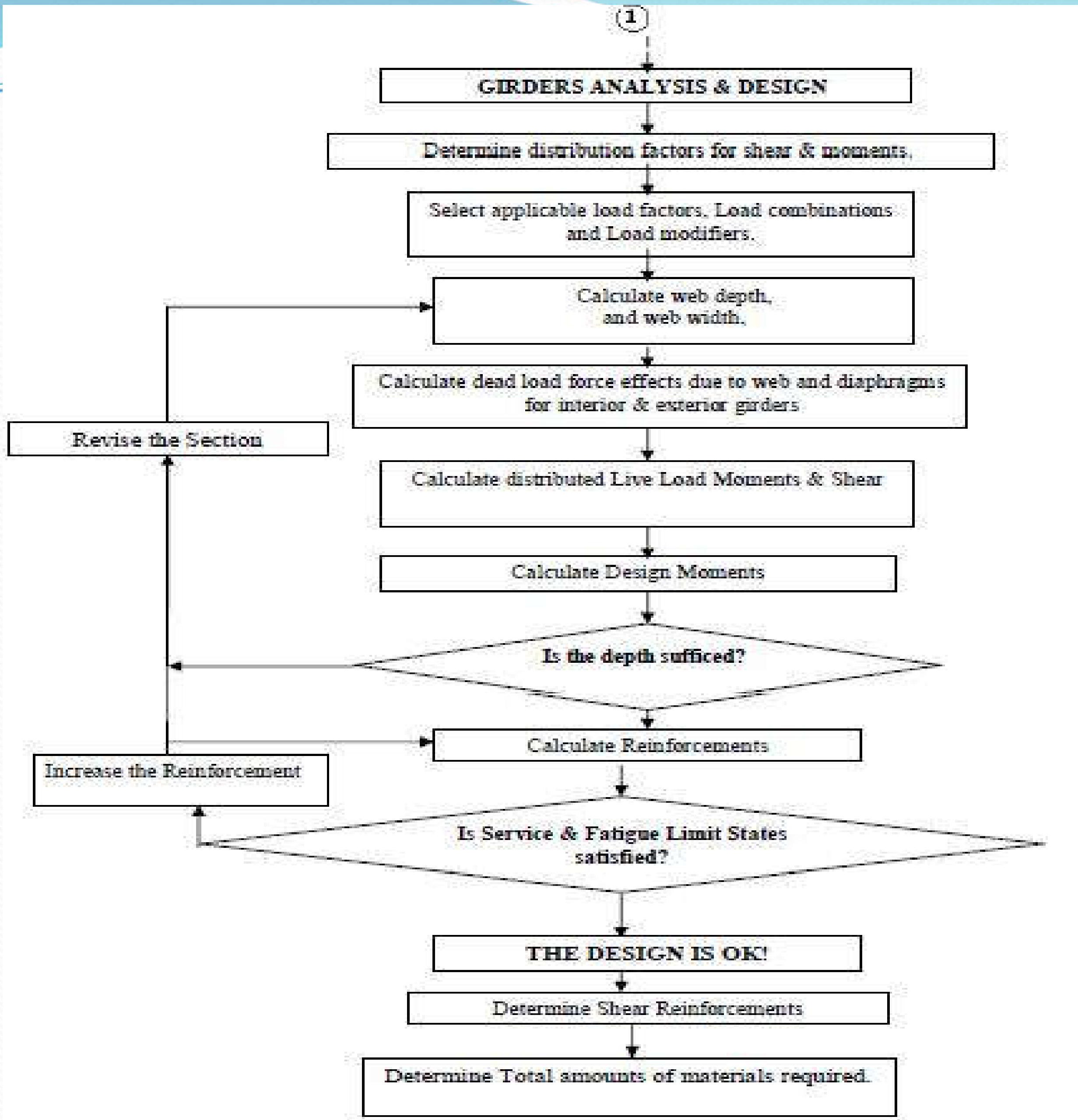
Check:

- Serviceabilty limit requirements,

- fatigue limit state



Materials Properties (concrete, steel)
Span length, width of Bridge dimensions to be designed
DECK ANALYSIS AND DESIGN
Determine positive and negative live load moment Strip widths
Select applicable load factors Load Combinations and Load modifiers
Calculate top flange thickness, t_s
Determine Dead load force effects
Calculate Live Load force effects, and load transferred to girders.
Revise the Section
Calculate positive and negative Design Moments
Is the depth (slab thickness) sufficed?
Calculate Reinforcements
Increase the Reinforcement
Is Serviceability Limit States satisfied?
THE DESIGN IS OK!
Deck overhang and curb designs
1



Flow chart for the design of RC girder bridges

Design Example

T-GIRDER BRIDGE DESIGN

Design Data and Specifications

i) Material Properties

Steel strength, f_y=	400 MPa
Concrete strength, f'_c =	28 MPa
Concrete density, y_c =	2400 kg/m^3
Bituminous density, y_b =	2250 kg/m^3
The modulus of elasticity of steel, E_s =	200 GPa

ii) Bridge Span and Support Dimensions

Clear span of the bridge, C_s =	15 m
Road way/ clear carriage width, R_w =	7.32 m
Bottom width of the concrete barrier/ post, B_c=	0.35 m
Curb width including B_c, (B_c+C_w) =	1.18 m
Curb depth C_d =	0.25 m
Bearing shelf width, W_{rs} =	0.5 m
Girder Spacing, G_s =	2.3 m
Diaphragm Spacing, D_s =	5 m
Number of Diaphragm/s=	4
Skewness=	0 °
Concrete Barrier wall is used.	5 kN/m
Thickness of Asphalt Layer (Wearing Surface) =	100 mm
Concrete Cover for the slab (bottom) =	25 mm
Concrete Cover for the slab (top) =	60 mm
Concrete Cover for the girders =	50 mm

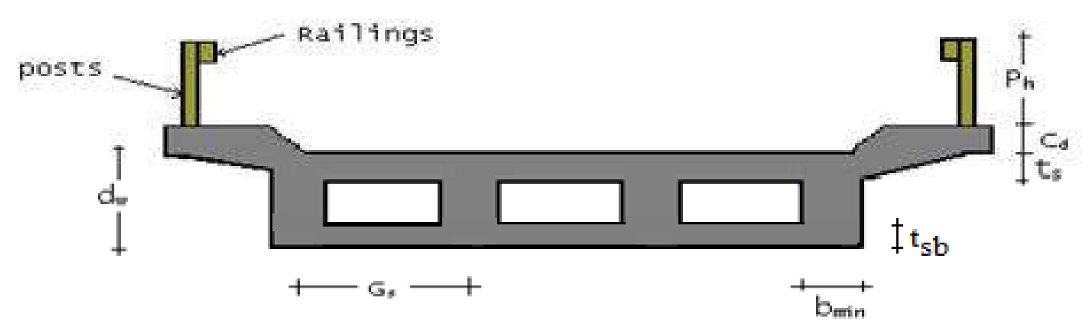
*** Design Method: LRFD**

Specifications:

-AASHTO LRFD Bridge Design Specifications, 4th ed. 2007

-Ethiopian Roads Authority, ERA Bridge Design Manual, 2013

Analysis and Design of Box Girder Bridges



Box Girder Bridges

- Box-girder bridges contain top deck, vertical web, and bottom slab with girders spaced at 1.5 times the structure depth.
- Concrete box girder bridges are economical for spans of above 25 to 45m. (Chinese Standard 15 to 36 m)
- They can be reinforced concrete or prestressed concrete.
- Longer span than 45m will have to be prestressed.
- They are similar to T-beams in configuration except the webs of T-beams are all interconnected by a common flange resulting in a cellular superstructure.

Box Girder Bridge . . .

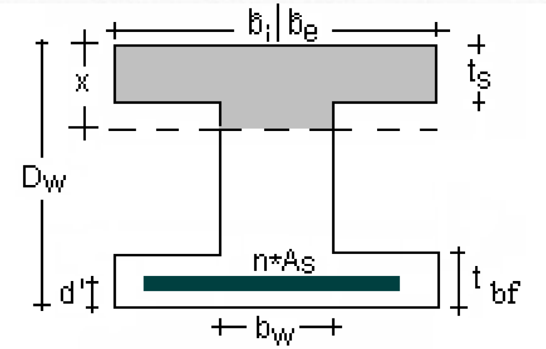
- The top slab, webs and bottom slab are built monolithically to act as a unit, which means that full shear transfer must be provided between all parts of the section.
- Reinforced concrete box girders have high torsional resistance due to their closed shape and are particularly suitable for structures with significant curvature.
- They provide space for utilities such as water and gas lines, power, telephone and cable ducts, storm drains and sewers, which can be placed in the hollow cellular section.
- When the exterior webs are inclined their slope should preferably be 1H: 2V.

Box Girder Bridge . . .

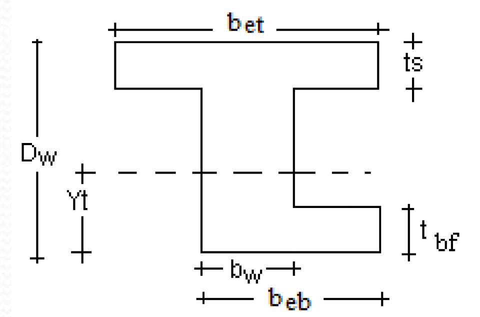
Concrete box girder bridges have several advantages over other types;

- The relatively shallow depth of box girders is all advantage where headroom is limited like in urban overpasses.
- Monolithic construction of the superstructure and substructure offers structural as well as aesthetic advantage.
- The high torsional strength of the box girder makes it particularly suitable for sharp curve alignment, skewed piers and abutments.

Box Girder Bridge . . .

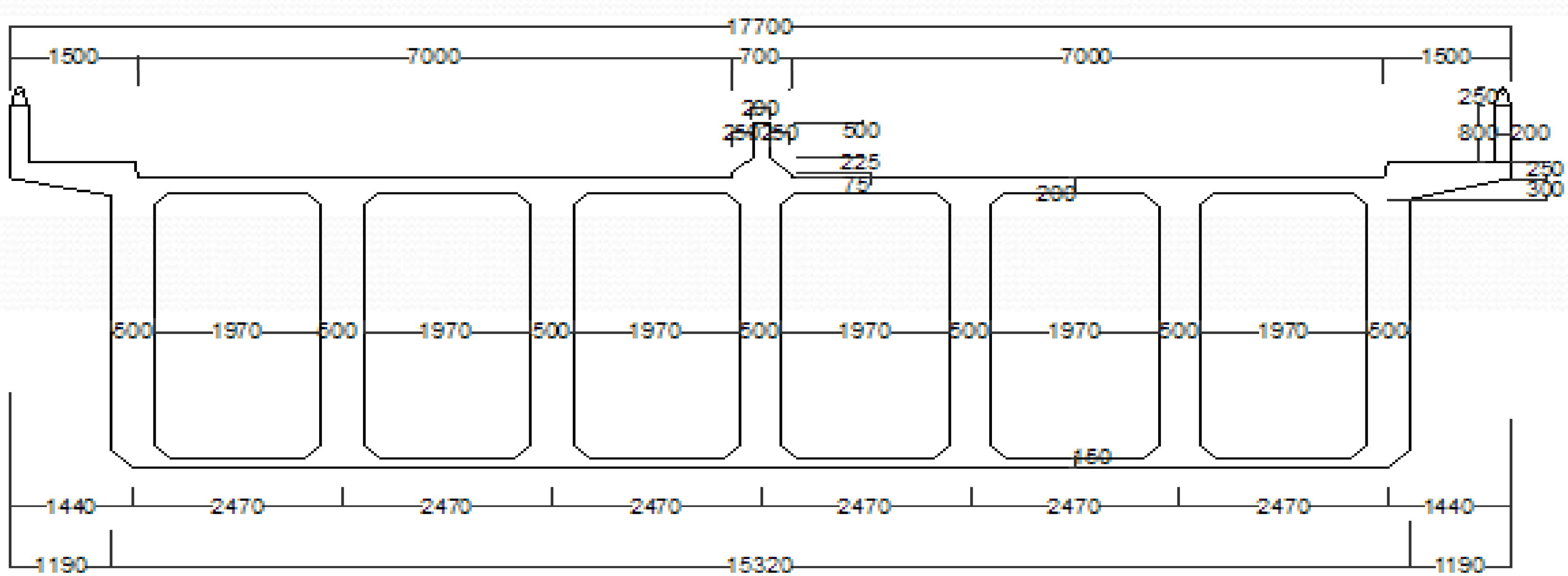


Interior Girder



Exterior Girder

Cross sections of Box girder



Cross-Section for philipos River Bridge.

Box Girder Bridge . . .

The effect of curvature on the torsional behavior of a girder must be considered regardless of the amount of curvature since stability and strength of curved girders is different from that of straight girders (Hall and Yoo, 1996).

In lieu of a refined analysis, Eq. C4.6.1.2.4b-1 may be appropriate for determining the lateral bending moment in I-girder flanges due to curvature (Richardson, Gordon, and Associates, 1976; United States Steel, 1984).

$$M_{lat} = \frac{M\ell^2}{NRD} \qquad \text{(C4.6.1.2.4b-1)}$$

where:

M_{lat} = flange lateral bending moment (kip-ft)

M = major-axis bending moment (kip-ft)

ℓ = unbraced length (ft)

R = girder radius (ft)

D = web depth (ft)

N = a constant taken as 10 or 12 in past practice

Horizontally curved cast-in-place multi-cell concrete box girders may be designed as single-spine beams with straight segments, for central angles up to 34 degrees within one span, unless concerns about other force effects dictate otherwise.

Box Girder Bridges . . .

Effective Flange Width for interior and exterior beams
(AASHTO Article, 4.6.2.6)

For interior beams, the effective flange width maybe taken as the least of:

- One-quarter of the effective span length;
- 12 times the average depth of the slab, plus the greater of web thickness or one-half the width of the top flange of the girder; or
- The average spacing of adjacent beams

$$b_i = \min \begin{cases} \dfrac{L_{eff}}{4} \\ 12t_s + b_w \\ web\ spacing \end{cases}$$

Box Girder Bridges . . .

Effective Flange Width for interior and exterior beams *(AASHTO Article, 4.6.2.6)*

For exterior beams, the effective flange width may be taken as one-half the effective width of the adjacent interior beam, plus the least of:

- One-eighth of the effective span length;
- 6 times the average depth of the slab, plus the greater of one-half the web thickness or one quarter of the width of the top flange of the basic girder; or
- The width of the overhang

$$b_e - \frac{b_i}{2} = \min \begin{cases} \dfrac{L_{eff}}{8} \\ 6t_s + \dfrac{b_w}{2} \\ width\ of\ overhang \end{cases}$$

Box Girder Bridge . . .

Bottom Flange thickness

For Box-girder bridge, as specified in AASHTO, article 5.14.1.5.1b, the bottom flange thickness shall be not less than:

- 140mm;
- the distance between fillets or webs of non prestressed girders and beams divided by 16; or
- the clear span between fillets, haunches, or webs for prestressed girders divided by 30, unless transverse ribs at a spacing equal to the clear span are used.

Box Girder Bridge . . .

Bottom slab Reinforcement

(AASHTO article 5.14.1.5.2b / ERA Bridge design manual 2013 , article 5.5.2.4)

A uniformly distributed reinforcement of 0.4 percent of the flange area shall be placed in the bottom slab parallel to the girder span, either in single or double layers. The spacing of such reinforcement shall not exceed 450mm.

Skewed Bridges

Skewed Bridges

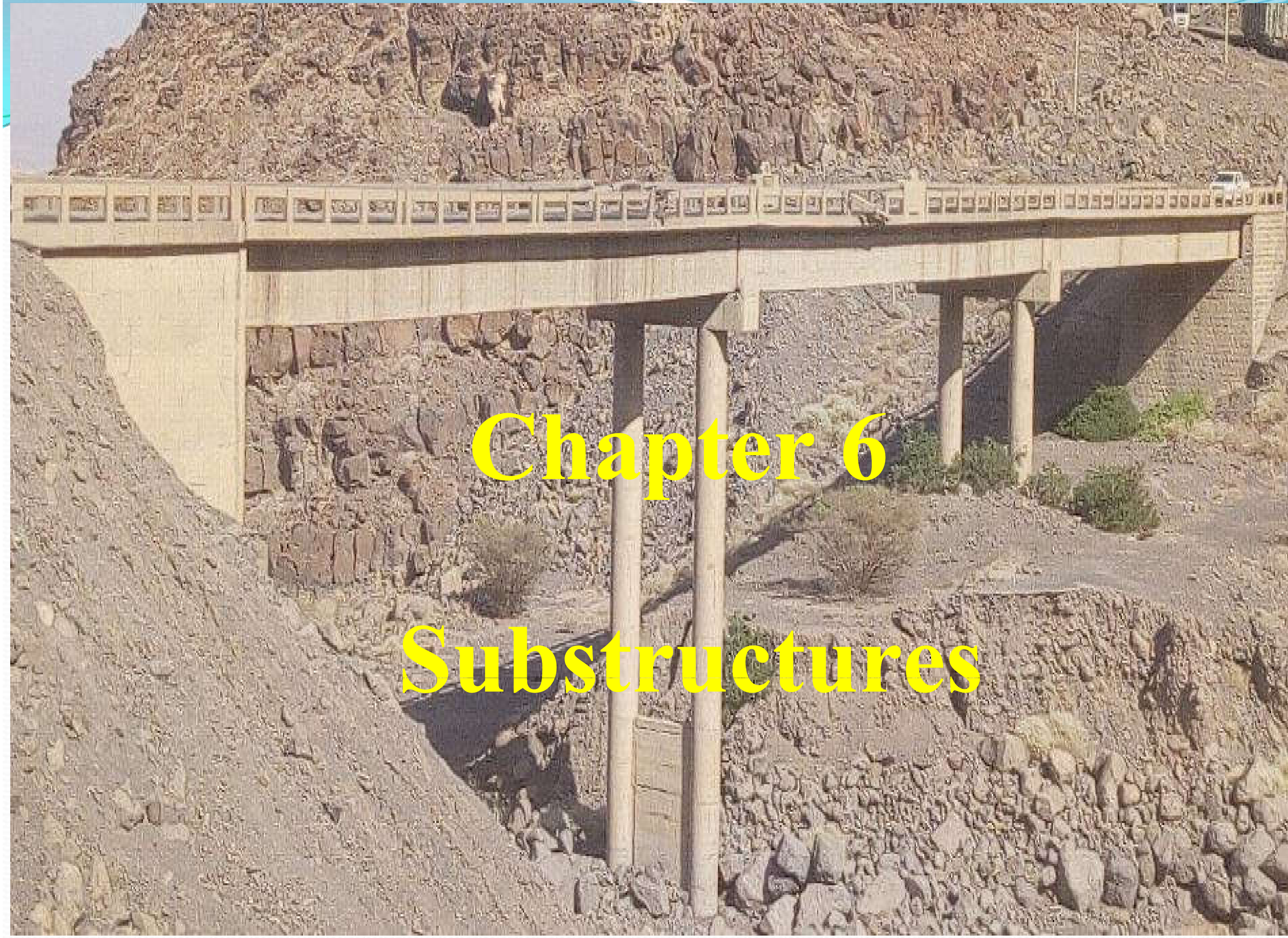
- When the line supports are skewed and the difference between skew angles of two adjacent lines of supports does not exceed 10 degrees, the bending moment in the beams may be .

Table 4.6.2.2.2e-1-Reduction of Load Distribution Factors for Moment in Longitudinal Beams on Skewed Supports

Type of Superstructure	Applicable Cross-Section from Table 4.6.2.2.1-1	Any Number of Design Lanes Loaded	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	$1-c_1(\tan\theta)^{1.5}$ $c_1 = 0.25\left(\frac{K_g}{12.0Lt_s^3}\right)^{0.25}\left(\frac{S}{L}\right)^{0.5}$ If $\theta < 30°$ then $c_1 = 0.0$ If $\theta > 60^0$ use $\theta = 60^0$	$30^0 \le \theta \le 60^0$ $3.5 \le S \le 16.0$ $20 \le L \le 240$ $N_b \ge 4$
Concrete Deck on Concrete Spread Box Beams, Cast-in-Place Multicell Box Concrete Box Beams and Double T-Sections used in Multibeam Decks	b, c, d, f, g	$1.05 - 0.25 \tan\theta \le 1.0$ If $\theta > 60^0$ use $\theta = 60^0$	$0^0 \le \theta \le 60^0$

Table 4.6.2.2.3c-1—Correction Factors for Load Distribution Factors for Support Shear of the Obtuse Corner

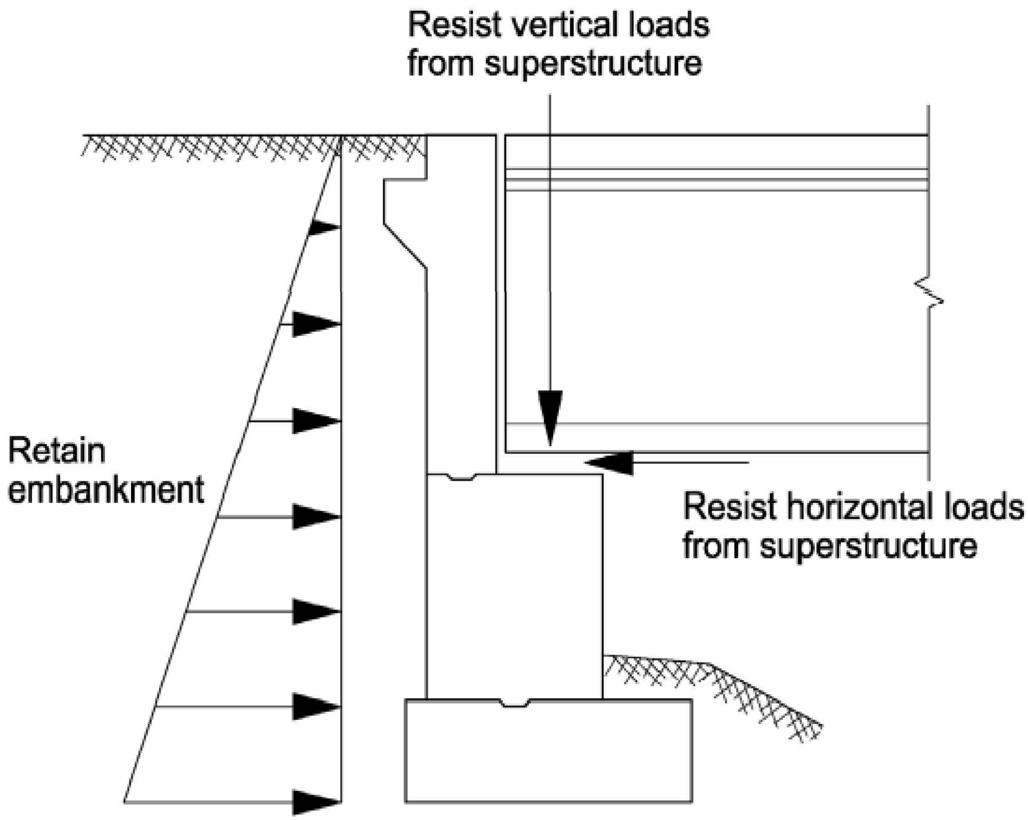
Type of Superstructure	Applicable Cross-Section from Table 4.6.2.2.1-1	Correction Factor	Range of Applicability
Concrete Deck, Filled Grid, Partially Filled Grid, or Unfilled Grid Deck Composite with Reinforced Concrete Slab on Steel or Concrete Beams; Concrete T-Beams, T- and Double T-Section	a, e, k and also i, j if sufficiently connected to act as a unit	$1.0+0.20\left(\frac{12.0Lt_s^3}{K_g}\right)^{0.3}\tan\theta$	$0° \le \theta \le 60°$ $3.5 \le S \le 16.0$ $20 \le L \le 240$ $N_b \ge 4$
Cast-in-Place Concrete Multicell Box	d	$1.0+\left(0.25+\frac{12.0L}{70d}\right)\tan\theta$	$0° < \theta \le 60°$ $6.0 < S \le 13.0$ $20 \le L \le 240$ $35 \le d \le 110$ $N_c \ge 3$
Concrete Deck on Spread Concrete Box Beams	b, c	$1.0+\frac{\sqrt{\frac{Ld}{12.0}}}{6S}\tan\theta$	$0° < \theta \le 60°$ $6.0 \le S \le 11.5$ $20 \le L \le 140$ $18 \le d \le 65$ $N_b \ge 3$
Concrete Box Beams Used in Multibeam Decks	f, g	$1.0+\frac{12.0L}{90d}\sqrt{\tan\theta}$	$0° < \theta \le 60°$ $20 \le L \le 120$ $17 \le d \le 60$ $35 \le b \le 60$ $5 \le N_b \le 20$



Chapter 6

Substructures

Abutments



Abutment is component of a bridge

- provides the vertical support to the bridge superstructure at the bridge ends
- connects the bridge with the approach roadway
- retains the roadway base materials

Abutments and Retaining Structures . . .

Types of Abutments

Abutments can be classified based on the load resisting mechanism as:

1. **Gravity abutments:** resist the load acting with its dead weight and dead weight of retained soil or backfill on its inclined back face.

2. **Cantilever abutments:** load resistance derived from cantilever action and usually constructed from reinforced concrete.

Abutments and Retaining Structures . . .

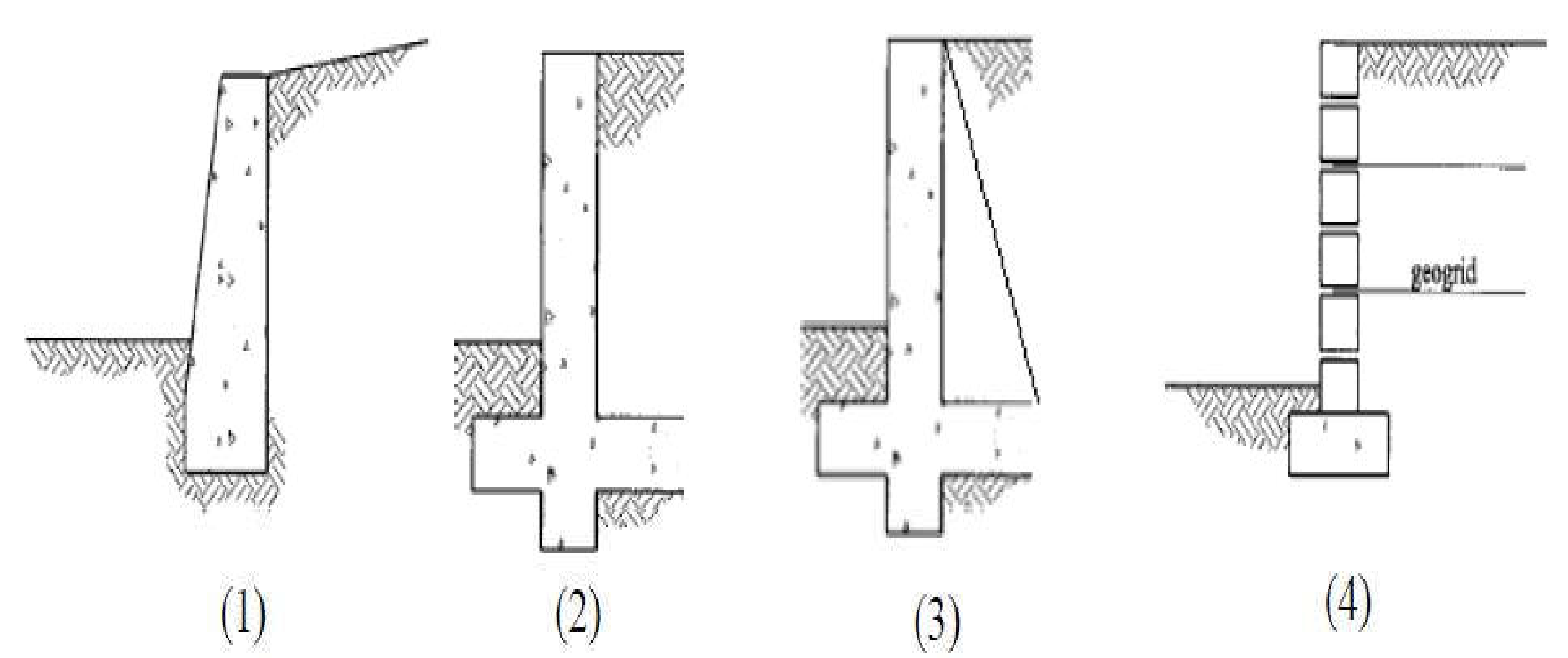
Types of Abutments .....

3. Counterfort abutments: are similar to cantilever ones but ties called counterforts are provided to tie the stem to the footing.

4. Reinforced earth abutments: The essential concept is the use of multiple-layer strips or fibers to reinforce the fill material in the lateral direction so that the integrated fill material will act as a gravity retaining structure

Abutments and Retaining Structures . . .

Types of Abutments …..



Abutments and Retaining Structures . . .

An abutment should be designed so as to withstand damage from:

- Earth pressure (Vertical and lateral soil pressures)
- gravity loads of the bridge superstructure and abutment,
- live load on the superstructure or the approach fill,
- wind loads, and
- transitional loads transferred through the connections between the superstructure and the abutment.
- live-load surcharge on the abutment backfill materials

Procedure for Design of Abutments

Step 1. Select preliminary proportions of the wall.

Step 2. Determine loads and earth pressures.

Step 3. Calculate magnitude of reaction forces on base.

Step 4. Check stability and safety criteria

a. Location of normal component of reactions

b. Adequacy of bearing pressure

c. Safety against sliding.

Procedures . . .

Step 5. Revise proportions of wall and repeat Steps 2-4 until stability criteria are satisfied and then check

a. Settlement within tolerable limits
b. Safety against deep-seated foundation failure.

Step 6. If proportions become unreasonable, consider a foundation supported on driven piles or drilled shafts.

Step 7. Compare economics of completed design with other wall systems.

Abutments and Retaining Structures . . .

Abutment Type Selection

- The selection of an abutment type needs to consider all available information and bridge design requirements. Those may include bridge geometry, roadway and riverbank requirements, geotechnical and right-of-way restrictions, aesthetic requirements, economic considerations, etc.
- Knowledge of the advantages and disadvantages for the different types of abutments is important in choosing the right type of abutment for the bridge structure.

Selection of abutments

1. Construction and maintenance cost.
2. Cut or fill earthwork situation.
3. Traffic maintenance during construction.
4. Construction period.
5. Safety of construction workers.
6. Availability and cost of backfill material.
7. Superstructure depth.
8. Size of abutment.
9. Horizontal and vertical alignment changes.
10. Area of excavation.

Selection of abutments . . .

11. Aesthetics and similarity to adjacent structures.
12. Previous experience with the type of abutment.
13. Ease of access for inspection and maintenance.
14. Anticipated life, loading conditions, and acceptability of deformation

Abutment Wingwall

Abutment wingwalls act as a retaining structure to prevent the abutment backfill soil and the roadway soil from sliding transversely.

A wingwall design similar to the retaining wall design. However, live-load surcharge needs to be considered in wingwall design.

Abutments wing wall . . .

- Wing walls are analyzed as cantilevers extending from the abutment body.
- Wing walls must be long enough to retain the roadway embankment based on the allowable slopes at the abutment. A slope of 2:1 is usually used, and a slope greater than 2:1 is usually not permitted (WisDoT 12.4.1).

- ***During design of abutments and wing walls, all the load modifiers are takes as unity.***

Abutments and Retaining Structures . . .

Abutment Slope Protection

- Flow water scoring may severely damage bridge structures by washing out the bridge abutment support soil.
- To reduce water scoring damage to the bridge abutment, pile support, rock slope protection, concrete slope paving may be used.

Piers

Piers are an integral part of the load path between the superstructure and the foundation. Piers are designed to resist the vertical loads from the superstructure, as well as the horizontal superstructure loads not resisted by the abutments.

- Piers provide vertical supports for spans at intermediate points and perform two main functions:
- transferring superstructure vertical loads to the foundations
- resisting horizontal forces acting on the bridge
- Resisting high lateral loads caused by seismic events.

Piers . . .

Generally piers are subjected to the following loads:

- Vertical and horizontal loads from the bridge superstructure (DL)
- Live loads and impact from the superstructure
- Wind loads on the structure and the live loads
- Centrifugal force from the superstructure
- Longitudinal force from live loads (vehicular braking force)

Piers . . .

- Drag forces due to the friction at bearings
- Stream flow pressure
- Ice pressure
- Earthquake forces
- Thermal and shrinkage forces
- Ship impact forces
- Force due to prestressing of the superstructure
- Forces due to settlement of foundations

Piers . . .

Design Criteria: In general, the design of a highway bridge pier should address:

- Resistance to overturning;
- Resistance to sliding forces;
- Bending strength
  - Bending strength is dependent upon the axial force;
  - Bending moment will be magnified by the axial force due to the P-Δ effect.
- Shear strength In the plastic hinge zone of a pier, the shear strength is also influenced by bending.
- Resistance to earthquake (consideration of ductility)

To design piers, use the classical approch

Flexural resistance of a concrete member is dependent upon the axial force acting on the member.

Interaction diagrams are usually used as aids for the design of the compression members.

Piers . . .

The total cross-sectional area of rectangular hoop (stirrup) reinforcement

The total cross-sectional area A_{sh} of rectangular hoop (stirrup) reinforcement for a rectangular column shall be either

$$A_{sh} = 0.30 a h_c \frac{f'_c}{f_{yh}} \left(\frac{A_g}{A_c} - 1 \right)$$

or

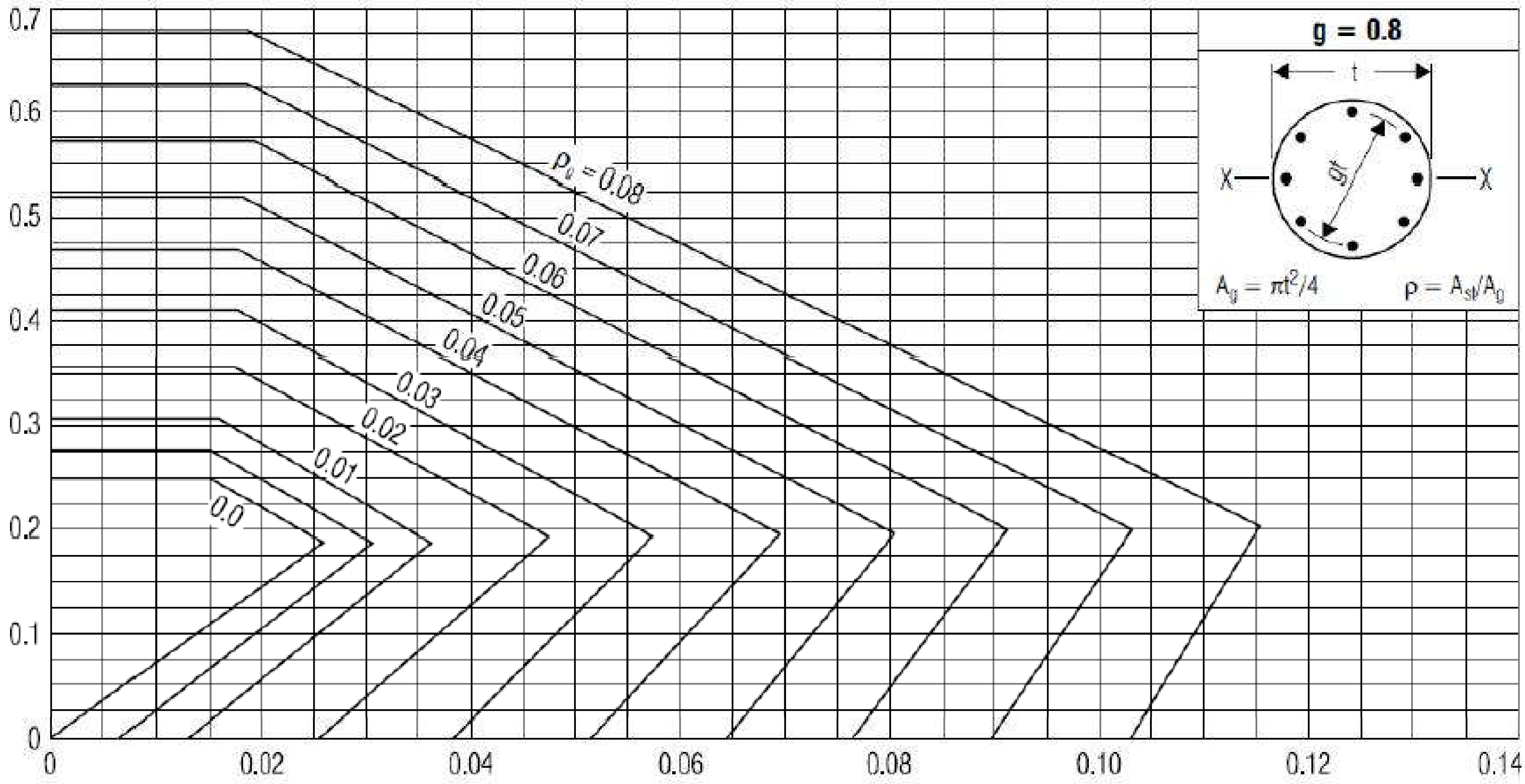
$$A_{sh} = 0.12 a h_c \frac{f'_c}{f_y} \left(0.5 + \frac{1.25 P_u}{A_g f'_c} \right)$$

whichever is greater

where

a = vertical spacing of hoops (stirrups) with a maximum of 100 mm (mm)
A_c = area of column core measured to the outside of the transverse spiral reinforcement (mm^2)
A_g = gross area of column (mm^2)
A_{sh} = total cross-sectional area of hoop (stirrup) reinforcement (mm^2)
f'_c = specified compressive strength of concrete (Pa)
f_{yh} = yield strength of hoop or spiral reinforcement (Pa)
h_c = core dimension of tied column in the direction under consideration (mm)
ρ_s = ratio of volume of spiral reinforcement to total volume of concrete core (out-to-out of spiral)
P_u = factored axial load (MN)

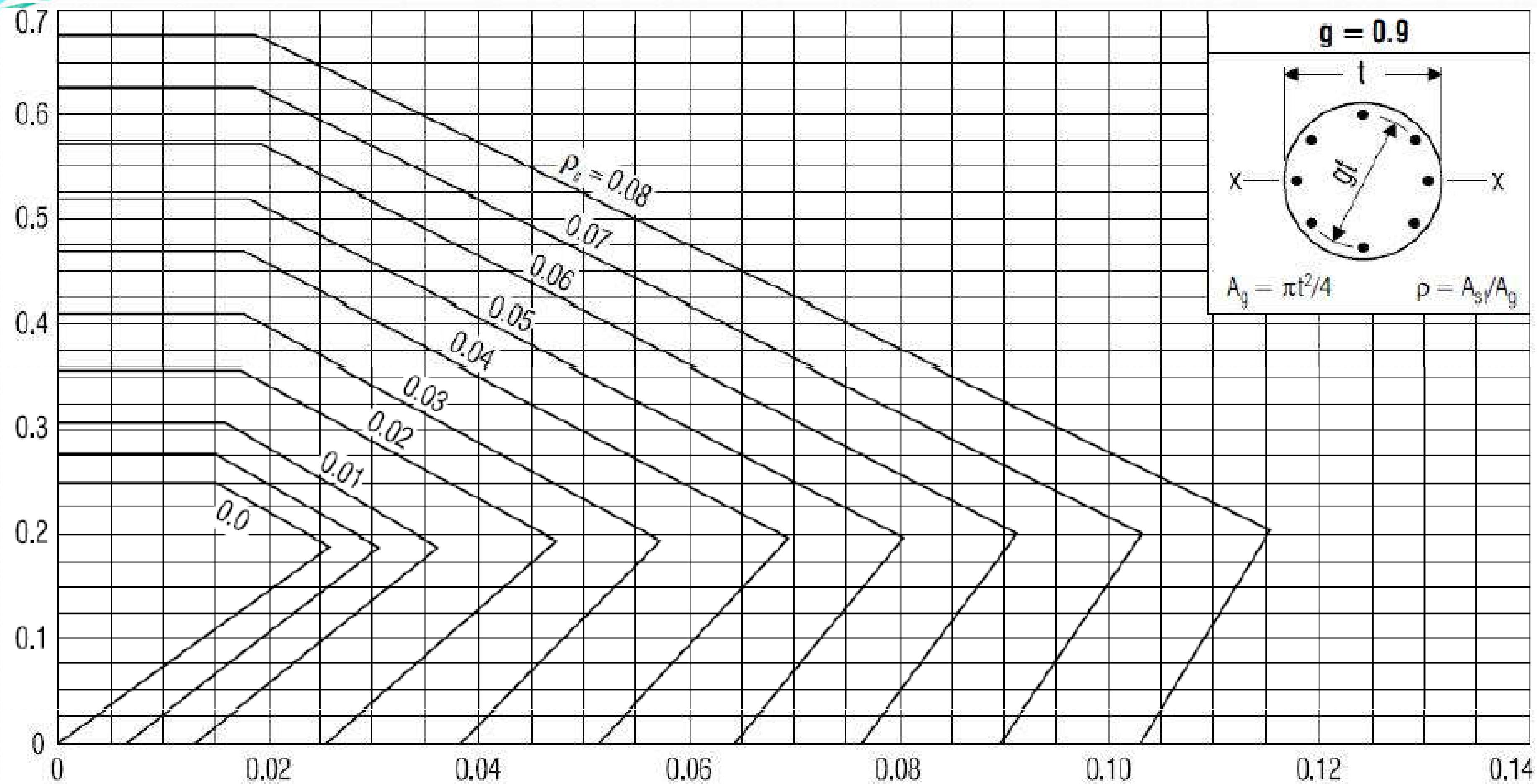
Piers . . .



x-axis: $\dfrac{M}{f'_c \cdot t \cdot A_g}$ y-axis: $\dfrac{P}{f'_c \cdot A_g}$

Interaction diagram for round spiral columns $g = 0.8$, $f'_c = 3{,}000$ psi and $f_y = 40{,}000$ psi *(Adapted from Reinforced Concrete Design Handbook, ACI-SP-3, American Concrete Institute.)*

Piers . . .



x-axis: $\dfrac{M}{f'_c \cdot t \cdot A_g}$ y-axis: $\dfrac{P}{f'_c \cdot A_g}$

Interaction diagram for round spiral columns g = 0.9, f'_c = 3,000 psi and f_y = 40,000 psi *(Adapted from Reinforced Concrete Design Handbook, ACI-SP-3, American Concrete Institute.)*

Shear Strength

Under the normal load conditions, the shear seldom governs the design of the column for conventional bridges since the lateral loads are usually small compared with the vertical loads.

- In seismic design, the shear is very important.
- AASHTO LRFD provides a general shear equation that applies for both beams and columns

Pier Types

Solid Wall Pier: A solid wall pier consists of a solid wall which extends up from a foundation consisting of a footing or piles.

Pier Types . . .

Hammerhead Pier: A hammerhead pier utilizes one or more columns with a pier cap in the shape of a hammer.

Pier Types . . .

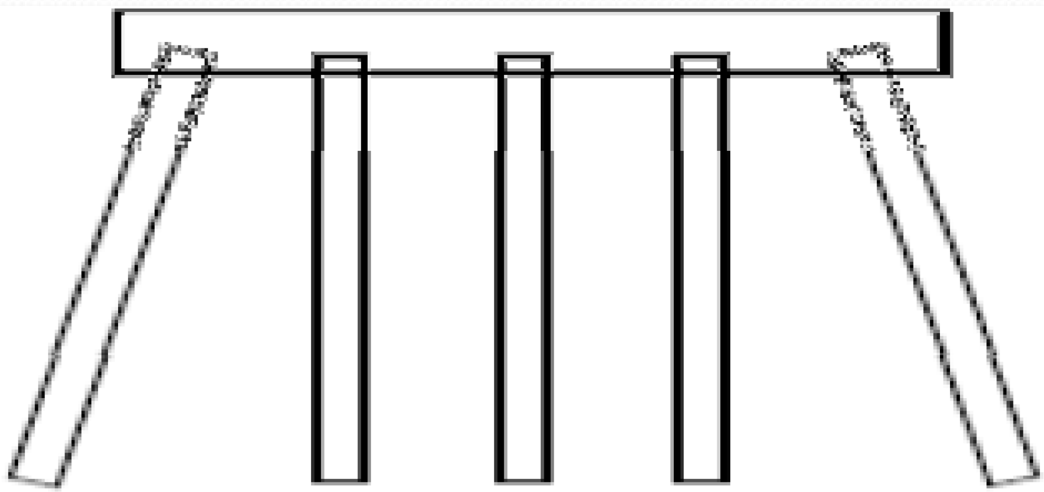
Column Bent Pier: A column bent pier consists of a cap beam and supporting columns in a frame-type structure.

Multi-Column Piers

A minimum of three columns shall be provided to ensure redundancy should a vehicular collision occur.

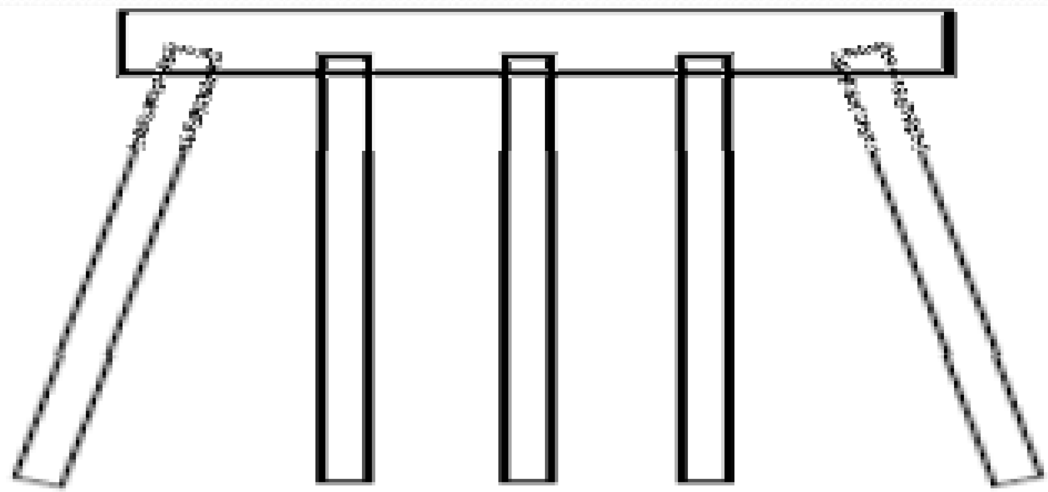
Pier Types . . .

- **Pile Bent Pier:** The pile bent pier is a variation on the column bent pier with the supporting columns and footing replaced with individual supporting piles.



Pier Types . . .

- **Pile Bent Pier:** The pile bent pier is a variation on the column bent pier with the supporting columns and footing replaced with individual supporting piles.



Piers . . .

Location of Piers

- Piers shall be located to provide a minimum interference to flood flow. In general, place the piers parallel with the direction of flood flow.

Load Application

- When determining pier design forces, a thorough understanding of the load paths for each load is critical to arriving at loads that are reasonable per *AASHTO LRFD.*

Loading Combinations

- Piers are designed for the Strength I, Strength III, Strength V and Extreme Event II load combinations as specified in **LRFD [3.4.1].**

Loading Combinations

Load Combination Limit State	Load Factor										
	DC		DW		LL+IM BR CE	WA	WS	WL	FR	TU CR SH	IC CT CV
	Max.	Min.	Max.	Min.							
Strength I	1.25	0.90	1.50	0.65	1.75	1.00	0.00	0.00	1.00	0.5*	0.00
Strength III	1.25	0.90	1.50	0.65	0.00	1.00	1.00	0.00	1.00	0.5*	0.00
Strength V	1.25	0.90	1.50	0.65	1.35	1.00	1.00	1.00	1.00	0.5*	0.00
Service I	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	0.00
Extreme Event II	1.25	0.90	1.50	0.65	0.50	1.00	0.00	0.00	1.00	0.00	1.00

Remark: Reinforced concrete pier components are also checked for the Service I load combination.

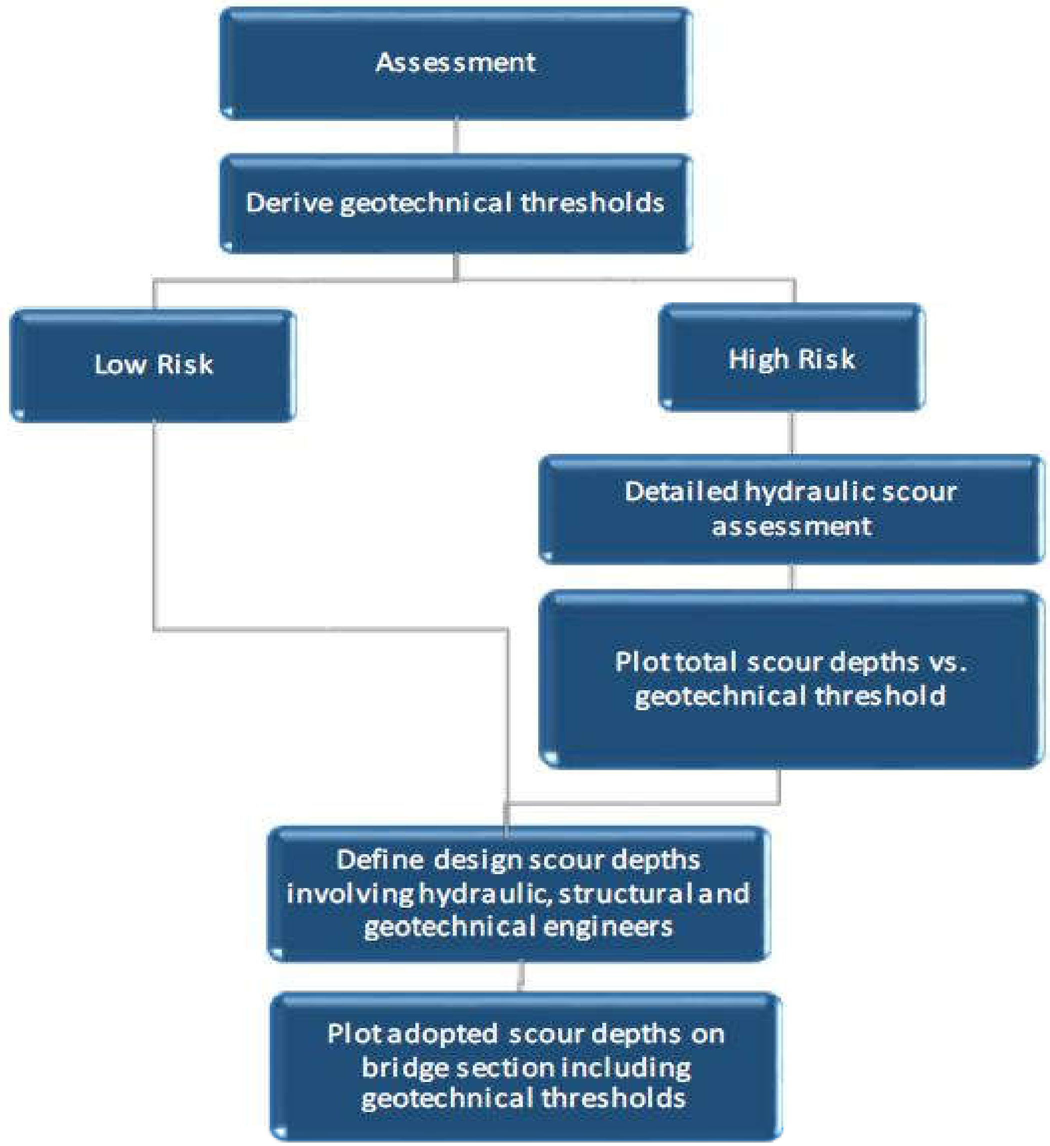
Scour

Scour is the washing away of streambed material by water channel flow. Typically, scour occurs when the water channel becomes narrowed or constricted.

The removal of material from under a pier's foundation, often associated with scour, is known as undermining or undercutting.

- If scour occurs at a specific localized point in the channel, it is known as ***local scour.***
- If scour takes place over a large area of the channel it is known as ***general scour.***

Scour



Recommended scour assessment methodology (Art 5.4.2 ,
Bridge Scour Manual, Austroads Guide to Bridge Technology, Jan. 2019)

Scour

Bridge . No. A1-6-008 along Gewane – Undufo raod segment.

Scour protection

Scour protection is required when some restriction is made to the flow of the flood.

Riprap scour protection

Scour protection . . .

Gabion and Reno mattresses

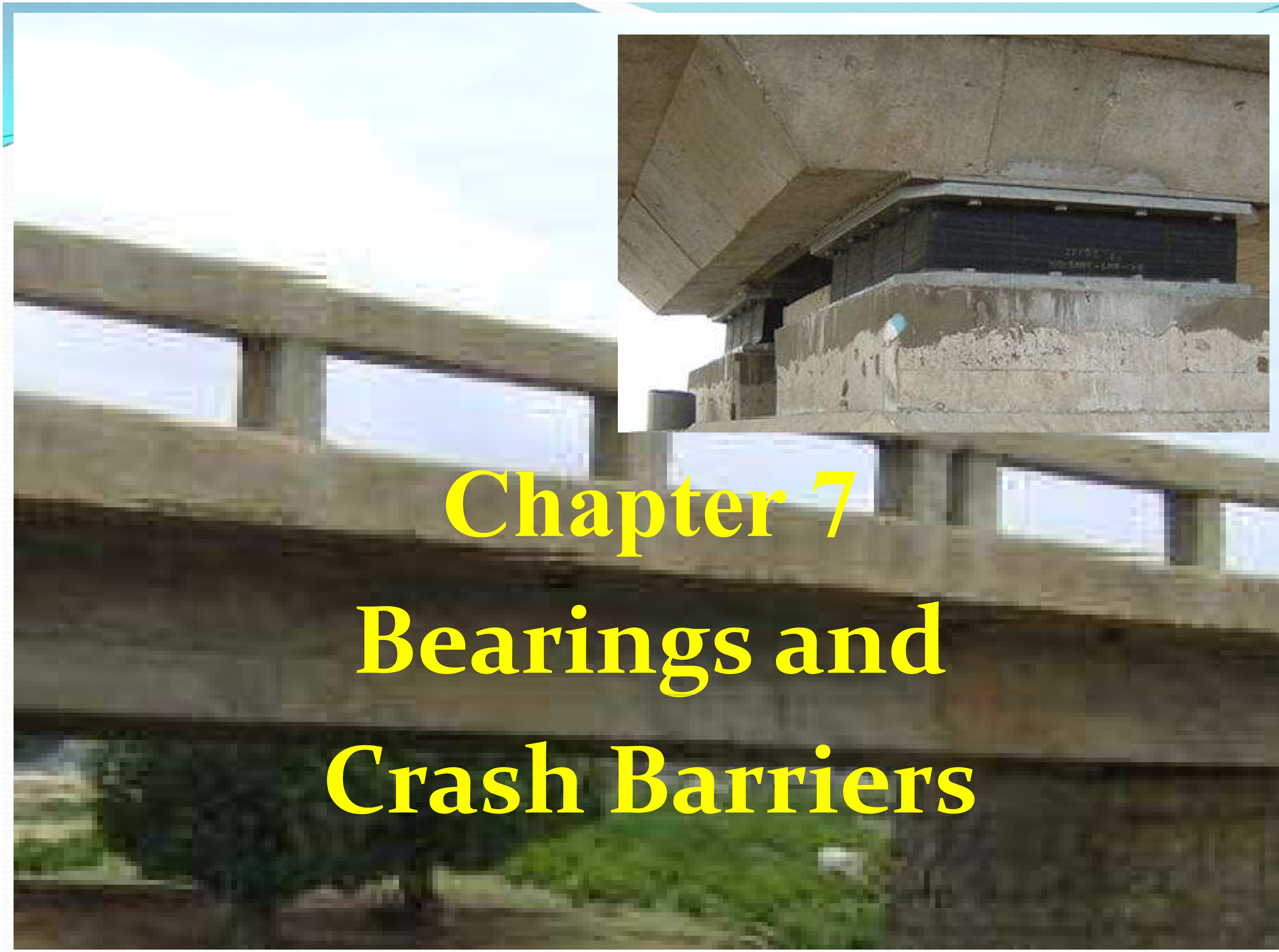
Scour protection . . .

Filter Blankets

Scour protection . . .

Vegetation cover

Chapter 7
Bearings and
Crash Barriers



Bearings

- Bearings are structural devices positioned between the bridge superstructure and the substructure.

Their principal functions are as follows:

1. To transmit loads from the superstructure to the substructure, and
2. To accommodate relative movements between the superstructure and the substructure.

Bearings . . .

- The forces applied to a bridge bearing mainly include superstructure self-weight, traffic loads, wind loads, and earthquake loads.
- Movements in bearings include translations and rotations. Creep, shrinkage, and temperature effects are the most common causes of the translational movements, which can occur in both transverse and longitudinal directions.
- Traffic loading, construction tolerances, and uneven settlement of the foundation are the common causes of the rotations.

Types of Bearings

- Bearings may be classified as fixed bearings and expansion bearings.
- Fixed bearings allow rotations but restrict translational movements.
- Expansion bearings allow both rotational and translational movements.

Types of Bearings . . .

Sliding Bearings

- A sliding bearing utilizes one plane metal plate sliding against another to accommodate translations.

Rocker and Pin Bearings

- It typically consists of a pin at top that facilitates rotations, and a curved surface at the bottom that accommodates the translational movements

Roller Bearings

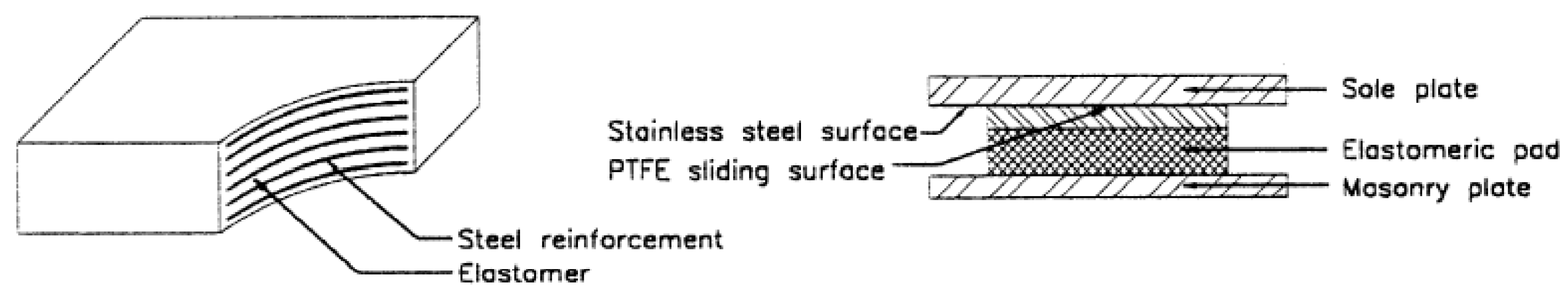
- Roller bearings are composed of one or more rollers between two parallel steel plates.

Types of Bearings . . .

Roller bearings have been used in both steel and concrete bridges.

Elastomeric Bearings

- An elastomeric bearing is made of elastomer (either natural or synthetic rubber). It accommodates both translational and rotational movements through the deformation of the elastomer.



(a) Steel Reinforced Elastomeric Pad

(b) Elastomeric Pad w/PTFE Slider

Selection of Bearings

- Generally the objective of bearing selection is to choose a bearing system that suits the needs with a minimum overall cost. The following procedures may be used for the selection of the bearings.

1. **Determination of Functional Requirements**
2. **Evaluation of Bearings**
3. **Preliminary Bearing Design**

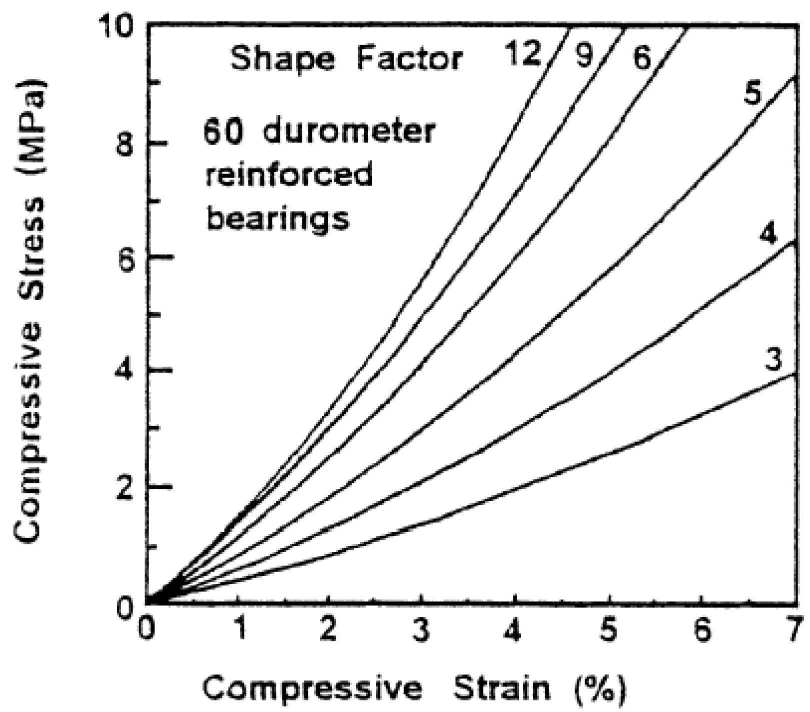
Design of Elastomeric Bearings

The design procedure is according to AASHTO-LRFD and is as follows:

- 1. Determine girder temperature movement
- 2. Determine girder shortenings due to post-tensioning, concrete shrinkage, etc.
- 3. Select a bearing thickness based on the bearing total movement requirements
- 4. Compute the bearing size based on bearing compressive stress
- 5. Compute instantaneous compressive deflection

Design of Elastomeric Bearings . . .

- 6. Combine bearing maximum rotation.
- 7. Check bearing compression and rotation.
- 8. Check bearing stability
- 9. Check bearing steel reinforcement.



Railings

Railings are provided along edges of bridges for protection of traffic and pedestrian. There are three types of railings.

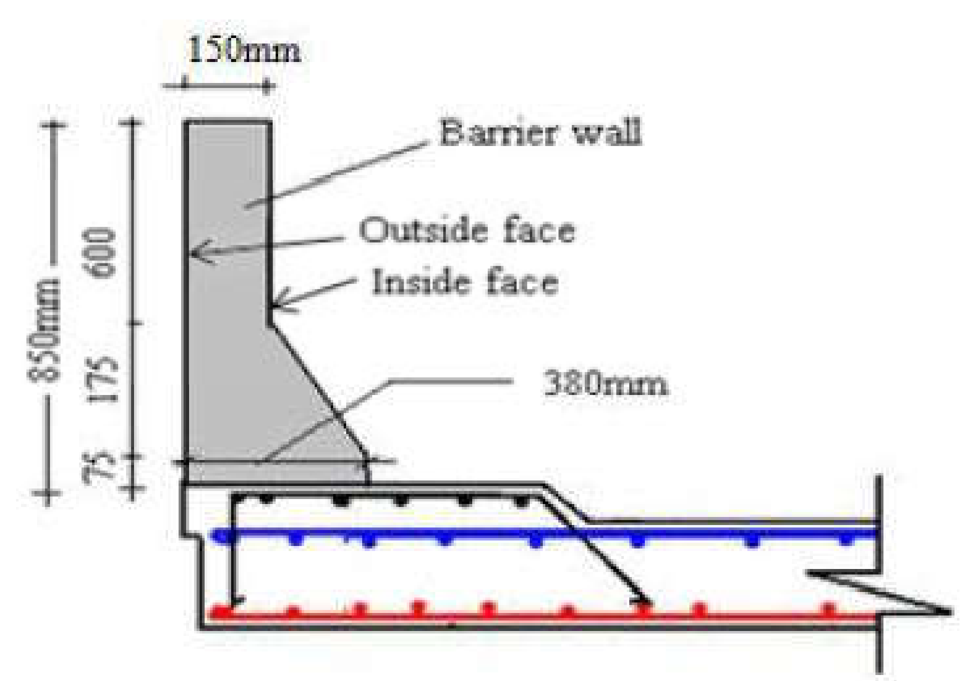
1. Traffic railings
2. Pedestrian railings
3. Combination railings

Purposes

- - primarily containing the average vehicle
- - protecting the occupants of a vehicle in collision with the railing
- - protecting other vehicles near the collision
- - protecting people and property on roadway and nearby areas underneath

Concrete /Crash Barriers

- Crash barriers keep vehicles within their roadway and prevent them from colliding.
- Concrete barriers usually have smooth finishes
- These barrier walls usually have vertical faces to prevent vehicles from climbing the barrier.



Concrete /Crash Barriers

Design

- Strength limit state and extreme event limit state are considered for the design of barriers. The design forces for a TL-4 barrier as per AASHTO: Table A13.2.1 is used.

Design forces for a TL-4 barrier (AASHTO: Table A13.2.1)

Direction	Force (KN)	Length(m)
Transverse (F_t)	240	1.07
Longituidinal (F_l)	80	1.07
Vertical (F_v)	80	5.5

Concrete /Crash Barriers

$$F_t = R_w$$

$$R_w = \frac{2}{2l_c - L_t}\left(8M_b + 8M_w + \frac{M_c L_c^2}{H}\right) \quad \text{and} \quad L_c = \frac{l_t}{2} + \sqrt{\left(\frac{L_t}{2}\right)^2 + \frac{8H(M_b + M_{wH})}{M_c}}$$

Where:

M_b - additional flexural resistance of beam in addition to M_w, if any, at top of wall (kN-m)

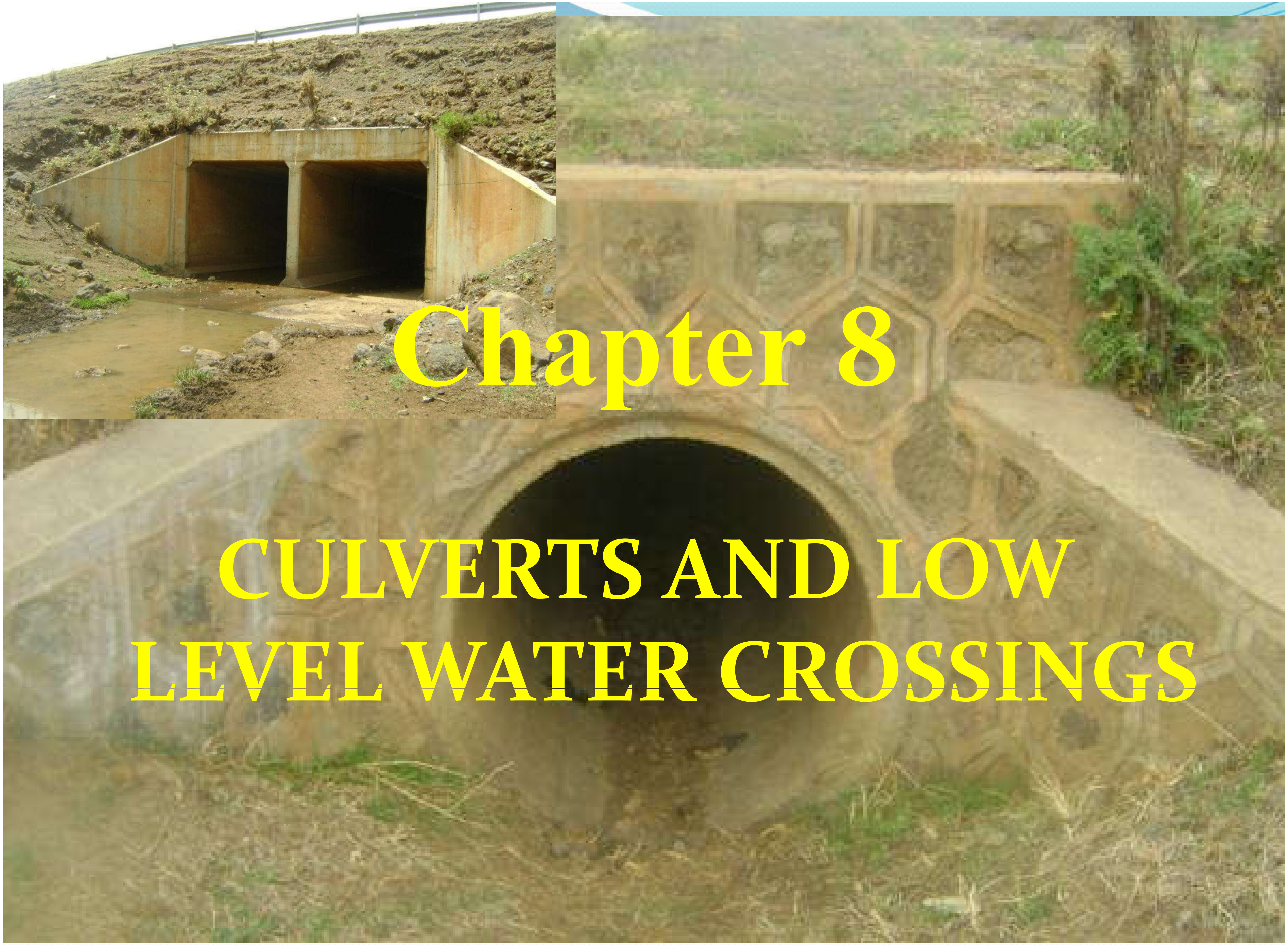
M_w - flexural resistance of the wall about its vertical axis (kN-m)

Mc - flexural resistance of cantilevered walls about an axis parallel to the longitudinal axis of the bridge (kN-m/m)

H - height of wall

L_t- longitudinal length of distribution of impact force, Ft (kN)

L_c - critical length of yield line failure pattern (m)



Chapter 8

CULVERTS AND LOW LEVEL WATER CROSSINGS

CULVERTS

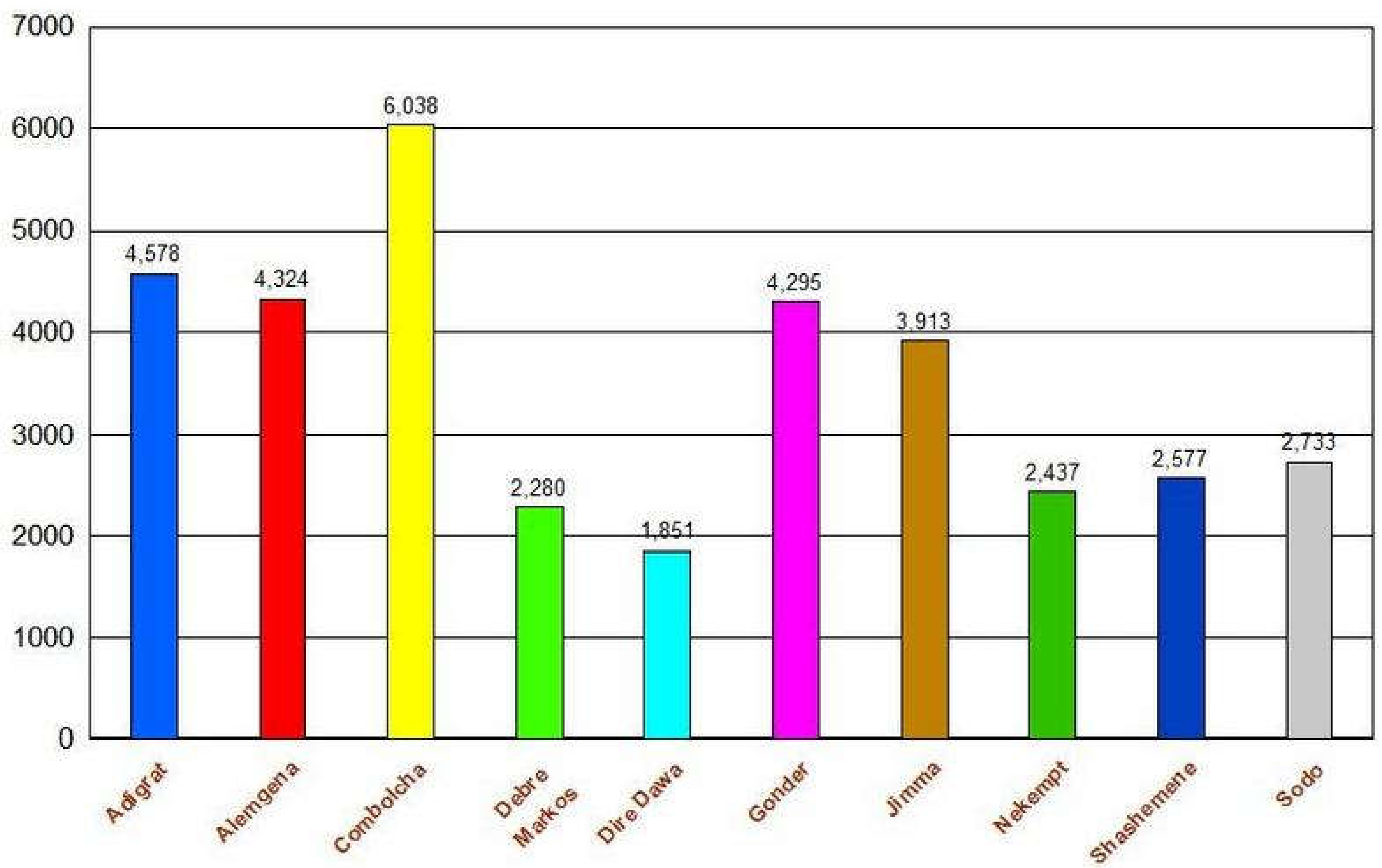
- Culverts are cross drainage structures used to drain rain run off collected by side ditches from one side of the highway to the other.
- Span<4m
- Where the waterway opening is less than about 15m^2, and particularly where the road crosses the waterway on a relatively high embankment, a culvert will usually be cheaper than a bridge.

Culverts . . .

Culverts are usually constructed from:

- - Precast concrete jointed pipes
- - Single or multi-cell reinforced concrete boxes, prefabricated or built in situ
- - corrugated steel pipes made of prefabricated panels

Culverts by District Offices



Total No. of Culverts= 35,026 (as of 2018)

Types of Culverts

1- CONCRETE PIPE CULVERTS

- Cheap and often serves for several years

2- FLEXIBLE STEEL CULVERTS

- They are better suited to unstable conditions than rigid concrete structures.
- They are also easier to transport and faster to construct
- Corrosion is a problem

3- REINFORCED CONCRETE BOX CULVERTS

- Twin or multiple cell box culverts are used where the horizontal opening is more than about 4m.

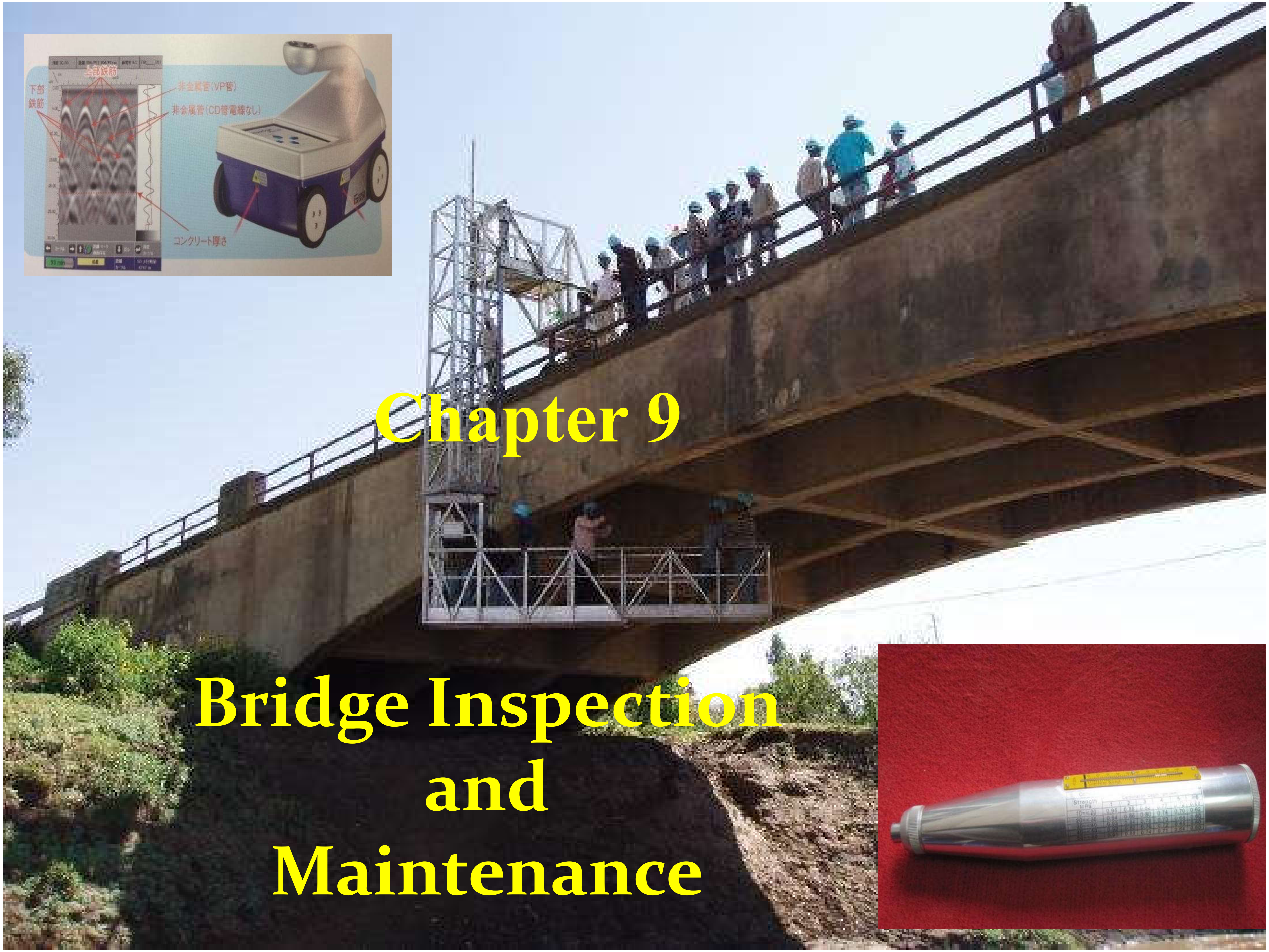
LOW LEVEL WATER CROSSINGS

- In favourable conditions, low level water crossings can provide economical and relatively simple alternatives to conventional bridges. These are of two basic types:
  - - fords and bed-level causeways
  - - vented causeways and submersible bridges

- Both types are appropriate for sites where traffic volumes are low or where a reasonably short detour provides access to an all-weather bridge.
- For most of the year the maximum depth of water over the carriage way should be less than 150mm.

FORDS

- Fords are the simplest form of river crossing at places where the stream is wide, shallow and slow, the approaches gentle, and the surface firm.
- All types of ford may require scour protection on the downstream side



Chapter 9

Bridge Inspection and Maintenance

Bridge Management

- **Bridge Management is**
  - the means by which a bridge stock is cared for from conception to the end of its useful life.
  - a fundamental and crucial activities in order to

    ✓ maintain bridges in good conditions

    ✓ ensure safe bridges and prevent the bridges from damages by conducting inspections and maintain regularly.

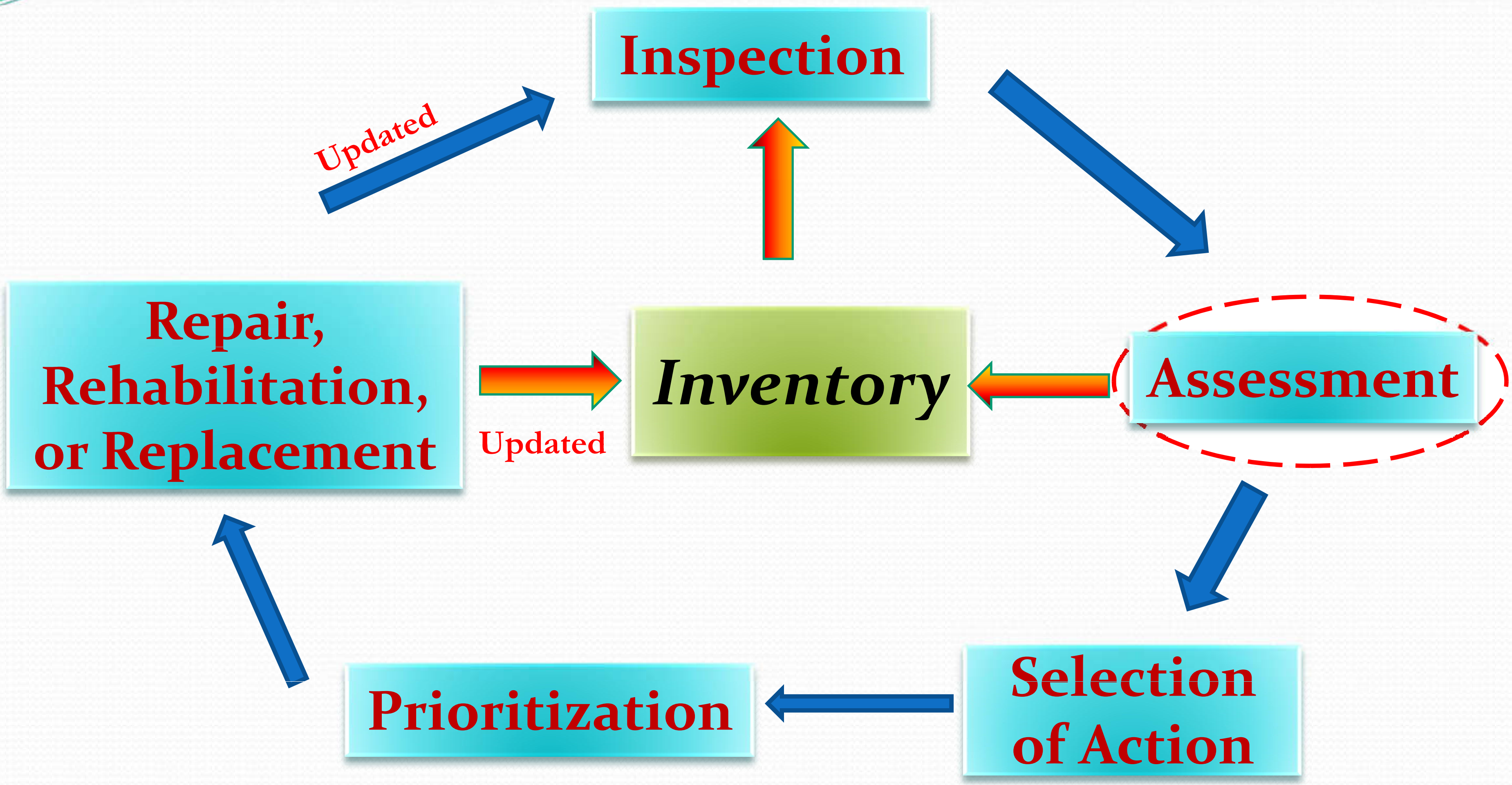
    ✓ assessment/ decision making system/ development

Bridge Management

Bridge Management includes-

- Collection of inventory data
- Inspection (Regular, major ,Emergency)
- Assessment of condition and strength
- Repair works (Repair, strengthening or replacement)
- Prioritizing allocation of funds
- recording and updating of bridge data / bridge data base /
- Safety

Bridge Management



Bridge Management Cycle

Old Bridges

In various state of deterioration

Load capacity reduced

Performance assessment ???

Repaired, strengthened or replaced

PROBLEM

-An issue to maintain a fully functioning transportation.

- Accommodation of modern and existing traffic is in question.

WHY?

Causes

- Overloading
- Aging
- Poor design
- Poor construction

Major defects

- Flexural cracks,
- Shear cracks,
- Concrete peel off,
- Rebar exposure,
- Honey comb & abrasion

Bridges Inspection

- is one of the maintenance management and all the technical activities to maintain the function of bridges to meet the requirement of the structures through its service life.

  - **objectives of inspection**
    - to get the present condition on bridges
    - to prevent damages by removing causes of damage
    - to identify damages in early stage & proper measures are taken before serious damage develop
    - to find progress of damage
    - to try to reduce restriction and protection of safe and smooth traffic

Classification

- **According to ERA bridge inspection manual**

Bridge inspections are classified into **three** types in terms of purposes and frequencies.

• Regular inspection • Major inspection • Emergency inspection	→	• Visual Inspection • Detailed Inspection

Types and frequency of Inspection

- **Regular inspection** – once a year / visual inspection /
- **Major inspection** – once in every 3 years / detail inspection/
- **Emergency inspection** -shall be conducted when natural disasters and severe traffic accidents may be occurred.

Visual Inspection

- **Visual inspection:** as a preliminary step of bridge assessment, involves gathering of data from existing bridge via regular visual control based on various standards. Results of inspection are saved in the records (bridge book, sheets and digital files as well) for further analysis.

Bridge failure due to overloading

Wochite Bridge, which is Steel Panel/ Bailey bridge, is located in Alemgena district located 203.21km from Addis Ababa in Alem ketema - Dogolo road segment.

Cause of damage :-over loading

(Dec 06 /2012)

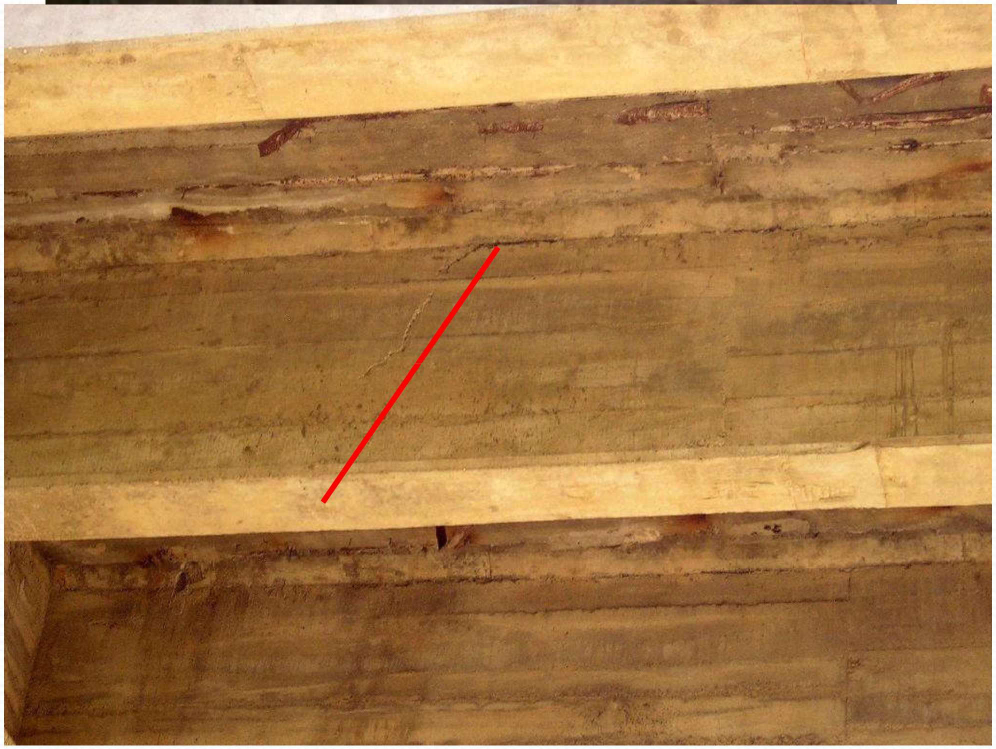
Bridge failure due to overloading _Gibe Bridge

due to aging

- Most of bridges are constructed during Italian’s invasion and serving for more than 7 decades and currently they are not capable & suitable to the current traffic flow and heavy loads due to aging, reduction in load carrying capacity and accidental impact.

Major Bridge Defects

No.	Components	Damage/Defects
1	Concrete	• Flexural crack • Shear crack • Concrete peel off/ delamination • Rebar exposure • Material deterioration • Honey comb • Abrasion • Potholes
2	Steel	• Bent or damaged steel beams, girders, or truss members. • Cracked or spalled concrete members, other than curb and railing. • Broken or weakened chord members of failed truss joints. • Unusual looseness or vibration of truss members. • Loosened or decayed timber deck over an extended area. • Defective bearings on substructure or in deck at expansion joints. • Settled bents or piers. • Lack of paint on steel members, other than curb and railing. • Extensive fire damage. • Excessive noise or vibration from operating machinery. • Lack of lubricant in machinery bearings. • Loose bolts (signs of wear looseness of connections) • Broken timber stringers. • Ineffective supplemental bents • Buckling of components



Maintenance

Maintenance is an actions necessary for retaining or restoring to the specified operable condition to achieve its maximum useful life.

Maintenance applies to:

- Existing concrete deck repairs
- Deck protective systems
- Deck drainage
- Bearings retrofit
- Concrete structure repairs
- Steel girders rehabilitation.

- **Bridge Maintenance is carried out for the following reasons**

- To keep the performance of a bridge to the required level during its service life and ensure safety of users;

- To preserve the planned level of traffic safety and quality during the whole lifetime of the bridge;

- To keep or increase the design load bearing capacity of the bridge or its elements irrespective of the occurrence of any defects or accident

➢ Rehabilitation of Gibe Bridge

- Strengthening the structure by External prestressing through the use of 6 post- tensioned high tensile steel Cables and each consisting of 12 strands (increase load carrying capacity of the bridge)
  - The rehabilitation work carried out by Salini Costruttori S.P.A , Nov 2007)

Gebre-Hara Bridge

Alamata-Mehoni

Strengthening the Girders: using Steel-Plate-Bonded Reinforcement (SPBR)

Crack repaired using Injection method

The End !