

**ADDIS ABABA UNIVERSITY**  
**ADDIS ABABA INSTITUTE OF TECHNOLOGY**  
**SCHOOL OF CIVIL AND ENVIRONMENTAL ENGINEERING**



**ASSESSMENT OF RAILWAY BRIDGE DESIGN AND  
CONSTRUCTION:CASE STUDY OF ADDIS ABABA LIGHT  
RAIL TRANSIT PROJECT**

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**A Thesis in Railway Engineering**

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November,2016

Addis Ababa

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A Thesis

Submitted in Partial Fulfillment of the Requirements for the Degree of Master of Science

The undersigned have examined the thesis entitled ‘**Assessment of Railway Bridge Design and Construction: Case Study of Addis Ababa Light Rail Transit Project**’ presented by **Kassahun Abebe Nadew**, a candidate for the degree of **Master of Science** and hereby certify that it is worthy of acceptance.

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## **UNDERTAKING**

I certify that research work titled “Assessment of Railway Bridge Design and Construction: Case Study of Addis Ababa LRT Project” is my own work. The work has not been presented elsewhere for assessment. Where material has been used from other sources it has been properly acknowledged / referred.

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## **ABSTRACT**

Urban rail transit is an all-encompassing term for various types of local rail systems providing passenger service within and around urban or suburban areas. Many of this urban rail transit system encompass different types of aerial structures/bridges. Apart from similar highway bridges, fulfilling all the design and construction requirements, Railway aerial structures come in many forms, and each has a different level of interaction with the tracks carried.

Addis Ababa light rail project is currently under construction by Chinese contractors (CREC: China railway engineering corporation).which is the first in the country of its kind as far as light railway transit system is concerned.

The research mainly focus at assessing of the design approach of different design standards and improved construction methods in a manner to suitably fit our existing conditions.

Finally, the study adopts the best practice of design and construction for use in our light railway bridges and also hint for further practice.

## **ACKNOWLEDGMENTS**

Foremost, I would like to express my sincere gratitude to my advisor Dr.AsnaKeAdamu for the continuous support of my Master Degree study and research, for his patience,motivation,enthusiasm, and immense knowledge. His guidance helped me in all the time of research and writing of this thesis.

Finally, I must express my very profound gratitude to my parents for providing me with unfailing support and continuousencouragement throughout my years of study and through the process of researching and writing this thesis.

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## 1. Introduction

### 1.1. Background

#### Design and Construction of Light Railway bridges

An elevated guide way or viaduct is a aerial structure that is similar to a bridge. The difference between these two structures has not been clearly established, but normally an elevated guide way is a longer structure that is not used for crossing over an specific obstacle, like a river or a canyon. Instead, it crosses a city, over previously built streets and other structures.

In any case, in engineering terms, bridge, and elevated guide way and the viaduct are the same structure. The structural engineering terms that apply to one also applies to the other. Therefore, when referring to all bridge-like structures including elevated guide-ways, light railway bridge and viaducts, only the word bridge is used. The same terminology has, therefore, been used in this research as well, [1].

A bridge is a structure built to span physical obstacles such as a body of water, valley, or road, for the purpose of providing passage over the obstacle. Bridge is the key element in a Transportation System. It provides essential links in highways and railways at obstacles. To achieve this purpose, bridges must be designed and constructed properly, [2].

There are different types of bridges depending on different criteria. These classifications are based on:

- Material: Steel bridges, concrete bridges, stone/masonry bridges, composite bridges, wooden bridges
- Usage: Pedestrian bridges, highway bridges and railway bridges
- Span: Short span, medium span and long span
- Structural form: slab bridges, girder bridges, truss bridges, arch bridges, suspension bridges and cable stayed bridges.

Selection of bridge type for a specific project must consider different factors. In general, all factors are related to safety, serviceability, economy and aesthetics. These factors are;

- Geometric condition of the site
- Subsurface condition of the site
- Functional requirement
- Aesthetics
- Economy and ease of maintenance
- Construction and erection consideration
- Legal consideration

Since Girder Bridge is sole part of this research, distinctive features of these bridge types will be discussed shortly. Major features of girder bridges are:

- Widely constructed
- Used for short and medium span
- Carry load in shear and flexural bending
- Stability concerns limit the stress and associated economy
- Economical and long lasting solution for vast majority of bridges [3]

### **Light Railway bridges**

In general, Railway bridges like highway bridges provide passage to railway traffic. Railway bridges have distinct difference from Highway Bridge, which include:

1. The ratio of live load to dead load is much higher for Railroad Bridge than for similar sized highway structure. This can lead to serviceability issues such as fatigue and deflection control governing designs rather than strength.
2. The design impact load on railroad bridges is higher than on highway structure
3. Simple span structures are preferred over continuous structure for Railroad Bridge. Many of the factors that make continuous span attractive for highway structure are not as advantageous for railroad use. Continuous spans are also more difficult to replace in emergencies than simple spans

4. Interruptions in service are typically much more critical for railroads than for highway agencies. Therefore, constructability and maintainability without interruption to traffic are crucial for railroad bridges.
5. Since the bridge support the track structure, the combination of track bridge movement should be considered in design and detailing.
6. Seismic performance of highway and railroad bridges can vary significantly. Railroad bridges have performed well during seismic events.
7. Railroad bridges serve a longer service life than highway bridges, [4].

Moreover, light rail bridges are widely utilized in urban light rail transit system with some distinct characteristic from heavy freight railway bridges which will be discussed in the next section.

### **Urban rail transit**

Urban rail transit is an all-encompassing term for various types of local rail systems providing passenger service within and around urban or suburban areas. The American Public Transportation Association (APTA) defines light rail transit as, "an electric railway system characterized by its ability to operate single or multiple car consists (trains) along exclusive rights-of-way at ground level, on aerial structures, in subways or in streets, able to board and discharge passengers at station platforms or at street, track, or car-floor level and normally powered by overhead electrical wires", [5].

Many of this urban rail transit system encompass different types of aerial structures/bridges. Apart from similar highway bridges, fulfilling all the design and construction requirements, Railway aerial structures come in many forms, and each has a different level of interaction with the tracks carried. At one extreme are heavy ballasted track bridges that have very little structural interaction between the rails and the structure. At the other end of the spectrum are concrete deck structures with continuous welded rail (CWR) directly affixed to the deck. This design is very typical of modern rail transit aerial structures. These structures can have significant interaction between the rail, which does not move, and the structure, which must expand and contract with changes in temperature. Somewhere in the middle of the spectrum are

open deck bridges, which are very commonly used for long railroad structures and are often found on older urban rapid transit railways.

The design of bridge structures for light rail transit systems involves choosing a design code, determining light rail vehicle (LRV) forces, confirming track configuration requirements, and applying rail/structure interaction forces. This interaction is affected by such factors as the bearing arrangement at the substructure units, track work terminating on the aerial structure, type of deck construction, and type of rail fasteners.

As of 2010, there is no nationally accepted design and construction code that has been developed specifically for light rail transit aerial structures. In addition to owner-specific and local design codes, designers must choose between the Standard Specifications for Highway Bridges, published by the American Association of State Highway and Transportation Officials (AASHTO), the Manual for Railway Engineering (MRE) issued by the American Railway Engineering and Maintenance of Way Association (AREMA), and, more recently, the AASHTO Load and Resistance Factor Design (LRFD) Bridge Design Specifications. Unfortunately, neither set of AASHTO specifications nor the AREMA manual accurately defines the requirements of an aerial structure to resist light rail transit loads, although the AASHTO specifications are generally more applicable. Research shows that several recent LRT projects' aerial structure design criteria have been established as LRFD based. This is evidence of LRT aerial structures moving toward being designed according to the LRFD methodology, and this apparent trend will likely continue into the future. Proper design and construction approach should be implemented according to the specific project requirement. Otherwise, besides economic, serviceability and aesthetic penalty, catastrophic result might follow, [6].

Addis Ababa light rail project is currently under construction by Chinese contractors (CREC: China railway engineering corporation).this is the first for the country as far as light railway transit system is concerned. However, Ethiopia has long history in developing cross country railway system starting to 1912 E.C, the Addis Ababa-Dire Dawa-Djibouti line. This line is also under construction.

The Addis Ababa light rail system covers the East-West and North-South (16.25km) transport corridors. The East-West corridor (17.26km) passes through the city center from the eastern

peripheries of the city to the west. N-S route has the total length of 2345m of which are elevated road section while the E-W route has the total length 2730m of which also have elevated section.

The design and construction of these bridges are carried by the Chinese construction company (CREC) under Build, Operate and Transfer (BOT) agreement. Therefore, both the design and construction are governed by the Chinese standard. In the mean while, Ethiopia has not yet developed its own design and construction standard for railway bridges. This research along with other similar studies tries to assess improved design and construction approach for urban LRT bridges which will be constructed in the country.

The assessment mainly focus on the design approach of different design standards, particularly the widely practiced American railway structural design standard (AREMA) developed by the American railway engineering and maintenance association in joint committee with the American association of state highway and transportation. The other one is the Chinese design standard for railway structure and ERA Bridge Design Manual. Along with these standards, other local code provisions are included in the assessment.

In parallel with assessing design approaches, practically improved construction methods suitable for our existing conditions are also studied and finally adopted practices both on the design and construction approach and future study area which are not included in the research are presented briefly. Further, this study performs critical assessment and evaluation on the design and construction of the current Addis Ababa light rail transit project as case study.

## **1.2. Objectives of the study**

In the past few years, bridge engineering is showing much improvement both in design and construction science. This development helps in improved performance of bridges, efficient cost utilization, easy construction, maintenance and durable structure. The invention of special construction equipments, latest analysis computer program and new innovative design approach is behind this development. Ethiopian is building huge infrastructure around the country. Addis Ababa Light Rail Transit project one of the kinds. The Addis Ababa LRT project consists major bridge passages over congested traffic road ways. Design and construction of bridges in the

country has different practice. Precast prestressed bridges in particular railway structure technology are practiced by oversea companies. Knowledge transfer is also limited and therefore new approach should be exhaustively explored. In general, this research tries to assess this experiences and try to adapt to suit to the current situation. Hence; the objective of this research;

1. Assess the latest design standers for the design and construction of LRT bridge structures which includes,
  - American Railway Engineering and Maintenance Association (AREMA), is found to be well tested and refined experience, though mainly in heavy railway bridges. The AREMA manual is more or less compatible with American association of state highway and transportation (AASHTO) bridge design specification in which our design standard of bridge is adopted. This similarity helps to easily modify and adopt latest railway bridge design experiences
  - Chinese design standard referred as “Code for Design on Reinforced and Prestressed Concrete Structure of Railway Bridge and Culvert” .This standard is preferred for the reason that the code has already been employed in our country and critical assessment is required to evaluate the feasibility of the standard to our country. Along with these standards, local codes which are similar in project scale also discussed.
  - Other includes Germany code and local codes for the state of Minnesota, Massachusetts and Phoenix are also referred when it is found necessary.
2. Discuss the current bridge construction approaches, specifically urban LRT bridge construction methodologies and their feasibility to our demand.
3. Assess the design and construction approach of the Addis Ababa light rail transit project in particular the light transit bridge aspect.
4. Recommend the assessment and evaluation findings which is expected to be improved design and construction approach of LRT bridge structures.

### 1.3. Organization

The section of this study is organized as briefed below.

Section 1 address introduction where the background of the study, objective & scope are briefed. Section 2, reveals historical background of light rail transit, general design considerations of bridges, review of AREMA bridge design manual, review of bridge-rail interaction, review of major types of railway track system, review of design of box girder railway bridges and review of basics of prestressed concrete structure.

Section 3 discuss on review of bridge construction methods which includes review of segmental bridge construction, assessment of balanced cantilever, progressive placing, incremental launching and Span-by-Span method.

Section 4 focused on the Addis Ababa LRT Rail Transit discussing overview of proposed design practice, current design practice and proposed guideline for design and construction of light Rail transit project

This section finally ends with conclusion and recommendation of the research findings.

## 2. Literature Survey

### 2.1. Historical background of light rail transit

Light rail transit evolved from streetcar technology. Electric streetcars dominated urban transit in just about every significant American city up through World War II. But once the war was over, "old-fashioned" trolley lines were converted to bus operation in droves, all in the name of "modernization." By 1965, only a handful of legacy streetcar systems still survived. Since the appearance of light rail transit system, construction of the infrastructure became the nightmare of the builders due to conceive structures that can be built in or over crowded areas of cities, with a minimal impact on existing conditions. Constructions of bridges are alternatively the best solution.

#### 2.1.1. The light Rail Revolution-Britain

The Tyne side PTA published its public transport plan in 1973 setting out proposals for the creation of a light rail transit system to take over the alignment of run-down local rail service (42km) and link them up by new infrastructure (13.5km) to create network of electrified suburban lines which would form the core of an integrated passenger transport system, including bus feeders to purpose-built interchanges. The initial system was opened in stages from 1980 to 1984; a 3.6km extension to Newcastle airport was opened in 1991, and plans are now advancing for an extension to Sunderland. Britain's first light rail system has been successful in assisting the regeneration of Tyn eside and carries over 40 million passengers/year.

Greater Manchester set up a Rail Study Group in 1982 to develop a long-term strategy for the local rail network, and this proposed that the Altrincham (12.3km) and Bury (16km) line, which were in need of investment, be convert to light rail and linked through the city centre on 2.7km of new street tramway to reach Piccadilly Station. The range of systems, from segregated and automated operation on formed rail alignments to conventional street tramway indicate the flexibility of the concept.

### **2.1.2. The light Rail Revolution Mainland Europe**

Light rail was first created in mainland Europe, as street tramways were upgraded with new rolling stock and segregated alignment. The concept owes much to the planning which took place in (Gothenburg) in Sweden, where over a period of 15 years an ordinary

## 2.2. Review of Design Approach

### 2.2.1. General Design Considerations of Bridges

#### Aim and purpose of bridge design

Bridge design and construction for any kind of bridge, including precast prestressed segmental concrete bridges, should always have the same fundamental objectives. These fundamental are safety, serviceability, economy and elegance (Aesthetics). A design can only be successful when all four goals have been achieved. The relative importance of each objective is going to be defined by the consequences of not achieving them.

#### a. Safety

A bridge is safe when it is known that it is not going to collapse under the applied loads. The achievement of safety for a bridge structure is mostly believed to be scientific process. A bridge designer follows design codes, and a bridge is designed to resist ultimate loads. If all the steps in the design and construction process are followed accordingly, and if the structure is inspected properly and given the necessary maintenance, the bridge is said to be safe, and the structure is not supposed to collapse as long as the values of the ultimate loads are not exceeded.

The fact is that the value of both, the loads to which the structure is subjected and the ultimate load which the structure can resist, are not known exactly. Both are functions of many different factors. Among other things, the loading will depend on the actual traffic patterns that use the bridge, the wind loads and earthquake loads to which the structure is subjected, the settlements taking place in the foundations, and the changes in temperature that occur in the structure. Likewise, the ultimate load the structure resists will depend, among other things, on the actual strength of materials, the workmanship given in the construction, the deterioration the structure suffers, and the inspection and maintenance given to the structure.

Design codes are not infallible. Bridges that have followed design codes have been known to collapse. Sometimes this is because the applied loads were larger than expected, even with the use of factors of safety. At other times it was because the structure was not constructed

according to its design, or because it was not given the proper inspection and maintenance. Design codes sometimes improperly simulate the structural action of bridges.

The factors of safety and equations found in design codes are trying to simulate what will happen to the structure in reality. Sometimes the simulations are not as close to reality as they need to be. Therefore, codes are revised and changed, trying to get closer to reality and helping in the design of safer structures.

### **b. Serviceability**

Serviceability is an issue very similar to safety. It is sometimes believed to be achieved completely through scientific procedures by following design codes.

A bridge is serviceable when it fulfills the need for which it was designed. The most important aspects of serviceability are appearance, function, and durability. Appearance refers to giving the impression that the structure is secure. Cracks and large deflections can give the impression that a structure will fail if used, although it may not. Function refers to how the bridge works when used.

The bridge should be safe for the users (and for any traffic that passes under it) during both normal and emergency conditions. Durability refers to remaining in service or remaining fit for use for a long period of time.

There are aspects of serviceability that can be quantified such as deflection, vibration, cracking, run-off of rain water on the deck, and traffic volumes. These aspects can be fulfilled by following design codes and following proper construction procedures.

### **c. Economy**

When a bridge is designed a number of decisions have to be made. These decisions will impact the cost of construction and other costs in the long run. They include the selection of materials, the type of construction to be used, how will the workers get organized, details of the structure, and the equipment to be used. Also, an appropriate strategy has to be established for construction, maintenance, rehabilitation, and ultimate disposal.

#### **d. Aesthetics**

The issue of achieving aesthetics in bridges can, at first, appear to be very complicated, because beauty is believed to be very subjective. In reality, by checking the literature written by experts in aesthetics of bridges, it can be found that there is agreement on most of the major issues of elegance in bridges.

At the same time, the bridge structure can be perceived as an individual entity, therefore its beauty will also depend on the bridge's structural form. Commonly believed ways of achieving an elegant bridge include: using efficiently the materials for a structure that is slender and transparent as opposed to heavy and massive; integrating harmonically all the structures components into one coherent, organized structure; making the structure symmetrical; designing for simplicity without unnecessary ornamentation; and, incorporating topographical features of the surroundings, [6].

##### **2.2.1.1. Design philosophy**

###### **2.2.1.1.1. The Nature of Design**

Design can be described as the process of conceptually creating something that does not yet exist. The nature of design is such that there is normally more than one feasible design (a design that will "do the job"). A design is defined by a set of design variables that can be used in analyzing the design. A design is considered to be feasible if it satisfies the performance criteria (or constraints) defined for the problem. Out of the available feasible designs, some are "better" than others. Generally the goal of the designer is to find optimum design. The optimum design is generally considered to be the one design that best satisfies the criteria for the project. Typically there is some kind of objective function that can be computed from the variables that define a design. The value of the objective function is used to compare feasible designs in order to determine the "best" or "optimum" design.

Another significant feature of design is that it is an iterative process. When the design process starts, there are a lot of unknowns. Consequently, the designer must make assumptions about various components of the design. As the design progresses, many of the assumptions can be

replaced by known facts when decisions are made about previously unknown components. As the assumptions are refined and replaced, the design is reanalyzed repeatedly.

#### **2.2.1.1.2. Loads and Forces consideration**

##### **General**

Structures or parts of structures subjected to transit loadings shall be designed considering loads and forces defined herein, and including loads due to system wide elements such as electrification, signalization and communication equipment where and when applicable. Other loads and forces need to be considered when there is a probability that the structure maybe subjected to such forces. Modification to the design capacity of the structure under such load should be approved by the Authority based on the probability of occurrence of such loads. The loading criteria and all other loads to which the structures are designed shall be shown on the structural drawings.

##### **a. Dead Load (DL)**

Dead load shall consist of the actual weight of all components of the structure and all elements that are permanently supported by the structure. This load includes but is not limited to the weight of track, ballast, ties, utilities, electrification including third rail and fasteners, signalization and communication equipments, and all other permanent fixtures on the structure.

Consideration shall be given to any system or facility such as piping, conduits, rigid stainless steel conduits, manholes, transformer vaults, tiebreaker houses, cables, pulling irons, catenary, and other services that will apply a load or force or cause a force to be transmitted to the design structure.

##### **b. Vehicle Live Load (LL)**

Vehicle Live Load shall consist of any non-permanent forces induced on the structure by transit vehicles, work train or other moving objects, construction loads, and loads due to maintenance operations.

Car Combinations:

Any combination revenue car lengths and loadings which produces the most critical loading conditions for bending, axial, shear and torsional stresses, as well as deflections, fatigue, and stability shall be Used for the design.

Extra-Ordinary-Car consist should be considered as a special case where applicable. This case represents a disabled revenue train being pushed by another revenue train. The combined length of both trains will be twice the typical length of train consist.

### **c. Longitudinal Breaking and Traction Force (LF)**

Longitudinal forces generated by the acceleration and deceleration of revenue/work trains. Consideration are to be given to combinations of acceleration and deceleration forces where more than one track occurs. For double track structures, three longitudinal loading cases shall be considered:

- Single track loaded; longitudinal force acting in anyone direction.
- Both tracks loaded; one train accelerating while the other train is decelerating, or both trains traveling in the same direction. Both longitudinal forces acting in the same direction.
- Both tracks loaded; both trains accelerating and decelerating. Longitudinal forces acting in opposite directions.

### **d. Derailment Load (DR)**

Derailment load is defined as a vertical load that is produced by the train live loading placed with its longitudinal axis parallel to the track. A separate check for slab punching shear associated with the derailment shall be performed. The punching shear check shall consist of an equivalent static wheel load with applicable impact factor distributed over an area of one square foot. When checking any component of the superstructure or substructure that supports two or more tracks, only one train on one track shall be considered to have derailed, with the other track(s) each loaded with a stationary train. No impact allowance shall be considered for the stationary train on the other track(s).

**e. Wind Load:****Wind Load on Structure (W)**

The aerial structures shall be designed to withstand wind loads of uniform pressure acting on the superstructure, the substructure and the live loading as specified herein. Wind loading on the various system elements, such as signalization and communication equipments, and all other fixtures on the structure shall be considered in the design of both superstructure and substructure. The magnitude and location of wind loads on system elements shall be determined by the specific code under consideration.

**Wind load on Live Load (WL)**

Design shall include a simultaneously applied transverse horizontal wind load and a longitudinal horizontal wind load for the entire length of track supported by the element being designed. The transverse loads shall be applied to the train as concentrated loads at the axle locations. Longitudinal force shall be applied to the rails and superstructure as a uniformly distributed load over the length of the train in a horizontal plane at the top of rail. The horizontal force component transmitted to the rails and superstructure by an axle shall be concentrated at the rail having direct wheel-flange-to-rail-head contact.

**f. Thermal Force (T)**

Provisions shall be made for deformations and stresses resulting from temperature variations. In addition, thermal gradient within the section of the structure shall be taken into consideration.

**g. Rail/Structure Interaction Force**

The design of the structural elements shall include the forces resulting from the interaction between rail and structure in non-ballasted aerial structures. No interaction loads need to be considered for ballasted structures.

The interaction forces are typically due to thermal variations between the rail and the deck. The rail fasteners would mobilize longitudinal and radial forces (on curved rail) resisting the differential movement between the deck and the rail. The design shall include

the loads mobilized in the structure that provides longitudinal and lateral restraint to the rail (when the structure is located on a curve). The analysis shall be based on the maximum differential temperature in the rails. Thermal rail/structure interaction investigation shall include the entire structure (from abutment to abutment) and a minimum 200 feet of tracks beyond each abutment.

#### **h. Rail Transverse Force (RT)**

The transverse force must be resolved into components parallel and perpendicular to the pier at each rail fastener and summed.

#### **i. Rail Longitudinal Force (RL)**

It is recommended that the direct fixation fasteners with satisfactory in-service performance history be installed. In lieu of in-service performance history, new direct fixation fasteners that have satisfactorily passed extensive laboratory testing may be used.

#### **j. Rail Break Force (RB)**

For direct fixation track, provision shall be made for longitudinal forces due to a rail break. Forces from a single broken rail at any one time shall be applied to the structure. Longitudinal break force shall be based on the maximum temperature differential in the rail. The mobilized forces in the structure are equal to the maximum restraint force in fasteners of the broken rail until the thermal force is equalized. The structure shall be designed to include horizontal forces at the fixed bearing due to the summation of each rail fastener's longitudinal restraint. The structure shall also be designed to include a twisting moment in a horizontal plane at the height of the low rail due to opposing directions of forces in the broken and unbroken rails.

#### **k. Shrinkage and Creep Force (S)**

Stresses and movements resulting from concrete shrinkage and creep shall be considered in the design and included in the specified load combinations. The shrinkage coefficient shall be assumed to be 0.0002 inches per inch for both prestressed and reinforced normal weight

concrete. Shrinkage coefficient for lightweight concrete shall be determined for the type of lightweight aggregate used.

#### **l. Collision Load**

This force is to be applied on the support element at an angle of  $x$  degrees (specified in the concerned design code) from the direction of road traffic. This condition occurs with the dead load of the structure but need not to be applied concurrently with other loadings.

#### **m. Earthquake Forces (EQ)**

The seismic design of structures supporting transit vehicles shall be directed by the Authority:

#### **n. Transit Bridge Railings**

Transit bridge railings and their connections to supporting structures outside the areas shall be designed,[8].

## 2.2.2. AREMA Bridge Design Manual

### Design Methods

The design of reinforced concrete members shall be made either with reference to service loads and allowable service load stresses as provided in the Service Load Design Section or, alternately, with reference to load factors and strength as provided in the Load Factor Design section. The design method to be used, SERVICE LOAD DESIGN or LOAD FACTOR DESIGN, shall be as directed by the Engineer.

### Design Loads

#### a. Dead load:

The dead load shall consist of the estimated weight of the structural member, plus that of the track, ballast, fill, and other portions of the structure supported thereby.

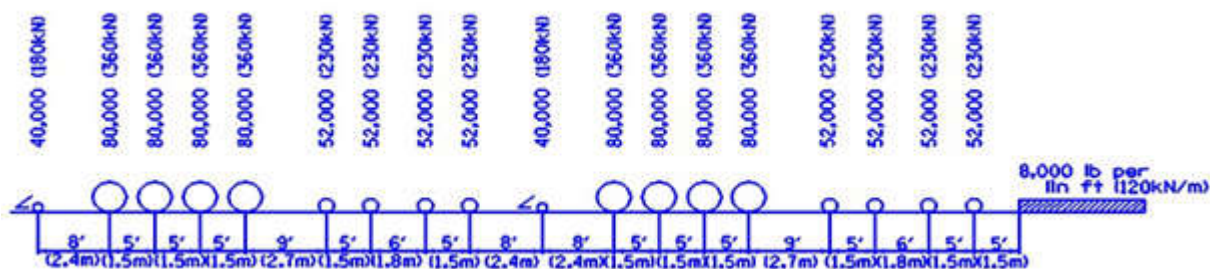
#### b. Live load:

(1) The recommended live load for each track of main line structure is Cooper E 80 (EM 360) loading with axle loads and axle spacing as shown in Figure 8-2-1. On branch lines and in other locations where the loading is limited to the use of light equipment, or cars only, the live load may be reduced, as directed by the engineer. For structures wherein the material in the primary load-carrying members is not concrete, the E loading used for the concrete design shall be that used for the primary members.

(2) The axle loads on structures may be assumed as uniformly distributed longitudinally over a length of 3 feet (900 mm), plus the depth of ballast under the tie, plus twice the effective depth of slab, limited, however, by the axle spacing.

(3) Live load from single track acting on the top surface of a structure with ballasted deck or under fills shall be assumed to have uniform lateral distribution over a width equal to the length of track tie plus the depth of ballast and fill below the bottom of tie, unless limited by the extent of the structure.

(4) The lateral distribution of live load from multiple tracks shall be as specified for single tracks and further limited so as not to exceed the distance between centers of adjacent tracks.



**Figure 2.1:** Copper E 80(EM 360) Axle Load Diagram

(5) The lateral distribution of the live load for structures under deep fills carrying multiple tracks, shall be assumed as uniform between centers of outside tracks, and the loads beyond these points shall be distributed as specified for single track. Widely separated tracks shall not be included in the multiple track groups.

(6) In calculating the maximum live loads on a structural member due to simultaneous loading on two or more tracks, the following proportions of the specified live load shall be used:

- For two tracks – full live load,
- For three tracks – full live load on two tracks and one-half on the other track,

For four tracks – full live load on two tracks, one-half on one track, and one-fourth on the remaining track.

(7) The tracks selected for full live load in accordance with the listed limitations shall be those tracks which will produce the most critical design condition on the member under consideration.

### c. Impact load:

(1) Impact forces, applied at the top of rail, shall be added to the axle loads specified. For rolling equipment without hammer blow (diesels, electric locomotives, tenders alone, etc.), the impact shall be equal to the following percentages of the live load:

For  $L \leq 4$  meters  $I = 60$

For 4 meters  $< L \leq 39$  meters  $I = 125/\sqrt{L}$

For  $L > 39$  meters  $I = 20$  Where  $L$  is the span length in meter.

This formula is intended for ballasted-deck spans and substructure elements as required.

(2) For continuous structures, the impact value calculated for the shortest span shall be used throughout.

(3) Impact may be omitted in the design for massive substructure elements which are not rigidly connected to the superstructure.

(4) For steam locomotives with hammer blow, the impact calculated according to Article 2.2.3d (1) shall be increased by 20%.

#### **d. Wind load:**

##### **Wind load on structure**

The base wind load acting on the structure is assumed to be 45 lb per square foot (2160 Pa) on the vertical projection of the structure applied at the center of gravity of the vertical projection in any horizontal direction. A base wind velocity of 100 miles per hour (160 km/h) was used to determine the base wind load. If an increase in the design wind velocity is made, the design wind velocity and design wind load shall be shown on the plans.

For Group II and Group V loadings, when a design wind velocity greater than 100 miles per hour (160 km/h) is advisable the base wind load may be increased by the ratio of the square of the design wind velocity to the square of the base wind velocity. This increase shall not apply to Group III and Group VI Loadings.

##### **Wind load on live load**

A wind load of 300 lb per linear foot (4.4 kN/m) on the train shall be applied 8 feet (2450 mm) above the top of rail in a horizontal direction perpendicular to the centerline of the track.

**e. Longitudinal force**

(1) The longitudinal force for E-80 (EM 360) loading shall be taken as the larger of:

- Force due to braking, as prescribed by the following equation, acting 8 feet (2450 mm) above top of rail.

$$(\text{Longitudinal braking force (kN)} = 200 + 17.5L)$$

Where L is the length in feet (meters) of the portion of the bridge under consideration

- Force due to traction, as prescribed by the following equation, acting 3 feet (900 mm) above top of rail.

$$(\text{Longitudinal traction force (kN)} = 200\sqrt{L})$$

Where L is the length in feet (meters) of the portion of the bridge under consideration

For design loads other than E-80 (EM 360), these forces shall be scaled proportionally. The points of force application shall not be changed.

(2) The effective longitudinal force shall be distributed to the various components of the supporting structure, taking into account their relative stiffness. The resistance of the backfill behind the abutments shall be utilized where applicable. The mechanisms (rail, bearings, load transfer devices, etc.) available to transfer the force to the various components shall also be considered.

(3) The longitudinal deflection of the superstructure due to longitudinal force computed in (1) above shall not exceed 1 inch (25 mm) for E-80 (EM 360) loading. For design loads other than E-80 (EM 360), the maximum allowable longitudinal deflection shall be scaled proportionally. In no case, however, shall the longitudinal deflection exceed 1-1/2 inches (38 mm).

Longitudinal force due to friction or shear resistance at expansion bearings:

Provisions shall be made to accommodate forces due to friction or shear resistance due to expansion bearings

**f. Other forces (Rib Shortening, Shrinkage, Temperature and/or Settlement of Supports)**

- (1) The structure shall be designed to resist the forces caused by rib shortening, shrinkage, temperature rise and/or drop and the anticipated settlement of supports.

The longitudinal force is prepared for C-E-80 live load. Hence give unreasonable value for light rail vehicle; although there is a recommendation to scale proportionally the live load other than C-E-80 but does not specify the ratio or any guideline.

**g. Earthquake load**

The railroad industry is vitally interested in maintaining reliability in its infrastructure to assure safety for its employees, passengers, customers' goods and the public at large. These guidelines have been developed specifically for Railroad facilities to help reduce damage from earthquakes.

This Provision provides a framework of considerations and methodologies for seismic design of new bridges, roadbed and other railroad facilities. This also addresses retrofit and post-seismic event response and inspection considerations.

Railroad bridges historically have performed well in seismic events with little or no damage. Contributing to this ability are several factors, unique to railroad bridges, which are consistent throughout North America.

First, bridges are traversed by track structure that functions as a restraint against longitudinal and lateral movement during earthquakes.

Second, configurations of railroad bridges typically differ from other types of bridges.

Third, the controlled operating environment permits different seismic performance requirements for railroad bridges compared to highway bridges.

## General Requirement

Structures shall be designed to satisfy the specified performance criteria. The main objectives of the required performance criteria are to ensure the safety of trains and to minimize the costs of damage and loss of use caused by potential earthquakes.

In order to provide a framework for evaluating seismic effects on railroad structures, a three-level ground motion and performance criteria approach consistent with the **railroad post-seismic event response procedures** is employed. The ground motion levels, the structure performance requirements and the railroad response levels are as shown in Table 2.1

Railroad Response Level	Ground Motion Level	Performance Criteria Limit State
I	1	Serviceability
II	2	Ultimate
III	3	Survivability

**Table 2.1:** Seismic Performance Criteria

## Ground Motion Levels

The ground motion levels reflect the seismic hazard at the site. They are defined in terms of peak ground acceleration levels associated with a given average return period. The average return period for each ground motion level may be determined based on seismic risk and structure importance classification, using the range of average return periods shown in Table 2.2

Ground Motion Level	Frequency	Average Return Period (Yrs)
1	Occasional	50-100
2	Rare	200-500
3	Very Rare	1000-2400

**Table 2.2:** Ground Motion Levels

Level 1 Ground Motion represents an occasional event with a reasonable probability of being exceeded during the life of the structure.

Level 2 Ground Motion represents a rare event with a low probability of being exceeded during the life of the structure.

Level 3 Ground Motion represents a very rare or maximum credible event with a very low probability of being exceeded during the life of the structure.

In general the following requirement shall be considered:

- Risk Factors
- Structure Importance Classification
- Immediate Safety

In design of new bridges the following should be considered.

Bridges with span not exceeding 150m in length are considered. Movable bridges, arch type bridges and bridges with span exceeding 150m in length may require additional analysis and design consideration.

- The conceptual phase should be followed by analysis for level 1 Ground Motion to size the various structure members
- Appropriate detailing provisions should be incorporated to allow the bridges to respond well during Level 2 and 3 Ground Motion.
- Structures located in areas of low ground motion levels need not meet the conceptual design requirement and detailing provision provided they are capable of withstanding the full Level 3 Ground Motion loadings within the elastic range.

The preferred configuration of bridges includes;

- Straight bridges alignment
- Normal piers
- Uniform pier stiffness
- Uniform span stiffness
- Uniform span mass

Bridges required special consideration are;

- Curved bridges alignments
- Skewed pier
- Varying pier stiffness
- Varying span stiffness
- Varying span mass

Superstructure:

The preferred superstructure characteristic consists of simple span, short span, light span and those with no hinges. Special consideration of preferred configuration includes continuous span, long span, heavy span and intermediate hinges respectively

### **Analysis Procedure**

#### **Serviceability Limit State**

Methods based on elastic analysis shall be used to determine stresses and deformations for the serviceability limit state. The methods recommended include:

- Equivalent Lateral Force Procedure that is applicable to regular bridges and
- Modal Analysis Procedure for multi-span irregular bridges.

The selection of the analysis procedure for the serviceability limit state shall be based on the bridge configuration as shown below.

- Single span-No analysis required
- Two span-ELF or MA procedure
- Multi -span regular-ELF or MA procedure
- Multi-span irregular-MA procedure

Notes:

1. ELF denotes Equivalent Lateral Force Procedure,
2. MA denotes Modal Analysis Procedure.

3. Irregular bridges are those structures with significantly irregular configuration or support stiffness.

### Load Combination and Response Limit

- a. The loads shall be combined in accordance with the formulas in Table 2.3 based on the structure material. These combinations shall be used for seismic loads.

Material	Design Method	Combination
Steel	Allowable Stress Design	D + E + B + EQ
Concrete	Load Factor Design	D + E + B + PS + EQ

**Table 2.3:** Load combination

D= Dead Load

E= Earth Pressure

B= Buoyancy

PS= Secondary Forces from Prestressing

EQ= Earthquake (Seismic)

#### NOTE:

(1) Effects of other loads, such as stream flow pressure, live load and friction shall be included if they have a significant likelihood of acting concurrently with earthquake loads.

(2) Buoyancy loads should be based on the water level that has a significant likelihood of occurring concurrently with earthquake loads and produces the most conservative load combination response limits given in Table 5.3.7 shall be satisfied for each structure material.

#### Detailing Provision:

Appropriate detailing provisions shall be incorporated into the structure to meet the performance requirements for the Level 2 and 3 Ground Motion.

## **Continuity Provisions**

The structure shall be designed with an uninterrupted load path to transfer lateral forces from the superstructure to the ground.

## **Superstructure**

The superstructure shall be designed to carry the lateral forces to the bearings or shear connectors. The lateral forces from the span may be carried to the end supports by the following load paths:

- a. Lateral bracing system.
- b. Lateral bending of the girders, including torsional effects as applicable.
- c. Diaphragm action of concrete decks or steel ballast pans provided that the deck is adequately connected to the girders. End cross frames or diaphragms shall be designed to carry the lateral forces to the bearings or shear connectors.

## **Bearings**

The bearings shall be designed to transfer the lateral forces to the substructure. Bearings may be supplemented by shear connectors to help transfer the lateral forces provided that the movement required to engage the shear connectors does not cause failure of the bearing device.

## **Ductility Provisions**

The ductility provisions contained herein shall be incorporated into the structure design.

## **Longitudinal Reinforcing Confinement**

Longitudinal reinforcing in concrete columns, pier walls and piles shall be adequately confined to allow the member to respond in the post-yield range. This requirement may be met by the following provisions.

### **Splices in Reinforcing**

Lap splices are not allowed in a main load carrying member within a distance “d” (effective depth) of any area designed to respond in the post-yield range.

### **Redundancy Provisions**

The redundancy provisions listed below are suggested to increase survivability during the higher level ground motion events.

### **Bearing Seats**

Bearing seats should be proportioned to accommodate the maximum relative movements caused by earthquakes. This requirement may be met by the following provision: Bearing seats supporting the ends of girders which are allowed to move relative to the seat during an earthquake shall be designed to provide a minimum support width, N, measured normal to the face of the abutment or pier, not less than that specified below:

$$N = (12 + 0.03L + 0.12H)(1 + 0.000125S^2) \text{ inches} \quad \{N = (305 + 2.5L + 10H)(1 + 0.000125S^2) \text{ mm}\}$$

L = length (ft or m) of the bridge deck to the adjacent movement joint, or to the end of the deck.

S = angle of skew (degrees) measured from a line normal to the span.

H = At abutments, H is the average height (ft or m) of piers supporting the bridge deck to the next movement joint, or H = 0 for single span bridges. At piers, H is the pier height (ft or m).

### **Shear Connectors**

Shear connectors may be provided to resist the maximum seismic loads. The shear connectors should be positioned so that they are engaged prior to failure of the bearing device.

### **Span Ties**

Span ties may be used to reduce the likelihood of unseating during the higher level ground motion events. The spans may be tied together by alternate means through the bent caps such as by anchor bolts, shear rods or common bearing plates provided the load path is adequately

verified. The span ties shall be designed to allow for the effects of thermal movement of the span.

### **Continuous Welded Rail**

Continuous welded rail (CWR) may be evaluated as a redundant load path for seismic loads or to increase bridge damping provided the following requirements are satisfied:

- a. No expansion joints are allowed in the CWR over the bridge length and at least 200 feet (60 meters) onto the embankments.
- b. CWR shall be adequately anchored to the ties over the bridge length and at least 200 feet (60 meters) onto the embankments.

### **Loading Combinations**

AREMA provides load combination for service load design, load factor and construction load.

Recommendations for the Design of Segmental Bridges:

AREMA recommendation for design of segmental bridge is similar to AASHTO LRFD design specification. [11]

### 2.2.3. Review of Bridge-Track interaction

#### Bridge-Rail Interaction

With widespread use of Continuous Welded Rail (CWR), the designer of an aerial structure must be aware of trackwork design and installation procedures, as well as vehicle performance and ride comfort issues. Trackwork design and installation procedures are especially critical in establishing the magnitude of the interaction forces between the rail and aerial structure. As the temperature changes, the superstructure (including both the deck and the supporting girders) expands or contracts. Depending on the type of trackform, the interaction of the track structure and the bridge structure will exhibit different behavior. On an “open” trackform (ballasted, open deck, or direct fixation), rails are effectively stationary because of both their continuity throughout the length of the bridge and their being anchored off the bridge. In the case of ballasted track, the relative movement between the superstructure and the combined ties and rails is accommodated by slight movements of the cross ties within the ballast. Effectively, the ballast beneath the cross ties is a shear plane between the track and the bridge. In a direct fixation bridge, the movement of the superstructure relative to the rails as the temperature changes imposes deformation on the fastening system that attaches the rails to the bridge deck. Open deck structures are more indeterminate, and some relative movement can be expected between the rails and the rail fastenings and between the bridge ties and the girders, [9].

#### Continuous Welded Rail

The majority of the early elevated rail transit systems used track-work composed of jointed rail supported on simple-span guide-way structures. Alternatives have been developed for modern rail transit track-work on aerial structures. Rather than the classical jointed rail with bolted connections every 39 feet [12 meters], the track-work is normally constructed with continuous welded rail. With either rail configuration, the rails can be fastened directly to the aerial structure’s deck, installed on ties and ballast, or installed on ties without ballast.

The bolted connections used with jointed rail allow sufficient longitudinal expansion and contraction to reduce the accumulation of thermal stresses along the rails. But there are some disadvantages; bolted joints.

- Generate noise and vibration
- Are troublesome to maintain
- Contribute to derailments if not maintained
- Cause rail fatigue in the proximity of the rail joints
- Cause wear of the rolling stock
- Reduce ride quality
- Increase the dynamic impact forces applied to the aerial structure
- Are points of high electrical resistance for traction power return currents

CWR has been the most common track configuration for rail transit systems for several decades. This is mainly due to its ability to overcome many of the disadvantages of jointed rail. Specifically, CWR

- Minimizes noise and vibration
- Reduces track maintenance
- Improves track safety
- Eliminates the joints that cause rail fatigue
- Limits wear of the rolling stock
- Provides a smooth, quiet ride
- Limits the dynamic impact forces applied to the aerial structure
- Provides a consistent path for traction power return currents.

The use of CWR, combined with direct fixation of the rails to the supporting structure, is an improvement in the support and geometric stability of the track-work. As a result, rider comfort and safety are enhanced, and track maintenance requirements are decreased. The use of CWR requires designers of track-work and aerial structures to consider issues that typically do not arise when using jointed rail, such as the following:

- Providing sufficient rail restraint to prevent horizontal or vertical buckling of the rails
- Providing anchorage of the CWR to prevent excessive rail gaps from forming if the rail breaks at low temperature
- Determining the effect a rail break could have on an aerial structure

- Calculating the thermal forces applied to the aerial structure, the rail, and the fasteners as the aerial structure expands and contracts and the CWR remains in a fixed position
- Providing a connection between the CWR and aerial structure (direct fixation fasteners) that is sufficiently elastic to permit the structure to expand and contract without overstressing the fasteners

An important element in the design of track-work using CWR is the consideration of rail breaks. Rail breaks often occur at structural expansion joints in the aerial structure and must be accommodated without catastrophic effects such as derailment of the vehicle. Depending on the length of the aerial structure, the CWR has to be sufficiently restrained on the aerial structure to limit the length of the gap if the rail does break. CWR is a standard now employed in the transit industry. Therefore, transit system designers must understand how it interacts with aerial structures as the temperature changes in order to provide a safe track and structure.

Expansion (sliding) rail joints are used in certain circumstances to reduce the interactive forces between the CWR and the structure. These include locations where special track-work is installed on the aerial structure and where the aerial structure includes very long spans and or spans of extremely sharp curvature. Rails can be attached to the structure in a variety of ways. The most common mechanism is the use of direct fixation fasteners with elastic spring clips. High-restraint rail clips have also been used in the vicinity of substructure units (piers and abutments) with fixed bearings, as well as adjacent to special track-work. Also, zero-longitudinal-restraint fasteners can be installed to minimize the interaction forces between CWR and the aerial structure. A common configuration will employ high-restraint rail fastenings in a track zone centered above a “fixed-fixed” bearing location while low- or zero-restraint fastenings are used at intermediate zones straddling “expansion-expansion” bearings.

The decision concerning which type of deck construction to use with CWR has profound implications for construction cost. Based on the difference in cost of aerial structures with and without CWR, and the resultant thermal effects considered in the structural design, the most conservative design using CWR could increase structure costs by approximately 20%.

However, there are many variables to consider when choosing the type of deck to use on any particular transit structure.

## Force Distribution between Rails and Superstructure

The thermal action in a direct fixation bridge exerts additional interactive axial forces and deformations on the rails and superstructure. Reaction loads are applied to the substructure (piers and abutments) through the fixed bearings and by shear or friction through the expansion bearings. The aerial structure must also resist lateral components of the longitudinal loads on curved track. When the cumulative resistance of the fastening devices (rail clips) along a length of superstructure is overcome, the superstructure slides relative to the rail.

Since CWR is not able to expand or contract, temperature increases above the rail installation temperature because compressive forces that could buckle the rail. Rail fasteners prevent buckling of the rail. Temperature decreases below the rail installation temperature cause tensile forces that increase the probability of a rail break (a “pull-apart”). A rail break not only results in a gap in the rail that could cause a derailment, it creates unbalanced forces and moments in the aerial structure.

Based on these thermal effects, there are three problems to address in the design of aerial structures with CWR:

- Controlling the stresses in the rail attributed to the different longitudinal motions of the rail and the superstructure because of temperature changes or other causes
- Controlling the rail break gap size and resulting loads into the superstructure
- Transferring the superstructure loads and moments into the substructure

A structural system is formed when CWR is installed on a direct fixation aerial structure. The major components of this system include the following:

- Long, elastic CWR, with ends anchored in the track beyond the abutments
- Elastic rail fasteners that attach the rails directly to the superstructure
- The elastic superstructure
- Elastic bearings connecting the girders to the substructure
- The elastic substructure anchored to rigid foundations

There are a number of principal design factors that affect the magnitude of the interaction movement and forces between the rails and the structure, including the following:

- The composition of the girder material (steel or concrete), which will affect the expansion/contraction response to temperature changes
- The girder length and type (simple span or continuous), which will affect the magnitude of thermal movement that the rail fasteners must accommodate
- The girder's support pattern of fixed and expansion bearings from adjacent spans on the piers
- The magnitude of the temperature change
- The rail fastener layout and longitudinal restraint characteristics, including these four concepts of fastener and restraint:
  - Frictional restraint developed in mechanical fasteners
  - Elastic restraint developed in elastic fasteners
  - Elastic restraint developed in elastic fasteners with controlled rail slip
  - Elastic and slip fasteners installed in accordance with the expected relative movements between girder and rail: to control rail creep, install sufficient elastic fasteners near the fixed bearing; to provide full lateral restraint and minimal longitudinal restraint, install slip fasteners over the balance of the girder length

Depending on the method used to attach the rails to the structure, the structural engineer must design the structure for longitudinal restraint loads induced by the fasteners, horizontal forces due to a rail break, and radial forces caused by thermal changes in rails on curved alignments.

Today's designer can use computer models to simulate the entire structure/trackwork system to account for variations in the stiffness of the substructure and the dissipation of rail/structure interaction forces due to the substructure's deflection.

The thermal force in the rail is calculated by the following equation:

$$F_r = A_r * E_{ra} * (T_i - T_o)$$

Where  $F_r$ =thermal rail force

$A_r$ =cross sectional area of the rail

$E_r$ =modulus of elasticity of steel

$\alpha$ =coefficient of thermal expansion

$T_i$ =final rail temperature

$T_o$ =effective construction temperature of the rail

Various solutions have been implemented in an attempt to minimize the interaction forces caused by placing CWR on aerial structures, including use of the following:

- Ballasted track instead of direct fixation track
- Zero-longitudinal-restraint fasteners
- High-restraint fasteners near the structure's point(s) of fixity and low-restraint fasteners on the remainder of the structure
- A series of rail expansion joints and low-restraint fasteners to allow the rail to move independently of the structure; this requires highly restrained zones to transfer traction and braking forces to the structure, [9].

### **Improved slab track over Rail transit bridges:**

Direct fixation (DF) track is the most common LRT trackform for use on aerial structures and in tunnels. It is also often used in areas where it would be difficult to maintain ballasted track in proper alignment and surface.

Direct fixation track is a “ballastless” track structure in which the rail is mounted on direct fixation fasteners that in turn are anchored to an underlying concrete slab. The slab could be a slab on grade, an aerial structure deck surface, or a concrete tunnel invert. Direct fixation track is also used for construction of at-grade track under unusual circumstances, such as when there is a relatively short segment of at-grade track between two direct fixation track structure decks.

The advantages of the direct fixation trackform include the following:

- Absorbs noise and vibration and provides vertical flexibility with resilient direct fixation fasteners

- Improves aesthetics by using shallower, less massive structures
- Results in a relatively low dead load compared to other trackforms
- Provides both electrical isolation and a means to efficiently adjust the line and grade of the track with rail fasteners
- Requires less maintenance and is easier to maintain than alternatives
- Retains track geometry much longer than ballasted track
- Provides relatively good ride quality
- Offers relatively good live load distribution

The important thing to note is that the practical construction tolerances for bridges, tunnel inverts, and slabs-on-grade are appreciably looser than those required for the top-of-rail profile of railway tracks. Installation of direct fixation trackwork therefore requires either unusually tight construction tolerances for the underlying support structure or a relatively simple way of compensating for deviations. In the case of bridges, the construction tolerances for the construction of the deck, including residual deck camber, are usually far less stringent than are necessary to provide a satisfactory top-of-rail profile.

Because of this, most rail transit systems use a concrete pad, or plinth, approximately 6 inches [150 mm] tall to support the direct fixation fasteners and anchor them to the superstructure. Intermittent gaps are provided along the length of the plinths to accommodate deck drainage and to provide openings for electrical (systems) conduits placed on the deck. In addition, the deck slab sometimes incorporates recesses to accept the second-pour plinths. Each plinth recess forms a shear key to help resist the lateral loads from the rail and vehicles and also slightly reduces the effective height of the plinth. The second-pour concrete plinths are carefully constructed to meet the alignment and profile requirements of the CWR and fasteners.

The reinforcing in the concrete plinths should be designed to assist in resisting the loads imparted by the rail fastener anchors to the plinths. In addition to the compression forces on the plinths, the fastener anchors must resist horizontal shear and tension forces as a result of the braking, accelerating, and lateral forces from the LRVs.

Although direct fixation is the current standard practice for deck construction, there are some disadvantages to consider. Disadvantages of direct fixation deck include the following: [9]

- Rail/structure interaction must address thermal forces
- Relatively high initial cost
- Tight construction control required
- Specialized rail fasteners required

## 2.2.4. Review of Major Types of Railway Track System

### 2.2.4.1. Slab Track and Ballasted Track

Although most of the current railway tracks are still of a traditional ballasted type, recent applications tend more and more towards non-ballasted track. The major advantages of slab track are: low maintenance, high availability, low structure height, and low weight. In addition, recent life cycle studies have shown that from the cost point of view, slab tracks might be very competitive.

Experiences In high-speed operation have revealed that ballasted tracks are more maintenance intensive. In particular, due to churning up of ballast particles at high-speed, serious damage can occur to wheels and rails, which is of course prevented In the case of slab track.

With the design of railway lines factors like life cycle cost, construction time, availability and durability play an increasingly important role. In this respect non-ballasted track concepts offer good opportunities. With the growth of traffic intensity becomes more and more difficult to carry out maintenance and renewal work.

In the past new projects were mainly assessed on the basis of investment costs, whereas today the principle of life cycle costing is strongly emerging. As a result of these new attitude, ballasted track concepts loose attractiveness In favor of slab track systems.

### 2.2.4.2. Ballasted track versus slab track

The general problem which occurs with ballasted track is the slow deterioration of the ballast material due to traffic loading. Ballast consists of packed loose granular material, of which the grains wander, wear, and break up causing increasing geometrical unevenness and clogging of the ballast bed by fine particles which cause drainage problems. Therefore, regular maintenance is time after time needed to restore the track alignment. It is clear that slab track provides far higher lateral and longitudinal stability, and deviations of the track alignment are smaller and less likely to occur, given a good slab foundation. The condition of the track geometry is,

therefore, very good and will remain likewise, Improving passenger comfort and considerably decreasing the amount of maintenance.

### **Ballasted track**

In comparison to slab track, advantages of ballasted track are;

- Proven technology,
- Relatively low construction costs;
- Simple replacement of track components;
- Relatively simple correction of track geometry (maintenance);
- Small adjustments of track lay-out (curves) possible;
- Good drainage properties;
- Good elasticity;
- Good damping of noise

In comparison to slab track, disadvantages of ballasted track are;

- The tendency of the track to "float" in both longitudinal and lateral direction after a period of time;
- Limited non-compensated lateral acceleration in curves caused by the limited lateral resistance provided by ballast;
- Pulverization of the ballast grains In the ballast bed resulting in particles damaging the rail and wheels;
- Reduced permeability due to contamination, the wear of the ballast, and intrusion of fine particles from the sub grade;
- A relatively heavy and high track structure requiring stronger constructions for bridges and via- ducts

The rate at which the track deteriorates is closely related to the quality of the track components and the way they are assembled (at construction), the track geometry, the homogeneity of the subgrade layers, and the supporting capacity of the sub ballast.

## Slab track

In comparison to ballasted track, the advantages of slab track (or ballastless track) are in general reduction of maintenance and a higher stability of the track:

- The track is to large extent maintenance free. Maintenance work like tamping, ballast cleaning, and track lining is unnecessary. Maintenance costs amount to 20 - 30% of the maintenance costs of ballasted track;
- increased service life, and possibility of almost full replacement at the end of the service life,
- Near maximum availability and hardly any hindrance to residents due to scarce nightly maintenance works;
- No drag forces at ballast with the passing of high-speed trains; the unconditional use of electro-magnetic wheel brakes,
- The excess of super elevation and cant deficiency of the track with mixed use of freight- and passenger trains does not cause altering of the track position,
- Simple corrections up to 26 mm in vertical position and 5 mm in horizontal position are possible to counteract small deformations;
- Reduced structure height and weight

Other reasons for applying slab track can be:

- The lack of suitable ballast material;
- Track accessibility to road vehicles;
- The abatement of noise and, in particular, vibration nuisance;
- Preventing the release of dust from the ballast bed into the environment.

In comparison to ballasted track. The disadvantages of slab track or ballastless track are in general.

- Higher construction costs;
- Higher airborne noise reflection,
- Large alterations in track position and super elevation can only be made possible by substantial amounts of work;

- Adaptability to larger displacements in the embankment is relatively small;
- In case of derailment, repair works will take much more time and effort;
- Transitions between ballasted track and slab track require attention.

The application of slab track may require extensive measures concerning the preparation of the foundation. The sub layers must be homogenous and capable of bearing the imposed loads without significant settlements. The resulting high construction costs have prevented widespread use of slab track for main lines so far. Continuation of slab track across bridges could pose problems if certain typical mechanical behavior is not considered. A bridge provides a solid foundation for slab track, but acts in fact as a discontinuity. Due to temperature changes longitudinal movements of the bridge-structure will occur. Traffic loading causes the spans of the bridge to bend and the edges to twist over the supports. The superstructure must be able to cope with these movements.

By applying slab track systems on short bridges, several solutions are possible:

- Fasteners with reduced clamping force: the movements of the bridge are compensated in the rail fastenings with reduced clamping force if the sleepers on top of the reinforced concrete roadbed are rigidly connected to the bridge deck or direct rail fastening systems are used.
- Embedded in bridge decks: in case of a continuous rail-support rigidly connected to the bridge, Maximum active extendable bridge-spans up to 15 m are permitted. Larger spans are possible by applying extension devices and joints
- Sliding slabs. the bridge structure can freely move underneath the slab track which "glides" on top. This option is limited to freely extendable bridge-spans up to 25 m.
- Track frame on roadbed- the track lies freely movable on top of a concrete or asphalt-concrete roadbed. This solution exists due to possible motions and twisting of the sleepers on top of bridge- structures and spans up to 10 m with frame spans limited to 25 m, [10].

### 2.2.5. Review of Design of box girder railway bridges

#### General

The box girder is the most flexible bridge deck form. It can cover a range of spans from 25 m up to the largest non-suspended concrete decks built; of the order of 300 m, [14]. Single box girders may also carry decks up to 30 m wide. For the longer span beams, beyond about 50 m, they are practically the only feasible deck section. For the shorter spans they are in competition with most of the other deck types.

The advantages of the box form are principally its high structural efficiency, which minimizes the pre-stress force required to resist a given bending moment, and its great torsional strength with the capacity this gives to re-centre eccentric live loads, minimizing the prestress required to carry them.

The box form lends itself to many of the highly productive methods of bridge construction that have been progressively refined over the last 50 years, such as precast segmental construction with or without epoxy resin in the joints, balanced cantilever erection either cast in-situ or coupled with precast segmental construction, and incremental launching.

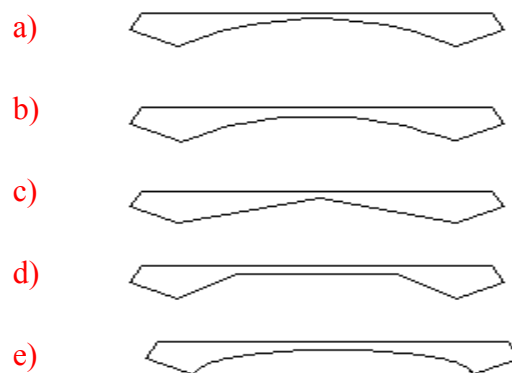
The visual impact of the depth of the box is reduced if it has a trapezoidal cross section. This inclination of the web makes it appear darker than a vertical surface, an impression that is heightened if the edge parapet of the deck is vertical. The trapezoidal cross section is frequently economical as well as good looking. In general, the width of the top of the box is determined by the need to provide points of support to the top slab at suitable intervals. The cross section area of the bottom slab is logically determined at mid-span by the need to provide a bottom modulus sufficient to control the range of bending stresses under the variation of live load bending moments. For a box of rectangular cross section of span/depth ratio deeper than about 1/20, the area of bottom slab is generally greater than necessary, resulting in redundant weight. Choosing a trapezoidal cross section allows the weight of the bottom slab to be reduced. Close to the piers, the area of bottom slab is determined by the need to limit the maximum bending stress on the bottom fibre and to provide an adequate ultimate moment of resistance. If the narrow bottom slab defined by mid-span criteria is inadequate, it is simple to thicken it locally.

For a very wide deck that has a deep span/depth ratio, this logic may give rise to webs that are inclined at a very flat angle. The designer should be aware of the difficulties in casting such webs, and make suitable allowances in specifying the concrete and in detailing the reinforcement.

Also, an important consideration in the design of box section decks is the distortion of the cross section under the effect of eccentric live loads. The effect of this distortion is reduced in a trapezoidal cross section. Boxes may have a single cell or multiple cells. Furthermore, it is more difficult to build multi-cell boxes, and it is worthwhile extending the single-cell box as far as possible before adding internal webs.

Variation of depth: Once the span of a box section deck exceeds about 45 m, it becomes relevant to consider varying the depth of the beam. This is not an automatic decision as it depends on the method of construction. For instance, when the deck is to be precast by the counter-cast method, if the number of segments is relatively low it is likely to be more economical to keep the depth constant in order to simplify the mould. On the other hand, if the deck is to be built by cast-in-situ balanced cantilevering, it is relatively simple to design the mould to incorporate a variable depth, even for a small number of quite short spans.

Clearly, this decision also has an aesthetic component. The depth may be varied continuously along the length of the beam, adopting a) circular, b) parabolic, c) Islamic, d) straight, e) elliptical profile, Fig 2.2. Alternatively, the deck may be haunched. The decision on the soffit profile closely links aesthetic and technical criteria.

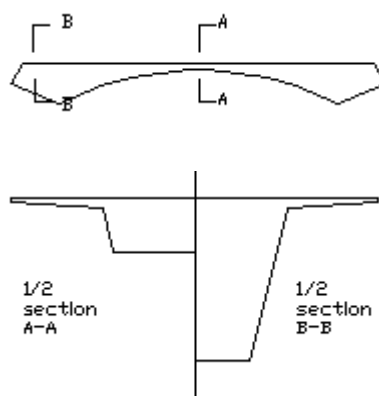


**Figure 2.2:** Different shape of box girder with depth variation

For instance, when the depth varies continuously it is often judged that an elliptical profile is the most beautiful. However, this will tend to create a design problem towards the quarter points, as at these locations the beam is shallower than optimal, both for shear resistance and for bending strength. As a result, the webs and bottom slab may need to be thickened locally, and the prestress increased. However, the economic penalty may be small enough to accept. The Islamic form is likely to provide the most flexible method of optimizing the depth at all points along the girder, but the cusp at mid-span may give a problem for the profile of the continuity tendons while for long spans the greater weight of the deeper webs either side of mid-span implies a significant cost penalty. Also, the appearance may not be suitable for the particular circumstance.

When the change in the depth of the box is not too great, haunched decks are often chosen for the precast segmental form of construction, as they reduce the number of times the formwork must be adjusted, assisting in keeping to the all-important daily cycle of production. However, here again there is a conflict between the technical optimization of the shape of the beam and aesthetic considerations. The beginning of the haunch is potentially a critical design section, both for shear and bending. This criticality is relieved if the haunch extends to some 25–30 per cent of the span length.

However, the appearance of the beam is considerably improved if the haunch length is limited to 20 per cent of the span or less. When variation of the depth is combined with a trapezoidal cross section, the bottom slab will become narrower as the deck becomes deeper, Figure 2.3. This has an important aesthetic impact, as well as giving rise to complications in the construction.



**Figure 2.3 :** Depth variation vs cross section

It is easier to cope with a haunched deck than a continuously varying depth, as in the former case the narrowing of the bottom slab is limited to a relatively small proportion of the segments, and the rate at which it narrows is constant.

If the bottom slab is maintained at a constant width, the web surfaces will be warped. For a deck that has a continuously varying depth, the timber shutters of a cast-in-situ cantilevering false work can accept this warp, whereas this may not be the case for the steel shutter of a precast segmental casting cell. However, for a hunched deck the warp would be introduced suddenly at the beginning of the haunch, which would probably be impossible to build, and would look terrible.

A successful detail employed on several occasions on precast segmental decks is to adopt a trapezoidal box which runs the full length of the span, and to add a parallel sided haunch. Refinements are to define the haunch by a step in from the trapezoidal section and to finish the haunch with a small step rather than fairing it into the soffit;

An additional advantage of this geometry is that it makes it possible to locate the bearings directly beneath the axis of the webs, greatly simplifying the pier diaphragm.

The number of webs per box: One of the principal aims of the bridge designer is to minimize redundant material. This is the discipline that is the basis not only for economy, but also for technical innovation and for beauty. Economy of materials in the design of box sections is achieved principally by minimizing the thickness of the deck members. This has the dual benefit of reducing the dead weight bending moments and shear forces, and reducing the cross-sectional area to be prestressed.

The choice of the method of adapting to a variable width of deck can have a very significant impact on the appearance of the deck, particularly if the width of the side cantilever is changed. Where possible, the side cantilever should remain constant, or should vary only slightly, and gradually. However, often this is not possible, and the various methods of widening the deck must be evaluated for both economy and appearance, [14].

### 2.2.5.1. Method of Analysis

#### a. Transverse Analysis:

The transverse design of box girder segments for flexure shall consider the segment as a rigid box frame. Flanges shall be analyzed as variable depth sections considering the fillets between the flange and webs. Combinations of track loads, if the structure may support more than one track, shall be positioned to provide maximum moments, and elastic analysis shall be used to determine the effective longitudinal distribution of wheel loads for each load location. Tracks shall be positioned on the structure in accordance with clearance policies. Consideration shall be given to the increase in web shear and other effects on the cross-section resulting from eccentric loading or unsymmetrical structure geometry.

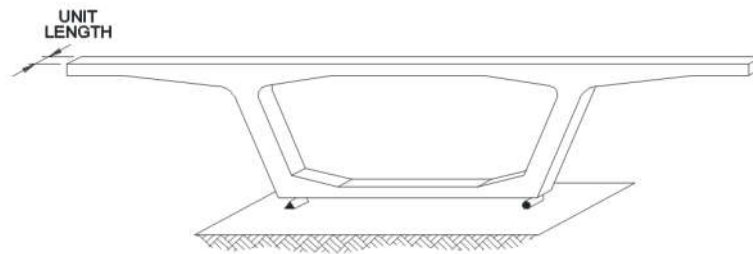
#### b. Longitudinal Analysis:

Elastic analysis and beam theory may be used to determine design moments, shears, and deflections. The effects of creep, shrinkage, and temperature differentials shall be considered, as well as the effects of shear lag.

### 2.2.5.2. Design Approach

To correctly represent the final system of the box girder, one would need to do a three dimensional analysis and incorporate all loads the box is subjected to along with proper boundary conditions. Due to complexity of this type of analysis, in particular the application of prestressing to three dimensional systems, this is seldom done. In lieu of this complex analysis, it is common practice to model the box as a 2-D (two dimensional) plane frame of unit length, as shown in Fig 2.4. If the thicknesses of the web and bottom slab vary along the length of the bridge, several 2-D frames may have to be analyzed in order to obtain a more representative interpretation of these varying cross-sectional properties. The 2-D frame model allows for load distribution to the webs and slab members relative to their stiffness.

A typical 2-D frame model is assumed to be supported at the lower end of the webs as shown in Figure While it could be argued that different boundary conditions exist for this model, this simplified assumption produces reasonable results.



**Figure 2.4:** Typical 2-D model of segmental bridges section

Live Load configuration should be strategically placed in order to produce the worst condition

Listed below are some common points where stresses are checked:

- Maximum negative bending moment at the root of deck overhang
- Maximum positive and negative bending moments at the center line between two webs
- Maximum negative bending moment in the top deck at the interior face of the webs
- Maximum negative and positive bending moments in the webs and bottom slab
- Maximum negative moment in the deck overhang where the taper begins

### Deck Design

The top deck of a box girder is subjected to complex external forces, static and dynamic loads, thermal gradients, construction, creep and shrinkage effects. Proper consideration should be given to these effects to prevent cracking and deterioration. Studies have shown that transverse post-tensioning of top decks improves long-term deck durability and results in low life cycle cost. It is recommended that for all post-tensioned box girders the top deck be transversely post-tensioned, even for short overhangs. For bridges not subjected to freeze-thaw action and de-icing chemicals, at least the deck should be partially prestressed. The top deck should be designed using elastic methods and then checked for ultimate limit states, not the other way around, [12]

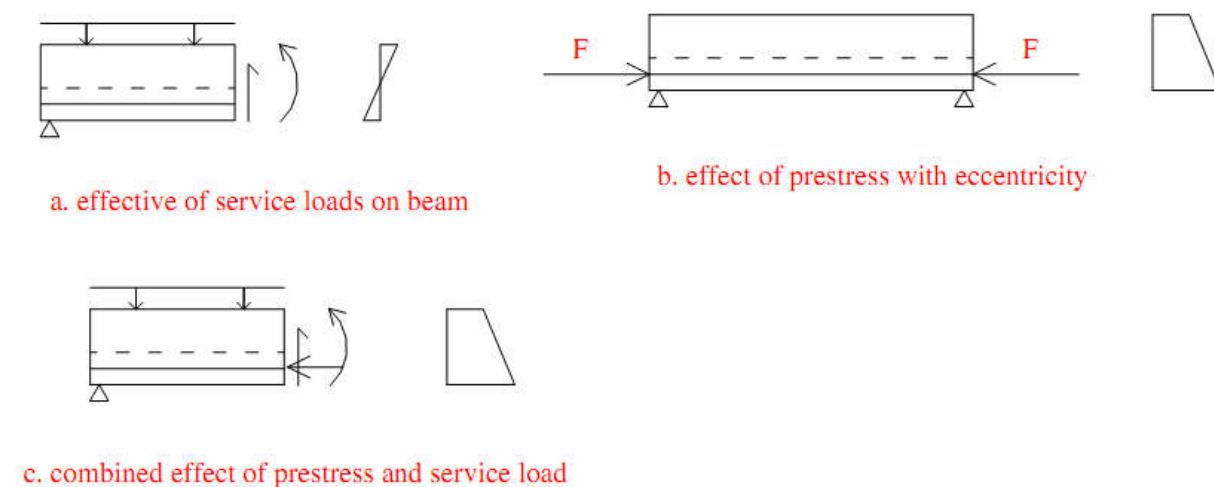
## 2.2.6. Review of Basic of Pre-stressed Concrete Structure

### 2.2.6.1. Introduction

#### Prestressed concrete

Prestressing can be defined as the application of pre-determined force or moment to a structural member in such a manner that the combined internal stresses in the member resulting from this force or moment and any anticipated condition of external loading will be confined within specific limits.

In general, prestress involves the imposition of stresses opposite in sign to those, which are caused by the subsequent application of workshop loads. For instance for prestressing wire placed eccentrically, the force in tendon produce an axial compression and hogging moment in the beam. While under service loads the same beam will develop sagging moments. Thus, it is possible to have the entire section in compression when service loads are imposed on the beam as shown in Fig 2.5. This is the main advantage of prestressed concrete. It is well known that reinforced concrete cracks in tension. But there is no cracking in fully prestressed concrete since the entire section is in compression. Thus, it can be said that prestress provides a means for efficient usage of the concrete cross-section in resisting the external loads.



**Figure 2.5:** pre-stress system

### 2.2.6.2. Advantage and Disadvantages of Prestressed Concrete

- It is free of cracks under working loads
- Sections are much smaller than that of the corresponding reinforced concrete structure
- Increased quality to resist impact, high fatigue resistance and increased live load carrying capacity.

On the other hand, prestressed concrete also exhibit certain disadvantages which includes:

- It requires high strength concrete that may not be easy to produce.
- It uses high strength steel, which might not be locally available
- It requires end anchorage, end plates, complicated formwork
- Labor cost may be greater, as it requires trained labor
- It calls for requires better quality control

### 2.2.6.3. Prestressing Systems

The prestress in concrete structure is induced by either of the two processes. Pre tensioning and post tensioning Pre-tensioning is accomplished by stressing wires, or strands called tendon to a pre-determined amount by stretching them between anchoring posts before placing the concrete. The concrete is then placed and the tendons become bonded to the concrete throughout their length. After the concrete has hardened, the tendon will be released from the anchoring posts. The tendon will tend to regain their original length by shortening and in this process they transfer a compressive stress to the concrete through bond. The tendons are usually stressed by hydraulic jacks.

The other alternative is post- tensioning. In post-tensioning, the tendons are stressed after the concrete is cast and hardened to certain strength to withstand the prestressing force. The tendon are stressed and anchored at the end of the concrete section. Here, the tendons are either coated with grease or bituminous material or encased with flexible metal hose before placing in forms to prevent the tendons from bonding to the concrete during placing and curing of concrete. In the latter case, the metal hose is referred to as a sheath or duct and remains in the structure. After the

tendons are stressed, the void between tendon and the sheath is filled with grout. Thus the tendons are bonded with concrete and corrosion is prevented.

Post-tension prestressing can be done on site. This procedure may be very important for certain cases. For large spans elements in building or bridges, the method may not be feasible as it requires transporting such members from precasting plant to a job site.

In post-tensioning it is necessary to use some device to attach or anchors ends of tendon's to the concrete section. These devices are called end anchorage.

The main difference between the two pre stressing system is:-

- i) Pre tensioning is mostly used for small member, whereas post-tensioning is used for larger spans.
- ii) Post-tensioned tendon can be placed in the structure with little difficulties in smooth curved profile. Pre-tensioned tendon can be used for curved profile but need extensive plant facilitates
- iii) Pre-tensioning system has the disadvantage that the abutment used in anchoring the tendon has to be very strong and cannot be reused until the concrete in the member has sufficiently hardened and removed from bed.
- IV) Lose of prestress in pre-tensioning is more pronounced than that of post-tensioning.

#### **2.2.6.4. Loss of Prestress**

It is difficult to measure the amount of prestress actually present in a prestressed concrete member. Only, the total force in tendon at the time of prestressing can be conveniently determined. Due to several factor to be presented subsequently the initial prestressing force in tendons may be altered with time. There will be loss of stresses in the steel.

The most common type losses that will occur in a prestressed concrete member are explained in the next subsections.

### a. Loss Due To Elastic Shortening (ES)

As the prestress is transferred to the concrete the member shortens and the prestressing steel shortens with it. Therefore, there is a loss of prestress in the steel. The loss of prestress due to this elastic shortening can be easily computed. Consider a pre-tensioned simple beam. First, let's assume the tendon profile is straight and at c.g.c level. (No bending).

The strain in concrete may be expressed by  $P$  is the total prestressing force just after transfer

$$\epsilon_c = \frac{P}{A_c \cdot E_c}$$

The strain in steel may be expressed by

$$\epsilon_s = \frac{P_0 - P}{A_s \cdot E_s}$$

$P$  is total prestressing force before transfer

$$\epsilon_c = \epsilon_s$$

$$\text{Or, } \frac{P}{A_c \cdot E_c} = \frac{P_0 - P}{A_s \cdot E_s}$$

$$\text{Or, } \frac{P_0}{P} = \frac{A_c + m \cdot A_s}{A_c} = \frac{A_g}{A_c}$$

The loss of pre stress  $\Delta\sigma$  is

$$\text{ES} = \Delta\sigma = \frac{P_0 - P}{A_g} = \frac{m \cdot P}{A_c} = \frac{m \cdot P_0}{A_g}$$

Where  $m$  = modular ratio

Loss of prestress due to elastic shortening of concrete, whether the prestressing force is applied at centroid of the section or not, can be given by

$$\text{ES} = \frac{E_s}{E_c} * f_{ci}$$

Where  $f_{ci}$  - is the stress in concrete at c.g.s level due to prestressing force at transfer ( $P_0$ ) and due to dead load moment at the section being considered.

In post tensioning, the tendons are not usually stretched simultaneously. Moreover, the first tendon that is stretched is shorten by subsequent stretching of all other tendon. Only the last tendon is not shorten by any subsequent stretching. An average value of strain change can be computed and equally applied to all tendons.

### b. Loss Due To Creep of Concrete

Creep is the property of concrete by which it continues to deform with time under sustained load at unit stresses within the accepted elastic range. This inelastic deformation increase at decreasing rate during the time of loading and its magnitude may be several times larger than that of the short term elastic deformation. The strain due to creep varies with the magnitude of stress. It is a time dependent phenomena. Creep of concrete result in loss in steel stress.

The loss of prestress due to creep of concrete may be computed as follows.

Creep coefficient is defined as

$$\theta = \frac{\epsilon_{cp}}{\epsilon_c}$$

Where

$\epsilon_{cp}$  is creep strain

$\epsilon_c$  is initial strain in concrete

Elastic strain in concrete at c.g.s level is given as

$$\epsilon_c = \frac{\sigma_{cs}}{E_c}$$

$$\epsilon_{cp} = \theta * \frac{\sigma_{cs}}{E_c}$$

Where  $\sigma_{cs}$  - Stress in concrete at c.g.s level due to sustained load. The strain in concrete at c.g.s levels due to creep equals the decrease in strain of steel.

$$\epsilon_{cp} = \epsilon_s = \frac{\Delta\sigma}{E_s}$$

$$\text{OR, } \frac{\Delta\sigma}{E_s} = \frac{\theta * \sigma_{cs}}{E_c}$$

$$\text{Or } \Delta\sigma = m \cdot \theta \cdot \sigma_c$$

Where m –modular ratio

The value of the creep coefficient will take different value depending on the age of concrete at loading to account the variation of modulus of elasticity of concrete with time. values to 2.2, 1.6 or 1.1 could be assumed for age of concrete at loading 7 days, 28 days or 1 year respectively[2]. It takes almost one year to develop full creep strains; therefore, only stresses of permanent nature should be used to compute creep. Stresses due to dead load and pre stress should be considered to compute creep. In pre-tensioned member, prestress loss due to creep is higher than post-tensioned member because, in the former, the prestress is imposed when the concrete is in its early stage of curing.

### c. Loss due to shrinkage of concrete

Shrinkage in concrete is a contraction due to drying and chemical changes. It is dependent on time and moisture condition but not on stress. The shrinkage strain varies due to several factors and may be range from 0.0000 to 0.0010 and in some instant beyond. At one extreme if the concrete is stored under water or under very wet conditions, the shrinkage may be insignificant. There may ever be expansion for some types of aggregates and cements. At the other extreme, for a combination of certain cements and aggregates and with the concrete stored under very dry conditions, shrinkage values as high as 0.0010 can be expected.

The amount of shrinkage varies widely, depending on the individual conditions. For the propose of design, an average value of shrinkage strain would be about 0.0002 to 0.0006 for usual concrete mixtures employed in prestressed construction.

As explained above, shrinkage of concrete is influenced by many factors but in this work the most important factors volumes to surface ratio, relative humidity and time from end of curing to application of prestress are considered in calculation of shrinkage losses. An average strain value of  $550 \times 10^{-6}$  mm/mm is taken in computational work. And modification factors will also be used to account the influence of the above-mentioned factors.

The loss of prestress due to shrinkage is given by

$$SH = \epsilon_{sh} * \epsilon_s$$

Where  $\epsilon_{sh}$  – shrinkage strain

$\epsilon_s$  – Modulus of elasticity of pre stressing steel

#### d. Loss due to relaxation of steel

Relaxation is assumed to mean the loss of stress in steel under nearly constant strain at constant temperature. It is similar to creep of concrete. Loss due to relaxation varies widely for different steels and its magnitude may be supplied by the steel manufactures based on test data. This loss is generally of the order of 2-8% of the initial steel stress.

#### e. Frictional Losses in Post-tensioned Members

Frictional loss occurs only in post-tensioned member. The friction between tendons and the surrounding material is not small enough to be ignored. This loss may be considered partly to be due to length effect (wobble effect) and partly to curvature effect. In straight elements, it occurs due to wobble effect and in curved ones, it occurs due to curvature and wobble effects.

If angle of curve is  $\alpha$  and  $P_1$  is force on pulling end of the curve, then force  $P_2$  on the other end of the curve, then force  $P_2$  on the other end of the curve, as shown in Fig 2.6.

$$P_2 = P_1 * e^{-\alpha * u}$$

Similarly, the relation between  $P_1$  and  $P_2$  due to length effect (wobble effect) is given as

$$P_2 = P_1 * e^{-K * L}$$

The combined effect is

$$P_2 = P_1 * e^{-(\alpha * u + K * L)}$$

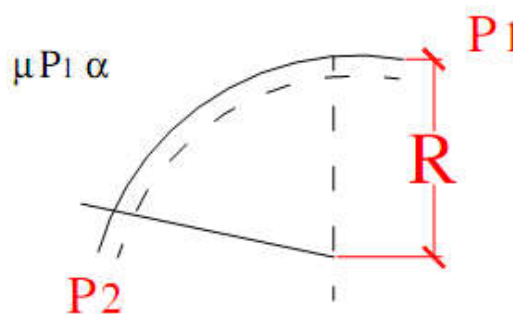


Figure 2.6: Relation between  $p_1$  &  $p_2$

Where  $\mu$  = Coefficient of friction

$K$  = Wobble friction coefficient per unit length of cable

$\alpha = L/R$  , length of the curve divided by the radius of curvature

$P_1$  = Jacking force

The value of  $\mu$  and  $K$  for different type of cables can be read from Codes

#### f. Total Amount of Losses

The total amount of prestressed losses is given by the cumulative figure of losses value outlined so far .it is useful in evaluating the effective prestresses. The total amount of losses to be assumed in design will depend on the basis on which initial prestressed is measured. Depending on the definition of initial prestress, the amount of losses to be deducted will vary accordingly. If the jacking stress minus the anchorage loss is taken as the initial prestress, then the total losses will include the elastic shortening, creep and shrinkage in concrete plus relaxation of steel. This is the most common practice. On the other hand If the jacking stress itself is taken as initial prestress, the anchorage losses should be included in the total losses. For post-tension member, for points far away from the jacking end, frictional loss should be deducted in effective prestress calculation. But, sometimes, the tendons are stressed temporarily beyond normal jacking stress to compensate for the losses from anchorage and friction. In such case the anchorage and frictional losses may be left out.

The magnitude of all losses other than friction and anchorage are usually estimated in percent of initial pre stress. For average steel and concrete properties cured under average air conditions, the value given in [table 2.4](#) could be taken as representative values [14].

	Pre-tensioning (%)	Post-tensioning (%)
Elastic Shortening	4	1
Creep of concrete	6	5
Shrinkage of concrete	7	6
Steel relaxation	8	8
Total	25	20

**Table 2.4:** Total amount of loss

## 2.3. Review of Bridge Construction Methods

### 2.3.1. Overview of Construction Method

Bridges, that are typically concrete box structure types, constructed using repetitive elements that are progressively connected together to form a completed structure. Construction Load Cases are particularly critical considerations in developing the design and building methods for segmental bridges. Before the advent of segmental construction, concrete bridges would often be made of several precast girders placed side by side, with joints between girders being parallel to the longitudinal axis of the bridge. With the modern segmental concept, the segments are slices of a structural element between joints which are perpendicular to the longitudinal axis of the structure.

When segmental construction first appeared in the early 1950s, it was either cast in place as used in Germany by Finsterwalder et al., or precast as used in France by Eugène Freyssinet and Jean Muller. The development of modern segmental construction is intertwined with the development of balanced cantilever construction. Balanced cantilever construction method will be discussed briefly under assessment of balanced cantilever construction [16].

By the use of the term balanced cantilever construction; we are describing a phased construction of a bridge superstructure. The construction starts from the piers cantilevering out to both sides in such a way that each phase is tied to the previous ones by post-tensioning tendons, incorporated into the permanent structure, so that each phase serves as a construction base for the following one.

The first attempts to use balanced cantilever construction, in its pure form, were made by Baumgart, who in 1929 built the Ríó Peixe Bridge in Brazil in reinforced concrete, casting the 68-m-long main span in free cantilevering. The method did not really prosper, however, until the post-tensioning technique had been sufficiently developed and generally recognized to allow crack-free concrete cantilever construction.

From 1950, several large bridges were built in Germany with the use of balanced cantilever construction with a hinge at midspan, using cast-in-place segments, such as:

- Moselbrücke Koblenz, 1954: Road bridge, 20 m wide, with three spans of 101, 114, 123 m plus short ballasted end spans hidden in large abutments; the cross section is made up of twin boxes of variable depth, connected by the top slab.
- Rheinbrücke Bendorf, 1964: Twin motorway bridges, 1,031 m long, with three main river spans of 71, 208, 71 m, built-in free cantilever construction with variable depth box sections.
- In France, the cantilever construction took a different direction, emphasizing the use of precast segments.
- Cast-in-place segmental construction is a relatively slow construction method. The work is performed in situ, i.e., exposed to weather conditions. The time-dependent deformations of the concrete become very important as a result of early loading of the young concrete. This method requires a relatively low degree of investment (travelers).
- Precast segmental construction is a fast construction method determined by the time required for the erection. The major part of the work is performed in the precasting yard, where it can be protected against inclement weather. Precasting can start simultaneously with the foundation work. The time-dependent deformations of the concrete become less important, as the concrete may have reached a higher age by the time the segments are placed in the structure. This method requires relatively important investments in precasting yard, molds, lifting gear, transportation, and erection equipment. Therefore, this method requires a certain volume of work to become economically viable. Typically, the industrialized execution of the structure leads to higher quality of the finished product, [16].

#### 2.3.1.1. Development of Prestressed Segmental Bridges

Application of prestressed concrete for bridge construction after developed by French engineer Eugène Freyssinet, has spread widely thereafter. Only prestressing made the slender, long-span concrete bridges of today possible. The basic principle of prestressing is to induce an initial compressive force in the concrete that will balance tensile stresses that occur in the member under service conditions before any tensile stresses occur in the concrete and cause cracking. The two methods of inducing these stresses in the structure:

- By imposed forces from reinforcing steel that is prestressed to a certain degree;
- By imposed “artificial displacements of the supports”, e.g. bearings.

The second method is much less used because of high losses of the prestressing force due to concrete creep and shrinkage. Prestressing tendons that are used for the first method consist of high-strength steel and are fabricated as wires, strands, or bars. For a continuous beam on several supports, most tension will occur in the lower fibers of the cross-section around midspan and in the upper fibers above intermediate supports. It is therefore most useful to place tendons in the locations where tensile stresses will occur in the structure under service. This thought naturally leads to the idea of implementing longitudinal tendons in the beam that are not simply straight but follow a curve from the top above supports to the bottom at midspan and back to the next support. In Balanced Cantilever Construction the top cables in reaching out from the cantilever base to support the cantilever dead load are called cantilever beam cables; the bottom cables in the middle of the span are called integration cables.

Prestressed concrete, compared with normal reinforced concrete has a higher degree of sophistication and causes higher cost for labor and for the prestressing tendons; on the other hand it saves cost through more economical use of material. Only prestressing makes long and slender concrete spans possible at all. [17]

### **Major Issues in the Design of Precast Prestressed Segmental Concrete Bridges**

In 1994, the American Segmental Bridge Institute (ASBI) published in the PCI Journal the results of a survey in which the performance of 96 segmental concrete bridges of the United States and Canada was evaluated (Miller 1994). The major conclusions of this report were:

1. Segmental concrete construction performs well over time, with consistently high conditions ratings for bridges that have been in use for up to 30 years.
2. The performance of post-tensioned segmental concrete is similar to that of other prestressed concrete bridges.

Though it has been found that precast prestressed segmental construction is a very good method of construction, it is still not perfect. In this section are presented important design decisions and possible sources of problems related to precast prestressed segmental concrete

bridge. Some of the decisions and problems are not exclusive to precast prestressed segmental construction. It will be shown how these decisions and problems will affect the achievement of the four before mentioned objectives of bridge design.

### **Non-Segmental Prestressing vs. Segmental Prestressing**

Currently, short-span bridges (up to 50 meters) are most commonly build using precast pretensioned concrete I-girders, taking advantage of all the benefits of using prestressed concrete (PCI, 1992). Still, although longer spans can be achieved with prestressed concrete than with reinforced concrete, sometimes there were requirements for even longer spans that could not be achieved with just prestressed concrete. Other materials, like steel, had to be used. Therefore, segmental construction got its start as an alternative concept for bridging longer spans.

Segmental bridges are those that are constructed from a number of short transverse segments. Usually, prestressing is applied to segmental bridge construction, allowing the construction of longer and/or thinner spans. Prestressing is done by using post-tensioning tendons in the longitudinal direction of the beams. The most typical configuration in prestressed segmental bridge construction is the box girder. This is true because in most cases, box girders are the most efficient and economical design for a bridge (Podolny and Muller, 1982). In fact, the box girder has been used in almost every type of bridge construction including simple span, continuous spans, cantilevers, arches, stayed girders, and suspension. When using box girders, permanent post-tensioning can also be applied transversely to the top slab to increase the strength of the deck, and vertically to the webs to increase the shear capacity of the box girder.

The box girders used are mostly single cell and double cell. There have been cases where three-cell box girders has been used, but these segments need temporary stiffeners to prevent buckling during construction (Wium and Buyukozturk, 1984). Since getting started as an alternative to bridging longer spans, segmental construction continued development because it offers other advantages. One of them is that horizontally curved alignments, humps or sag vertical curves, and transitioned cross slopes can easily be accommodated with segmental construction. Also, when compared to other types of construction, segmental prestressed

construction can be done by a small work force performing repetitive tasks. Besides, with segmental construction it is possible to reduce significantly the interference with existing traffic, eliminating expensive detours.

Finally, it is normally believe that segmental construction protects the environment (Podolny and Muller, 1982). If an elevated guideway is constructed instead of constructing a highway, a road, or a railway using cut-and-fill type of construction, then an environmentally sensitive area would be less affected with the narrower path of the alignment construction. Then, if the guideway was constructed using segmental construction, longer spans would be possible and the construction of the superstructure could be done from top of the piers, thereby protecting the environment even more.

### **Precast vs. Cast-In-Place**

As mentioned before, segmental bridges are constructed from a number of short transverse segments. The segments can be either:

1. Precast, meaning they are manufactured in a precast yard on or off-site, which then are taken to the bridge location where they are assembled; or
2. Cast-In-Place, meaning they are cast in their final position. Choosing between any of the two will bring advantages and disadvantages.

The advantages of the using precast concrete over cast-in-place concrete in prestressed segmental bridge construction are:

1. The precast segments can be fabricated while the substructure is being built, which means a saving in construction time.
2. The fabrication of these segments is done by the repetitive use of industrialized manufacturing techniques. This provides the opportunity of achieving high quality and high strength concrete.
3. There is no need for falsework and everything can be accomplished from the top of the completed portions of the structure. This is very useful when traffic has to remain undisturbed under the bridge or for high-level crossings.

4. The effects of shrinkage and creep are substantially reduced. This happens because usually, the segments have already matured to full design strength by the time of erection and post-tensioning.
5. Precast construction is less sensitive to weather conditions than cast-in-place construction.

The advantages of using cast-in-place segments instead of precast segments in segmental construction are:

1. There is no required dimensional control of high degree during the manufacturing and erection of the segments. Connections of the ducts for tensioning cables between segments can be done more easily.
2. There is room for error in cast-in-place construction. If the actual camber does not agree with predicted camber, corrections can be made as the construction progresses by revising the alignment of the following segments.
3. Joints can be treated better for transfer of bending and shear stresses and for water tightness for protection of the tendons. Longitudinal reinforcing steel can be placed between the segments.
4. The size and weight of the cast-in-place segments is not limited to the transportation equipment, as it is for the precast segments.

### **Precast Prestressed Segmental Concrete Construction**

All the advantages of precast concrete, prestressed concrete, and segmental construction come together in precast prestressed segmental concrete construction. This type of construction is most widely used in the construction of bridges.

### **Fabrication of the Precast Segments**

The fabrication of the segments can be distinguished by the type of joint to be used between the segments themselves (PTI and PCI, 1978). This joint can either be a cast-in-place joint or a contact joint. When using cast-in-place joints (also known as wide joints or broad joints), the precision of line of segments depend more on how accurate is the casting

of the joint during erection and less the accuracy of the segments (PTI and PCI, 1978). Segments using wide joints can be cast separately.

When using contact joints (also known as match-cast joints), the connecting surfaces between the segments fit each other very accurately, so only a thin layer of filling material is needed for the joint. Sometimes no filling material is needed at all. Segments using match-cast joints are cast by either long-line method or the short-line method. Short line and long line method will be discussed under casting methods.

### **Railroad Bridges**

Segmental prestressed construction is as suitable for rail transit and railroads as it is for highways. The procedure for designing an elevated railway is similar to that of a high-way bridge. The difference lies in the larger design life and impact loads for railroad structures. Therefore, elevated railways are stockier than highway bridges.

In the case of a railway bridge, the actual load applied to the structure is much closer to the design load than in the case of a highway bridge. Therefore, fatigue and durability of railway structures could be a problem and need careful consideration (Podolny and Muller, 1982). This is made even more important due to the fact that the maintenance and repair of railway structures could cause unacceptable disturbance in the train operation. Usually, ballast is used across the railroad bridge so that the ties and rails are kept in proper alignment and grade separate from the bridge structure. Typically, for rail transit, the ballast is not used, and either the ties are fixed directly on top of the deck, or the track is set directly on pads and shims attached to the bridge deck, eliminating the use of ties and ballast. When using ties, they can be made of wood, steel, reinforced concrete, or prestressed concrete. Whichever system is used to fix the track to the deck, it should have drains or sufficient slope so that water is not trapped.

Until about 30-40 years ago, all tracks, in all networks, were laid by leaving gaps between consecutive rails, and then jointing the rails with fishplates (Profillidis, 1995). The gaps and fishplates were used to permit elongations in the rail due to temperature changes. The use of fishplates has many problems (Profillidis, 1995):

1. It significantly reduces passenger comfort.
2. It causes considerable wheel and rail fatigue and wear.
3. A bumping noise is produced when the vehicle's wheels run on top of it.
4. It greatly increases maintenance expenses.

An alternative to jointed rails is the continuous-welded rail (CWR). A CWR is formed by putting together discrete pieces of rail, eliminating the use of fishplates. No thermal expansions along the length of the rail are permitted. This generates forces in the rail that it must be able to resist.

The cost of installing CWR is higher than that of installing a jointed rail. Nevertheless, the CWR provides an adequate return in capital for the initial investment by reducing maintenance cost, improving track stability, permitting the achievement of higher speed by the vehicles, reducing power consumption, slowing the development in track defects, reducing noise, and improving passenger comfort (Profillidis, 1995). There is a tendency to use continuous-welded rails wherever possible (Fryba, 1996). However, with the introduction of the continuous-welded rail into railroad bridges and guideways, a new problem came up with regard to the interaction between the superstructure and the rail. New forces and stresses are developed with changes in temperatures, [18].

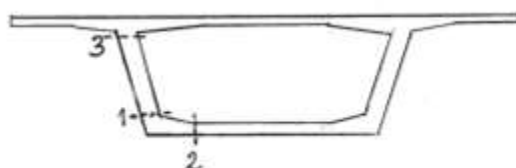
### 2.3.1.2. Casting method

#### 2.3.1.2.1. Cast-in-situ construction of boxes

One of the main disadvantages of box decks is that they are difficult to cast in-situ due to the inaccessibility of the bottom slab and the need to extract the internal shutter. Either the box has to be designed so that the entire cross section may be cast in one continuous pour, or the cross section has to be cast in stages.

##### a. Casting the deck cross section in stages

The most common method of building box decks in situ is to cast the cross section in stages. Either, the bottom slab is cast first with the webs and top slab cast in a second phase, or the webs and bottom slab constitute the first phase, completed by the top slab. When the bottom slab is cast first, the construction joint is usually located just above the slab, giving a kicker for the web formwork, position 1 in Figure below. A joint in this location has several disadvantages.



**Figure 2.7:** box section casting (a)

Alternatively, the joint may be in the bottom slab close to the webs, or at the beginning of the haunches, position 2. The advantages of locating the joint in the bottom slab are that it does not cross prestressing tendons or heavy reinforcement; it is protected from the weather and is also less prominent visually. The main disadvantage is that the slab only constitutes a small proportion of the total concrete to be cast, leaving a much larger second pour.

The joint may be located at the top of the web, just below the top slab, position 3. This retains many of the disadvantages of position 1, namely that the construction joint is crossed by prestressing ducts at a shallow angle, and it is difficult to prepare for the next pour due to the presence of the web reinforcement. In addition, most of the difficulty of casting the bottom slab has been re-introduced. The advantages are that the joint is less prominent visually and is

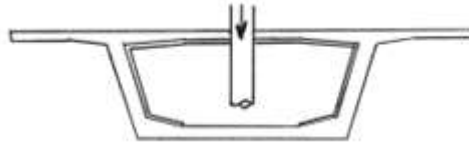
protected from the weather by the side cantilever, the quantity of concrete in each pour is similar and less of the shutter is trapped inside the box.

Casting a cross section in phases causes the second phase to crack due to restraint by the hardened concrete of the first phase. Although the section may be reinforced to limit the width of the cracks, it is not desirable for a prestressed concrete deck to be cracked under permanent loads. Eliminating cracks altogether would require very expensive measures such as cooling the second phase concrete to limit the rise in temperature during setting or adopting crack sealing admixtures

### **b. Casting the cross section in one pour**

There are two approaches to casting a box section in one pour. The bottom slab may be cast first with the help of trunking passing through temporary holes left in the soffit form of the top slab. This requires access for laborers to spread and vibrate the concrete, and is only generally possible for decks that are at least 2 m deep. The casting of the webs must follow on closely, so that cold joints are avoided. The fluidity of the concrete needs to be designed such that the concrete will not slump out of the webs. This is assisted if there is a strip of top shutter to the bottom slab about 500 mm wide along each web. This method puts no restriction on the width of the bottom slab, Figure 2.7(a).

Alternatively the deck cross section may be shaped so that concrete will flow from the webs into the bottom slab, which normally has a complete top shutter, Figure 2.8(b). This method of construction is most suitable for boxes with relatively narrow bottom flanges. The compaction of the bottom slab concrete needs to be effected by external vibrators, which implies the use of steel shutters. The concrete may be cast down both webs, with inspection holes in the shutter that allow air to be expelled and the complete filling of the bottom slab to be confirmed. Alternatively, concrete may be cast down one web first with the second web being cast only when concrete appears at its base, demonstrating that the bottom slab is full. The concrete mix design is critical and full-scale trials representing both the geometry of the cross section and density of reinforcement and prestress cables are essential.



A) Wide bottom slab cast through trunking



B) Narrow bottom slab with concrete cast down webs

**Figure 2.8** : Casting the cross section in one pour (b)

However the section is cast, the core shutter must be dismantled and removed through a hole in the top slab, or made collapsible so it may be withdrawn longitudinally through the pier diaphragm. Despite these difficulties, casting the section in one pour is under-used. The recent development of self-compacting concrete could revolutionize the construction of decks in this manner. This could be particularly important for medium length bridges with spans between 40 m and 55 m. Such spans are too long for twin rib type decks, and too short for cast-in-situ balanced cantilever construction of box girders, while a total length of box section deck of less than about 1,000 m does not justify setting up a precast segmental facility. Currently, it is this type of bridge that is least favorable for concrete and where steel composite construction is found to be competitive [14].

#### 2.3.1.2.2. Casting of segmental bridge section

All the casting methods to be discussed utilize the concept of match casting. The idea is to cast the segments so their relative erected position is identical to their relative casting position. This requires perfect fit between the ends of the segments and is accomplished by casting each segment directly against the face of the preceding one using a bond breaker to prevent bonding of the concrete. The segments are then erected in the same sequence in which they were cast.

### **a. Short line system**

The most common method for match casting segments is called the short line method. With this system, the rates of segment production will approach one segment per day in each line of forms. A good average to use for a project is four segments per line every five days. The length of the side forms is equal to the length of the segment being cast plus one or two inches to seal around the match cast joint. The side forms can be folded back away from the segment to permit removing the segment. This is done either with screw jacks or hydraulic rams. The inside form is collapsible and rolls on rails to allow removal so the concrete box segment can be lifted vertically.

All of the geometry of the bridge (horizontal or vertical curves and super elevation or transitions) is cast in by adjusting the old segment. The forms are never adjusted for geometry. Therefore, once a segment is in the old segment position its attitude is adjusted by screw jacks between the carriage and the soffit to provide the proper bridge geometry. A prefabricated reinforcing bar cage is then set in the new segment position. Once the side forms are closed and the inside form is rolled forward, the casting machine is ready for casting the new segment.

The designer of the bridge provides information to develop a theoretical casting curve—a curve along which the segments should be cast so the final desired alignment will be achieved after all deformation. The computation of these deformations is quite intricate and a good computer program is needed for maximum accuracy.

The short line system offers some advantages. The space required for setup is minimal, resulting in a centralized operation. Any geometry desired can be obtained by twisting the position of the cast-against segment. The primary disadvantage is the accuracy at which the cast-against segment must be set. Also, the casting machine must be flexible enough to conform to the twisted cast-against segment but rigid enough to adequately support the loads. This is particularly true when casting segments for a super elevation transition.

### **b. Long line system**

The long line system is similar except that a continuous soffit the length of a cantilever is built. All the segments are cast in their correct relative positions with the side forms moving down the

line as each segment is cast. Geometry control is established by adjusting the side forms and soffit. Variable depth structures may be cast by varying the elevation of the soffit: curves are cast by curving the soffit.

It is easy to set up a long line and to maintain control of the segments as they are cast. Also, the strength of the concrete is not as critical in a long line system because the segments do not have to be moved immediately.

### **c. Cast-in-place segments**

A third alternative is to cast the segments in their final position on the structure. Several projects have been constructed in this manner. Cast-in-place construction is advantageous for large, very heavy segments. Instead of handling the segments, only materials have to be transported, thus influencing type and size of required equipment. It is common to cast segments in place by using form travelers. Form travelers are movable forms supported by steel cantilever trusses attached to previously completed segments. The forms themselves may be constructed of either wood or steel. If balanced-cantilever erection (to be discussed) is used, a minimum of two form travelers is required.

The production rate with form travelers is nominally one segment every five days per traveler. Therefore, to approach the common precast segment production rate previously discussed, at least four travelers are required. Using concrete admixtures and partial post-tensioning can reduce the cycle time, but a practical minimum is probably three days per traveler.

Alignment variations and corrections are more easily accommodated in cast-in-place construction; but more correction will probably be necessary. This is related to the earlier age of the cast-in-place concrete when loaded, which causes more deformation from creep and shrinkage, [19].

### 2.3.2. Assessment of Balanced Cantilever Method

#### Balanced Cantilever segmental Bridges construction

##### Overview

Balanced cantilever segmental construction for concrete box-girder bridges has long been recognized as one of the most efficient methods of building bridges without the need for false work. [19]

Balanced cantilever construction denotes building a bridge superstructure from both sides of the pier table in a scales-like fashion. This erection method is also known under the name free cantilever construction. The pier table element, serving as a base from which cantilevering is begun, is usually between 6 and 12 m long. In order to balance the weight of both arms of the cantilever superstructure the segments will be about equally placed at both ends. Actual placement of new segments will hardly proceed exactly at the same times. Therefore the pier can undergo overturning bending moments and needs to be designed accordingly. Temporary towers with vertical prestressing or counterweights can provide additional support. Figure 4-1 schematically shows a typical construction stage in Balanced Cantilever Construction, [20].

As segments are attached to the cantilever ends one at a time, an overturning moment is created and must be resisted. This moment may be resisted by post-tensioning the pier segment down to the pier stem, providing temporary supports on either side of the pier or stabilizing the cantilevers with the erection equipment. The final choice belongs to the contractor but the designer must assume and detail a method of stress evaluation and parameters for the contractor. [19]

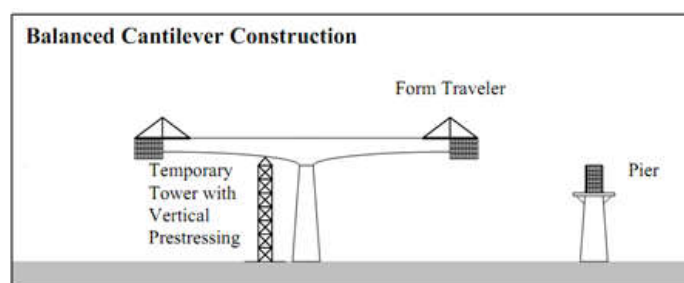
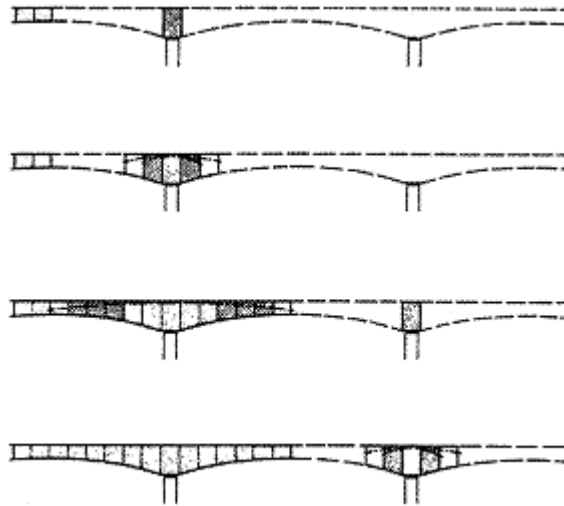


Figure 2.9: Balanced Cantilever Construction

This method has great advantages over other forms of construction in urban areas where temporary shoring would disrupt traffic and services below, in deep gorges, and over waterways where falsework would not only be expensive but also a hazard. Construction commences from the permanent piers and proceeds in a “balanced” manner to midspan (see Figure 2.10).



**Figure 2.10:** Balanced Cantilever Construction

A final closure joint connects cantilevers from adjacent piers. The structure is hence self-supporting at all stages. Nominal out-of-balance forces due to loads on the cantilever can be resisted by several methods where any temporary equipment is reusable from pier to pier.

The most common methods are as follows:

- Monolithic connection to the pier if one is present for the final structure;
- Permanent, if present or temporary double bearings and vertical temporary post-tensioning;
- A simple prop/tie down to the permanent pile cap;
- A prop against an overhead gantry if one is mobilized for placing segments or supporting formwork.

The cantilevers are usually constructed in 3- to 6-m-long segments. These segments may be cast in place or precast in a nearby purpose-built yard, transported to the specific piers by land, water,

or on the completed viaduct, and erected into place. Both methods have merit depending on the specific application, [22].

For cast-in-place balanced cantilevering a set of two form travelers is required, one for each arm of the cantilever. For multi-span bridges the form travelers can be dismantled after finishing cantilevering from one pier and can be set up for new use on the next cantilever.

In case of a bridge with variable box girder depth the pier table segment will be the most massive segment of the superstructure. This segment needs to be constructed prior to cantilevering to provide a working platform from which the two form travelers can start. It also includes diaphragms that facilitate the flow of forces from the cantilever arms into the piers. Because of size, geometry, and construction separate from the rest of the superstructure the pier table segment will take a considerable amount of time to construct. It can be put into place either with large precast segments or as cast-in-place with formwork mounted on the pier shaft, [23]

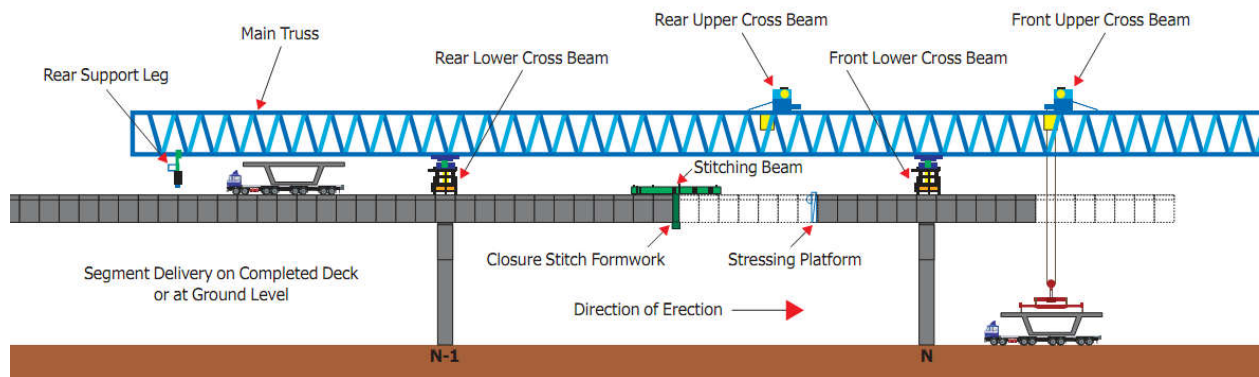
There are several ways of delivering the segments to the ends of the cantilevers. The most economical and probably most common method in the United States is to lift the segments with cranes. Sometimes barge cranes are convenient. Launching gantries come in all sizes and shapes. Some are equipped with two lifting devices enabling simultaneous attachment of segments, minimizing required overturning moment provisions.

Launching gantries are particularly advantageous when accessibility to the area beneath the structure is restricted by environmental consideration. By delivering segments across previously completed portions of the bridge, access to the area beneath is not required except to build the substructure. New bridges can be erected over existing traffic or buildings with minimal disturbance—a real advantage in urban areas. Launching gantries can be used to erect curved bridges as well as straight ones. Accessibility to the structure is generally the overriding consideration for balanced cantilever erection with a launching gantry, [19].

### **2.3.2.1. Balanced Cantilever Construction with launching gantries**

Features and advantage

- Delivery of segments along completed deck to rear of gantry minimises disruption to existing traffic networks.
- Necessary temporary works require little ground improvement and are generally elevated, therefore causing minimal disruption to existing roads, structures or services.
- Support craneage is reduced due to temporary works being relocated by gantry.
- Clear, unobstructed access to all work fronts is provided within gantry system.
- Work can proceed on multiple work fronts within the gantry; i.e. pier segment erection; cantilever construction and closure pour construction.
- Temporary loads are introduced directly into piers.
- Fast rates of erection are [24]

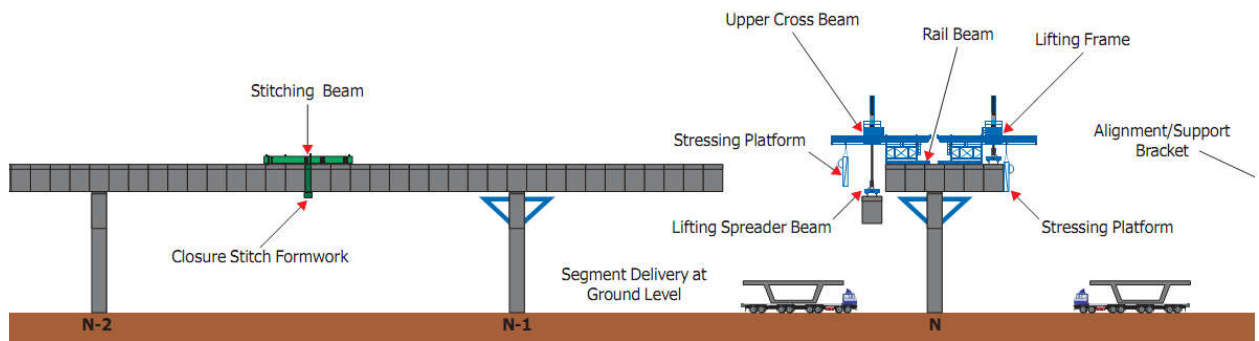


**Figure 2.11:** Balanced cantilever with launching gantries

#### a. Balanced Cantilever Construction with lifting frames

##### Features and advantage

- Relatively simple temporary works requirements.
- High rates of erection.
- Large segments can be erected.
- Optimized crew cycles.
- Multiple levels of segment alignment and adjustment are possible.
- Strand lifting units can be adopted and provide several levels of safety features.

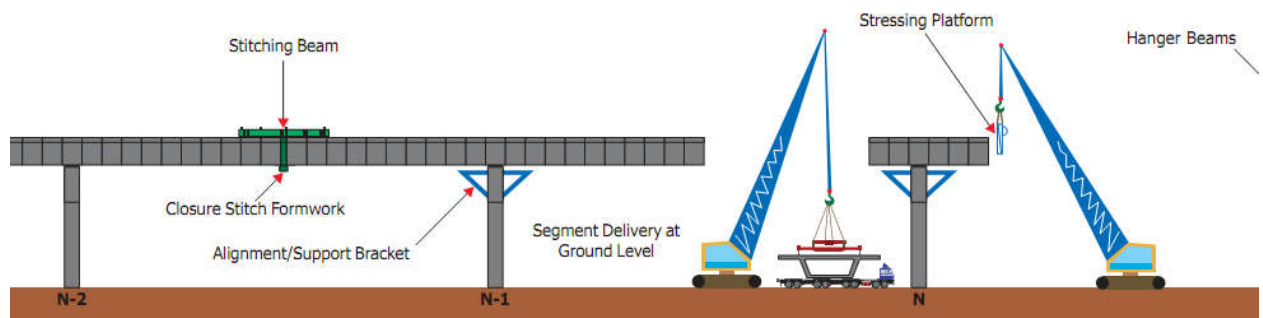


**Figure 2.12:** Balanced cantilever with lifting frames

### b. Balanced Cantilever Construction with crane

Features and advantage

- Temporary works requirements are minimized.
- Fast rates of erection are possible.
- Multiple work fronts are possible.
- Optimized crew size.
- Minimal engineering requirements. [24]



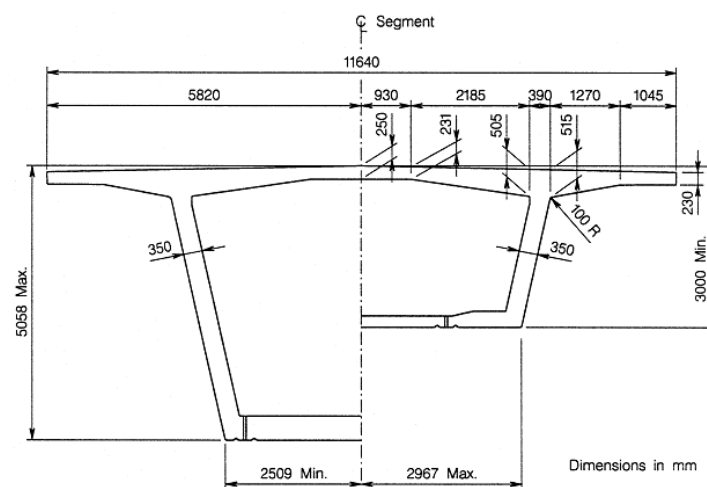
**Figure 2.13:** Balanced cantilever with crane

#### 2.3.2.2. Span Arrangement and Typical Cross Sections

Typical internal span-to-depth ratios for constant-depth girders are between 18 and 22. However, box girders shallower than 2 m in depth introduce practical difficulties for stressing operations

inside the box and girders shallower than 1.5 m become very difficult to form. This sets a minimum economical span for this type of construction of 25 to 30 m. Constant-depth girders deeper than 2.5 to 3.0 m are unusual and therefore for spans greater than 50 m consideration should be given to varying-depth girders through providing a curved soffit or haunches. For haunch lengths of 20 to 25% of the span from the pier, internal span-to-depth ratios of 18 at the pier and as little as 30 at midspan are normally used.

Single-cell box girders provide the most efficient section for casting – these days multicell boxes are rarely used in this method of construction. Inclined webs improve aesthetics but introduce added difficulties in formwork when used in combination with varying-depth girders. The area of the bottom slab at the pier is determined by the modulus required to keep bottom fiber compressions below the allowable maximum at this location. In the case of internal tendons local haunches are used at the intersection of the bottom slab and the webs to provide sufficient space for accommodating the required number of tendon ducts at midspan. The distance between the webs at their intersection with the top slab is determined by achieving a reasonable balance between the moments at this node. Web thicknesses are determined largely by shear considerations with a minimum of 250 mm when no tendon ducts internal to the concrete are present and 300 mm in other cases. Figure 2.14 shows the typical dimensions of a varying-depth box girder.



**Figure 2.14:** Typical dimensions of a varying-depth box girder

The features of this method that provide significant advantages over the cast-in-place method, provided the initial investment in the required equipment is justified by the scale of the project, are immediately obvious and may be listed as follows:

- Casting the superstructure segments may be started at the beginning of the project and at the same time as the construction of the substructure. In fact, this is usually required since the speed of erection is much faster than production output of the casting yard and a stockpile of segments is necessary before erection begins.
- Rate of erection is usually 10 to 15 times the production achieved by the cast-in-place method. The time required for placing reinforcement and tendons and, most importantly, the waiting time for curing of the concrete is eliminated from the critical path.
- Segments are produced in an assembly-line factory environment, providing consistent rates of production and allowing superior quality control. The concrete of the segments is matured, and hence the effects of shrinkage and creep are minimized.

The success of this method relies heavily on accurate geometry control during match casting as the methods available for adjustments during erection offer small and uncertain results. The required levels of accuracy in surveying the segments match-cast against each other are higher than in other areas of civil engineering in order to assure acceptable tolerances at the tip of the cantilevers.

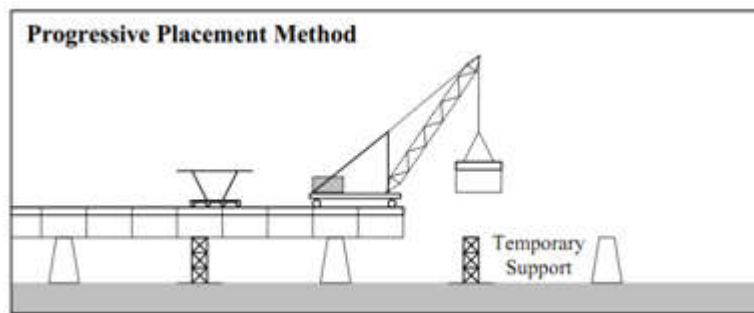
The size and weight of precast segments are limited by the capacity of transportation and placing equipment. A light template match-cast against the base of the pier segment allowed fast and accurate alignment control on the spans. [25]

An interesting pier design specifically feasible for cantilevering is that a pier consisting of transverse twin walls is advantageous as it provides stability for cantilevering but allows horizontal movement of the superstructure from thermal elongation through flexing of the wall panels. [26]

### 2.3.3. Assessment of Progressive Placing Method

#### Progressive Placement construction

The progressive placement method, in comparison with the balanced cantilevering method, is a one-directional process as shown in Figure 2.15. All cantilever segments are subsequently placed at the tip of a cantilever that is built across all spans. Both cast-in-place and precast segmental construction can be used. Often stay cables from the tip of a temporary tower on the superstructure support the cantilever. With growing cantilever superstructure this support mechanism has to be advanced.



**Figure 2.15:** Progressive Placement Method

Progressive placement has several advantages, as Mathivat (1983) points out. First of all, the placement process does not have to switch sides as it occurs in the balanced cantilevering method. Thus process control is simplified. In addition to this, good access to the placement location is given on the already completed part of the bridge superstructure. With the progressive placement method horizontal curves can easily be accommodated. From a structural point of view the progressive placement method is advantageous in substructure design. Only vertical forces from the dead load of the superstructure under construction are experienced. In comparison with incremental launching and balanced cantilevering, a simpler flow of forces takes place between superstructure and the piers. No horizontal forces are introduced in the piers and no unbalanced bending moments have to be withstood by the piers. It is therefore possible to immediately install the permanent bearings (Mathivat 1983).

Some disadvantages of the progressive placement method need to be dealt with during design and construction. As construction only progresses at the tip of one cantilever, progress is slower

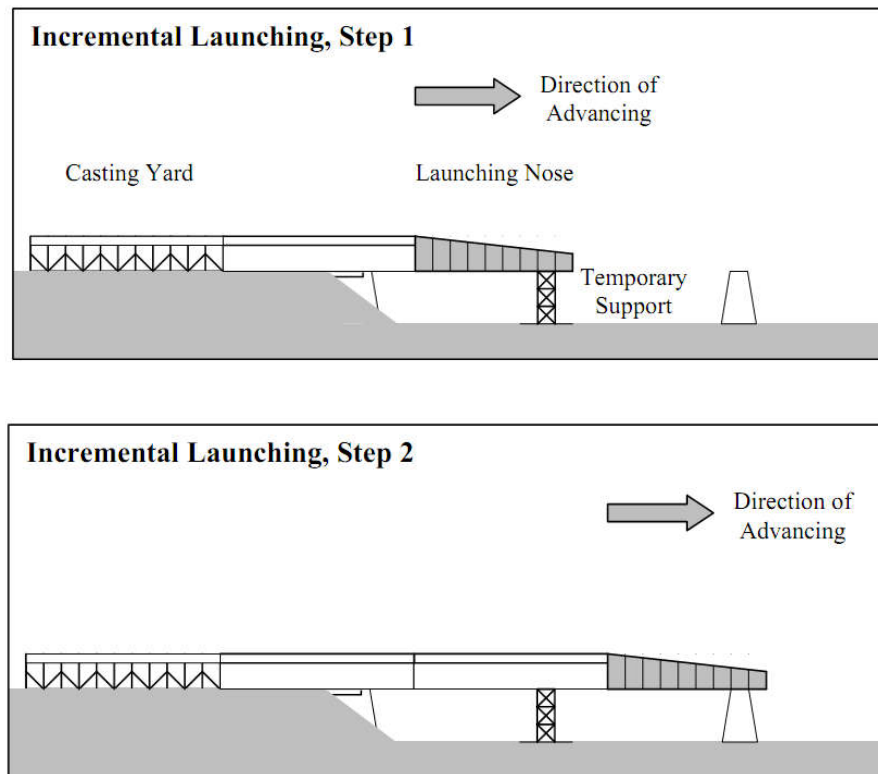
than in balanced cantilevering. Progressive placement resembles incremental launching in that the superstructure undergoes stresses very different from the permanent service conditions, including even stress reversals. In both cases the structure needs to incorporate temporary prestressing tendons to account for these stresses. Mathivat (1983) also points at the difficulty in erecting the first span with progressive placement. Other construction methods may have to be employed for this stage. [26]

#### **2.3.4. Assessment of Incremental Launching Method**

##### **Incremental launching method**

Incremental launching was developed by the German engineers Fritz Leonhardt and Willi Baur for the Rio Caroní Bridge in Venezuela (Podolny and Muller 1982), which was built from 1962 to 1964. The incremental launching technique, as opposed to other methods, consists of casting a continuous chain of segments at one particular location on site and then pushing the growing superstructure out over site to be bridged. A casting bed with adjustable formwork for the superstructure segments is set up. This casting bed can also be enclosed in a heated tent so that controlled casting and curing conditions are achieved. The normal cycle time, regardless of segment length is one week. Segment lengths according to Liebenberg (1992) typically range between 15 and 30 m.

Two different techniques for launching the bridge superstructure from the casting bed exist. Hydraulic jacks can pull the superstructure with steel rods, as it was done for the Rio Caroní Bridge (Podolny and Muller 1982). The second, more common method is to employ a pair of hydraulic jacks acting vertically and horizontally. Continuous repetition of lifting the superstructure off the abutment and then pushing it forward as far as the jack allows will achieve the launching in incremental steps. Figure 2.16,1-2 shows the launching process schematically. Podolny and Muller (1982) caution to design the jack capacity for more than the usual friction coefficient of 2 % because of imperfections that can occur during construction. Fig 2.16: Incremental Launching construction



**Figure 2.16:** Incremental Launching construction

In front of the cantilevering superstructure a lightweight steel launching nose is attached with tendons that reach the next support before the bridge superstructure itself arrives. Its purpose is to keep the bending moments in the superstructure smaller. Mostly the launching nose has a length of about 60 % of the bridge spans (Podolny and Muller 1982). Another way of reducing the bending moments is to implement temporary towers between the bridge piers. These towers need to be able to take the horizontal forces that arise from launching.

On top of all supports, including abutments, piers, and temporary towers temporary sliding bearings are installed during construction that will later be replaced with the permanent ones. Stainless steel plates are installed on the bearings. While the superstructure is advanced, Neoprene pads coated with Teflon and reinforced with steel plates are inserted between concrete and steel to reduce friction (Liebenberg 1992). Very low friction coefficients of 2% or less can be achieved with this method. Several advantages make incremental launching a very competitive erection method.

As with any cantilevering method it leaves the site below completely unobstructed during construction. Only for very long spans temporary towers or cable stays from above as supports are needed and possibly increased worker safety. Except for these the equipment necessary is reduced to the jacking mechanism, the adjustable stationary casting bed, and temporary sliding bearings, all of which may possibly be reused, which reduces the capital investment considerably. Podolny and Muller (1982) furthermore mention the cost savings due to avoidance of segment transportation and heavy construction equipment. They also point at less maintenance cost due to the higher prestressing of the superstructure. The controlled casting and curing conditions allow steady and quick construction progress.

Bridges that are erected with the incremental launching method should have a constant cross-section, especially in depth, and have a straight superstructure. It is possible to accommodate small variations in alignment and horizontal and vertical curvatures provided that they have a constant radius. Close control of the bridge geometry during casting and launching is very important. Sloping grades at the bridge site are also accommodated, in this case “the launch is usually in the downward direction”, more than 2 % slope would require a retarding mechanism to stop the movement of the superstructure.

Liebenberg (1992, p164) also gives a very clear statement of the main difficulty of the incremental launching method: “During launching, the section undergoes complete stress reversals as it progresses from a cantilever to the first support and thereafter over the following spans to its final position.” Clearly, this erection sequence generates a bending moment envelope in the structure depending on the span lengths that needs to be accounted for in designing the cross-section properties and the amount of reinforcement and prestressing tendons. The stresses due to the aforementioned high bending moments require much longitudinal prestressing both at top and bottom of the cross-section. Another disadvantage is the large workspace that is needed for the casting bed at the abutment and the adjacent storage areas.

Launching of the cured and post-tensioned segments required many personnel for supervision of all sliding bearings. An overall longitudinal slope of the bridge reduced the jacking forces necessary for launching. An overall curvature with radius 1,500 m in the horizontal plane and a constant 3.5 % cross slope of the bridge superstructure induced stresses due to restrained deformations during launching. Furthermore, the changing span lengths had to be considered in

coming up with the prestressing program to optimize the tendon profile and prestressing forces. [26]

### **Special Requirements**

There are two peculiarities associated with the technique, which must be appreciated by the designer. The first is that the alignment must be straight or, if it involves curves, the curvature must be constant. The second is that during launching, every section of the girder will be subjected to both the maximum and minimum moments of the span; and the leading cantilever portion will be subjected to slightly higher moments. This second constraint usually leads to slightly deeper sections, on the order of  $\frac{1}{15}$  the span, than would otherwise be considered. The girders must also be of constant depth as each section will at sometime be supported on the temporary bearings. Other considerations include the necessity for a large area behind the abutment for the casting operations, the requirement to lift the bridge off of the temporary bearings, and place it on the permanent ones when launching is complete and the need for very careful control of geometry during casting.

Incremental launching is generally considered for long viaducts with many spans of the same length. Spans up to 100 m can be considered; the requirement for constant-depth girders makes longer spans uneconomical. A single long span in the center of a project can be achieved by launching from both abutments and finishing at the long span with two converging cantilevers. The practical length limit for launching is about 1000 m. Bridges of twice this length can be considered by launching from both abutments. [27]

#### **2.3.4.1. Applicability and Limitations of Incremental Launching**

The use of the incremental launching method for bridge construction will never be the most efficient way to construct every single bridge. However, in the right location, the ability to erect the bridge superstructure without the need to intrude into either congested, restricted or environmentally sensitive areas beneath the bridge offers tremendous benefits to the owner, contractor and other stakeholders including:

- Minimal disturbance to surrounding area;
- Smaller, but more concentrated area required for erection; and

- Increased worker safety since all erection work is performed at a lower elevation

During the launching of a bridge, the superstructure acts as a continuous beam supported on roller or sliding bearings and is transversely restrained by lateral guides that prevent drifting movement. Any constraint eccentricity (vertical misplacement of launching bearings or transverse misalignment of lateral guides) will cause unintended secondary stresses and may cause launching problems such as excessive wear of bearing devices [28].

### 2.3.5. Assessment of Span-by-Span Method

#### Span-by-Span Construction

##### General

As with balanced cantilever and progressive placement, span-by-span construction activity is performed primarily at the deck level and typically implemented for long viaducts having numerous, but relatively short spans, e.g., <50 m. It was initially developed as a cast-in-place method of construction, on formwork, with construction joints at joint of contraflexure. The form traveler is supported either on the bridge piers, on the edge of the previously erected span and the next pier or, at times, even at the ground level. [29]

Segments can either span between supports, cantilever balance points or points of inflection. In the latter case, hinges are usually inserted at these points to enforce zero bending moments here. A relatively heavy supporting truss is necessary for the casting process.

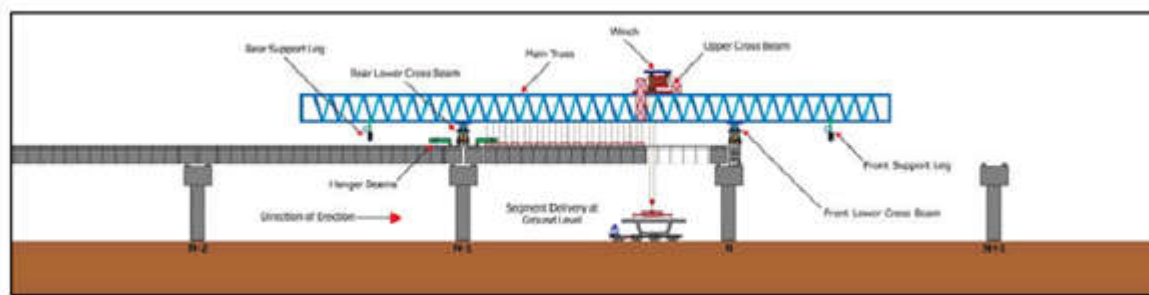
With the precast segmental method, segments are placed and adjusted on a steel erection girder spanning from pier to pier, then post-tensioned together in one operation. Although both the cast-in place and the precast span-by-span construction methods continue to be used, precast segmental has become the method of choice for most applications [29].

In the span-by-span method, supporting system is used to place all the segments of a given span during construction. This structure can be a truss fixed at the columns that supports the segments (in case of precast construction) or formwork (in case of cast-in-place construction) from below, or it can be a truss that holds the segments or formwork from

above the deck. Since this method uses the truss, the length of the spans of the bridge to be constructed is limited to the length of the truss.

The method used to erect precast units for any of the previous methods will depend on the site conditions. They can be lifted and put in place by using a crane on the ground or a barge. They can be lifted using equipment on the deck of previously erected units, or, in the case of erection using the span-by-span method, they can be placed using a launching gantry [18].

Lay out the characteristics of the span-by-span erection method as assembling all segments for a span in a set, which is “then aligned, jointed, and longitudinally post-tensioned together to make a complete span.” The principle of span-by-span erection is shown in Fig2.17



**Figure 2.17:** Principle of span-by-span erection

The actual construction can have several variants, the segments can be assembled on the ground and lifted in place as a group by a heavy-duty crane or they can all be put into their final position on erection girders along the spans to be completed. The second method was used for constructing different Viaduct projects. In this project different types of erection girders were used.

Erection girders were supported at their ends “by steel false work resting on the footings at each pier. After completion of a span the erection girders were set forward to the next span and adjusted. By then, the precast segments for this span had already been supplied and would be lifted in place by crane. Fine adjustments of the segments on the erection girders were possible by means of variable individual supports. Finally, post-tensioning would be performed to link all the segments together to form a complete span. The structure became self-supporting after casting of the closure joints with the pier table segments had been done and the longitudinal prestressing force was induced. Erection girder need not rest on the ground, but can also be

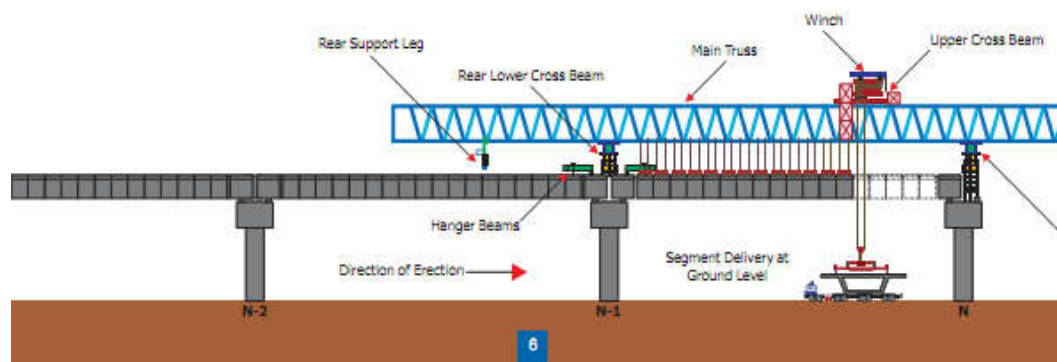
supported by already existing substructure or superstructure, e.g. the piers of a span that is to be constructed. Project specific design of substructure and superstructure and considerations as e.g. for traffic clearances set the boundaries for erection with erection girders. [29]

### 2.3.5.1. Span by span erection with launching gantries

The launching gantry can be supported by the completed piers, by the completed parts of the superstructure, or by both. They are very expensive and are only used economically in large projects. They are very useful when the area under the superstructure cannot be used for the delivering of precast units or for operating erection equipment. [18]

#### Features and Advantage

- Flexibility to use overhead or under slung gantries.
- Fast rates of erection are possible due to use of external post-tensioning
- Segment delivery is possible along completed deck to rear of gantry or at ground level.
- Smaller crew size is required compared to balanced cantilever construction.
- Good access provided within the gantry to all work fronts.[24]



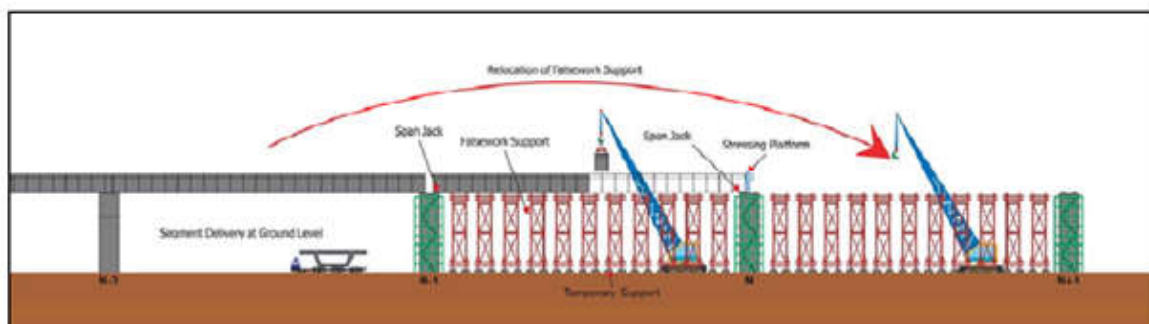


**Figure 2.18:** Lifting completed span of the Seven Mile Bridge, Florida, Using an overhead truss.

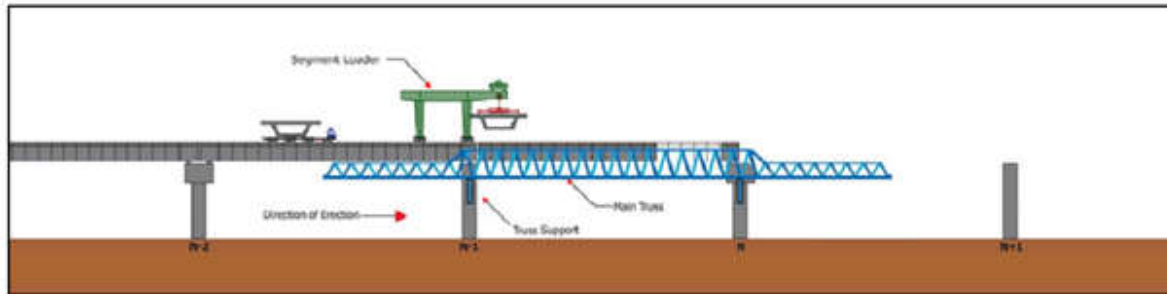
Span by span, as used today, utilizes post-tensioning tendons outside the concrete, but inside the box girder for ease of precasting and speed of installation together with dry joints, no epoxy, between segments. The post-tensioning tendons are continuous from pier segment to pier segment. [29]

### 2.3.5.2. Available Methods of Construction for the Span-by-Span Segmental Alternative

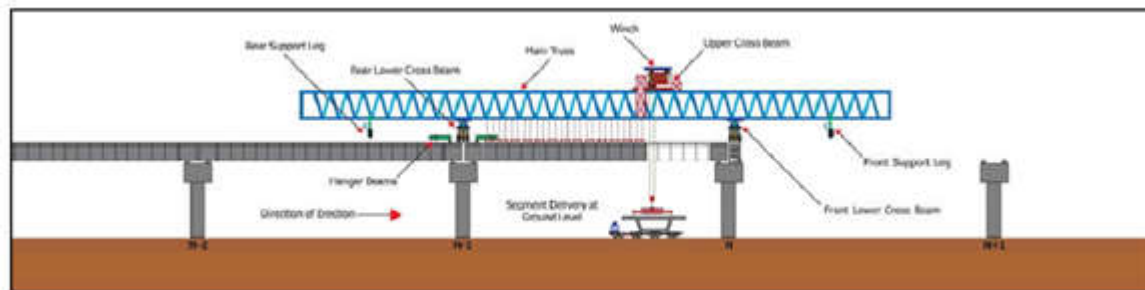
There are several methods available for span-by-span construction. The segments may be supported on temporary shoring, under-slung trusses, or from over-head gantries; before they are post-tensioned together to form a self supporting unit. Figures 2.19 through 2.21 illustrate these basic systems.



**Figure 2.19:** Temporary Shoring



**Figure 2.20:** Self-launching Under-slung Trusses



**Figure 2.21:** Overhead gantries

#### **a. Temporary Shoring Method**

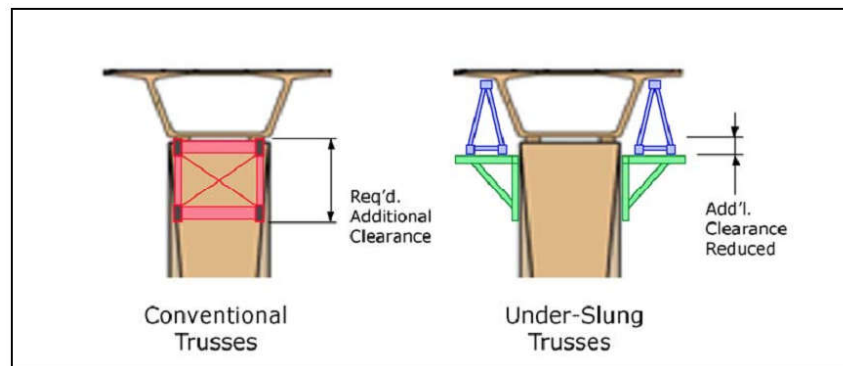
This is one of the most cost effective methods for erecting span-by-span if the conditions are right. The shoring is typically of standardized modules for economical fabrication and repeated usage over the length of the project. This is why this method is most suitable for low and level viaduct structures over grounds that are relatively open and level. Frequent crane movement is required for relocating the shoring, which makes the method more effective in an open field. Also the relatively narrow “pathway” between the existing and new superstructures would also make crane movements difficult if not impossible. For these reasons, the temporary shoring method is not considered to be a likely choice of construction method for this project.

#### **b. Under-Slung Trusses**

Under-slung trusses are typically supported on brackets mounted at the piers or temporary bents supported off the permanent footings as shown below. The trusses can be designed to be self-launching and equipped with launching noses or to be short and light-weight for relocating with the aid of a crane. The under-slung trusses depicted in Figures below support the superstructure segments near the juncture where the flange overhangs meet the webs; this type of trusses

reduces potential encroachment into traffic windows of crossing facilities beneath. The use of the longer self-launching trusses is a possibility, because the lowering of the “trapped” truss will be infrequent which makes it worth considering.

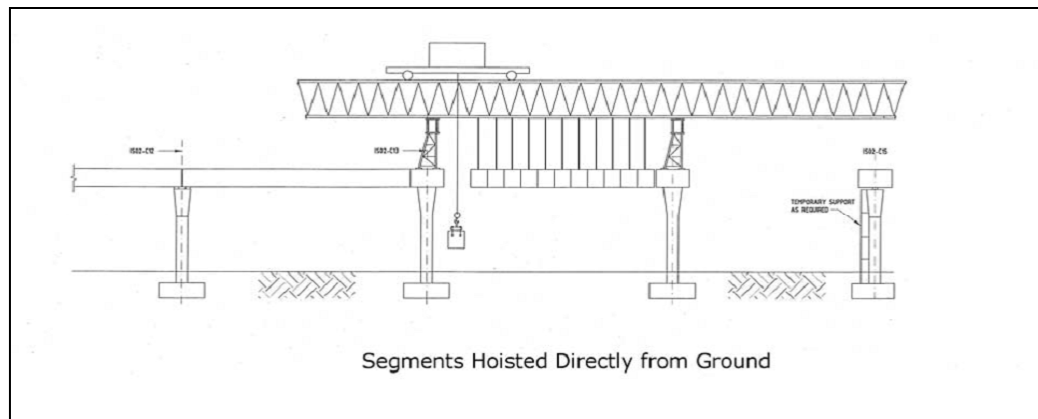
Self-launching under-slung trusses are often outfitted with a segment loader that will take segments that are fed to the work front via the completed portion of the structure and place them onto the trusses. Since there is water access on the Newtown Creek, erection of the precast segment can begin from the water side of the approaches and thus eliminate most if not all intermediate crane movements. In view of the relative economy of this method, the under-slung trusses should not be ruled out at this time.



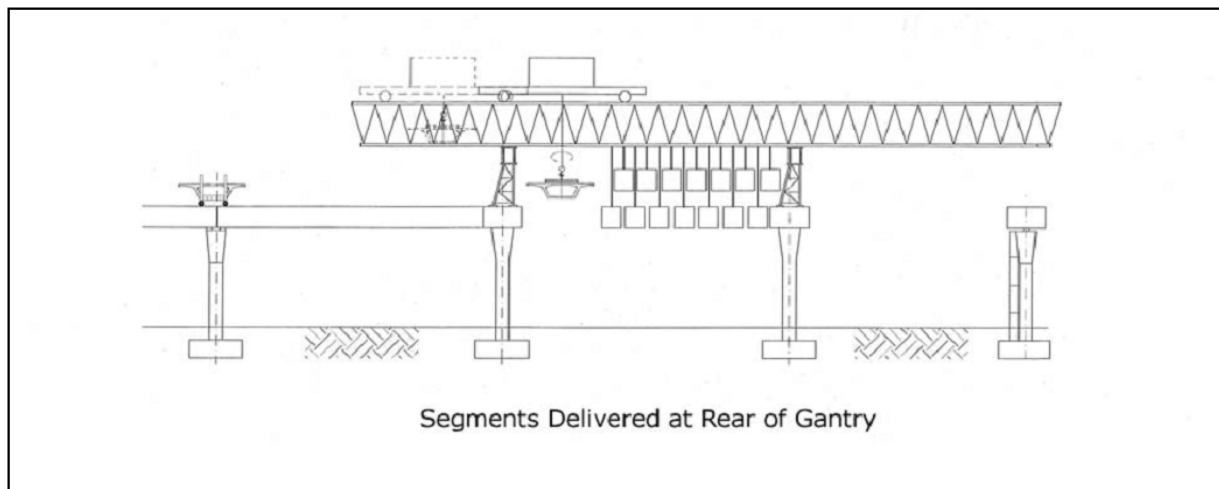
**Figure 2.22:** self-launching under-slung trusses

### c. Overhead Gantry

In comparison to the under-slung trusses, the overhead gantry is slightly more elaborate in design and in its operations. The size and configuration of the gantry does vary according to the constraints of each project. The cycle time for erecting a span with an overhead gantry is slightly longer than that needed using under-slung trusses. However, there are many factors that drive the overall cost and schedule and, historically, overhead gantries have been used on long approaches in spite of the marginally longer cycle time. A noteworthy feature of the overhead gantry is that the trolley and winch, typically outfitted on an overhead gantry, will replace the crane. Segments can either be delivered on the ground to one point of pick underneath the gantry (Figure 2.23) or to the rear of the gantry if ground delivery is not feasible for any reason (Figure 2.24). Ground delivery will of course require much less effort but is less likely for this project due to the congested site



**Figure 2.23:** Ground delivery of span-by-span construction



**Figure 2.24:** Rear delivery of span-by-span construction

In many cases, the critical path is likely to fall on the main span structure and not on the erection of the approach superstructures. The deciding factor on the type of equipment for erecting precast segments is expected to be the relative costs of regional field labor vs. greater investment in more efficient equipment such as the overhead gantry. Although the cycle time is marginally greater for the overhead gantry, the relative ease of advancing the equipment from span to span, and especially for the overhead gantry to laterally relocate from one line of girder to an adjacent line by lateral launching (or sometimes called side launching) would make the overhead gantry the logical first choice for considering in the design process.

As design of all elements (including the connectors and ramps) of the project advances, a more complete assessment of durations and logical sequence of events can be confirmed. At this time, and looking strictly in terms of construction durations for the approaches and the main spans, there seems to be ample time to use just one erection gantry to complete the approach spans for each of the two contracts that involve the approaches (or the entire project if there were only one contract) If the entire approach were to be built at one time. [29]

### 3. THE ADDIS ABABA LRT PROJECT

#### 3.1. Overview of Proposed Design Practice of light Rail Transit Project

##### 3.1.1. Description of Feasibility Study

###### Project Rout description

East-West (E-W) route has the total length of 17400m, of which 2730m elevated section. Each of the elevated stations has the total length of 340m. And the total elevated structure has the length of 3070m, which takes 17.644% of the whole route. While the N-S route has the total length of 14170m (2800 meters of shared section are not included), 2345m of which are elevated road section. All the elevated stations have the total length of 240m. And the total elevated structure has the length of 2585m, which takes 18.243% of the whole route.

###### Principles for Design

The construction of elevated structure should take the impact on city transportation into consideration and try to reduce the impact. When the elevated structure crosses critical sections (junctions), it should conform to the requirement of grade transportation and choose proper construction method and structure.

The elevated structure should adopt sound insulation and sound absorption measures in terms of shape, material and setting. And the elevated structure and tracks should reduce the impact from vibration and noise in order to reduce Vibration and noise.

The design of elevated structure should consider about grad and underground existing buildings, pipes and wires and try to reduce negative the impact on buildings and pipes and wires.

The elevated structure should not only meet the requirement of marshalling, but also consider about the electric supporting equipment, communication supporting system and anti beam falling system.

When the elevated structure crosses road, the clearance under the elevated bridge should meet the requirement of marshalling etc.

According to the requirements of different section geologic situation and master plan of the city and integrated with existing buildings, pipes, wires and transportation situation, reasonable construction method and structure should be adopted after comprehensive comparison among different technical aspect, economical aspect, construction techniques, environmental protections and functions.

Road and Bridge Altitude Division: Generally the road and bridge altitude division is no more than 3 meters. All are suitable for typical simply-supported spans of 30m. Alternatively the beams can be continuous and post-tensioned to form a series of 2-span structures.

The guideway beam can be provided with side walls which perform the function of vehicle retention and acoustic barrier. Included on the guideway are the concrete slab trackwork, catenary supports, cable conduits, train control, communication cables and service walkways.

The use of slab track and light rail vehicle loads can result in more slender and lighter guideway beams than on a conventional heavy railway structure.

At-grade tracks include small cut or fill sections up to 2 meters along which no retaining wall structures are needed. All at-grade sections will have track on ballast with concrete mono block ties. The elevated guideway and tunnel sections will use direct track fixation.

### **3.1.1.1. Bridge Structure and Shape**

#### **a. Beam Selection**

Since span bridge is the main body of elevated bridge, economical and good-shaped beam span structure is of great importance. The design of beam structure analyzes prestressed concrete T beam, prestressed compound I beam, prestressed concrete combined box-beam, prestressed concrete box-beam, prestressed concrete fish-bellied beam, U-shaped beam, steel beam and concrete plate beam, and compare different construction methods, such as precast of the whole arch erection, longitudinal erection of precast segment, the in situ supporting frame of bridge sites, the in situ sliding mould base integrated with the site situations and construction schedule plan in economical and technical aspects.

Box-beam can suit with different conditions, and be one of widely used elevated structures. It has closed section, high distortion resistance good dynamic stability and its

structure has a good bearing capacity. Its shape is concise and has a good adaptation; therefore it can be applied in straight road section, curve road section, reversing line and transition line. T-shaped beam has a good bending resistance and a light weight, and is convenient for prefabricate ceiling. But multi-layered T-shaped beam has to connect transverse shelf board with bridge surface as a whole, or even use transverse prestress. It is not as concise as box-beam viewing under the bridge, and the construction for transverse connection on the site needs a great workload. U-shaped beam is one of through beams. Its greatest advantage is that the construction distance from track top to beam top structure is comparatively short. And the both sides of its main beam has sound insulation function. But the beam body is a little bit higher from side views.

Furthermore it is not reasonable that concrete mainly locates in the tensile area. In general box-beam should be mainly adopted.

#### **b. Beam Span Alternatives**

The average height of the viaduct in this line is 7.5 meters, which belongs to low bridges. If the span is too small, the bridge must need too many piers and the image would be destroyed. Thus the span should not be too small. The analysis indicates that it is the most efficient scheme to adopt the ratio 1:4 between clear height under the bridge and clear width. In terms of image effect the beam span should be 30 meters. Moreover the economic analysis also indicates that the beam span of 30 meters is an economical span.

#### **c. Beam Height Alternatives**

It is not good for image design if the beam is too high. But a low beam would result in small strength of the beam, which cannot meet the requirement from LRT operation to bridge residual deformation limit. Integrated with beam bearing situation and the strength requirement from LRT operation this design adopts four beam heights as alternatives, which are 1.7m, 1.8m, 1.9m, 2.0m (3.1)

After analysis it can be concluded that the beam height of 1.8 meters is the best one. Therefore this project adopts the beam height of 1.8 meters. Major Technical Parameters of All Beam Heights for 30 Meters Double Tracks (Table 3.1)

Beam Height (m)	1.7	1.8	1.9	2
Strength Safety-Coefficient	2.01	2.02	2.02	2.01
Anti-Cracking Coefficient of Safety	1.4	1.42	1.44	1.45
The Residual Camber Three Years Later(mm)	4	3.9	3.8	3.3
The down deflection span ratio under static live load (stiffness)	1/3458	1/3951	1/4471	1/5024

**Table 3.1:** Major Technical Parameters of All Beam Heights for 30 Meters Double Tracks

#### **d. Simple-Supported Beam and Continuous Beam**

The bridge beam of viaducts in completed city railway system mainly adopts in situ simple-supported beam and in situ continuous beam. Compared to simple-supported beam, continuous beam has the advantages of integrated structure, comfortable ride, good stability and nice shape. The costs for both beams are similar. After the analysis about different beams and the current usage of viaduct beams in railway transportation, continuous in situ continuous box-beam is recommended for this project.

A detailed set of Structural Design Criteria will be developed after the LRT vehicle has been selected.

#### **3.1.1.2. Major Construction Method**

As the majority of the LRT route is located adjacent to congested highways, it is not considered that cast-in-place construction will be practical. Precast construction, using simply-supported beams, incremental launching or segmental construction, is considered to be more appropriate.

Major Beam Construction scheme:

##### **a. Integral cast in-situ scheme**

The design of this project would be affected by many factors (existing roads, planned street net, the rolling stock entrance and exit and turnout). There are many types for beam structure, bridge arch span as well as width. The scheme of in-situ has the advantages as

follows: In-situ construction can well suit with different requirements of span length and width. The in-situ construction for main beam can well control the construction quality and conveniently control the positions of different components (such as prestressing tendon on curve beam) on the beams.

The construction of full space support has the disadvantages as follows: the foundation has to be treated in order to prevent large-scale settlement. The measures of foundation changing and filling, buttress setting and preloading should be taken.

#### **b. The erection of precast segment scheme**

This scheme adopts the erection of precast segment. The transportation for hoisting is not difficult and hoisting technique is mature, which facilitates the large-scaled manufacture of bridge beam precast. But a second tension is needed for the construction when segments form whole arch. This construction has many procedures with a higher cost.

#### **c. Hoisting of precast of the whole arch**

This scheme adopts hoisting of precast of the whole arch. Although the process for construction is fast (one arch erection can be completed within one day on average.) And this construction is also favorable to environment (fence is not necessary). But large-scaled beam fabricating yard, beam storing yard, beam transportation access line, large-scaled hoisting equipments (each arch of the beam is over 400 tons both lines) as well as large beam transportation trolleys are needed. The beam transportation and beam hoisting are very difficult. Based on the analysis mentioned above it is recommended to adopt the scheme of scaffold integral cast in-situ,[8].

### 3.1.2. Overview of Current Design Implementation of Addis Ababa light Rail Transit Project

#### Design Specification

This design relates to reference drawing for common span double-track juxtaposed simply-supported box girder of Addis Ababa E-W & N-S(phase I) Light Rail Transit Project of Ethiopian Railway Corporation. The reference drawing for girder was prepared based on the design concept of safety, reliability, economy, applicability, easy construction & erection and easy maintenance and repair and in light of the technical characteristics of the project. The drawing indicates the pre fabricated post-tensioned simply supported box girder (station girder) with a length of 20m ( $L=20m$ ).

#### Applicable Scope and environment

- a. Applicable scope: the drawing is applicable to Addis Ababa E-W & N-S (phase I) Light rail transit project of Ethiopian railway corporation and minimum radius of curve<sup>®</sup> is 200m.
- b. Design speed: the max speed (V) is 70Km/h
- c. Design reference period: the design service is 100 years in normal operation condition
- d. applicable environment: for ground structure without protection measure in general atmospheric condition ,the carbonized environment exposure class 1 and 2.corresponding measure should be taken in case of corrosive environment.
- e. Seismic intensity: the seismic ground motion peak acceleration is 0.1g.

#### Design load

##### a. Dead load

- i. The dead weight of structure should be based on the volume weight of concrete ( $26.5KN/m^3$ ).please refer to table3.2for detail of dead weight of main girder in this design.
- ii. Secondary dead load include loads of auxiliary such as the trackside equipment, cable, duct, railings, pipeline and the supporting device, cast in place wet joint of bridge deck

board, cross slabs, waterproof layers and paving layers of bridges deck etc.Refer to table3.2 or values of secondary dead load employed in this design.

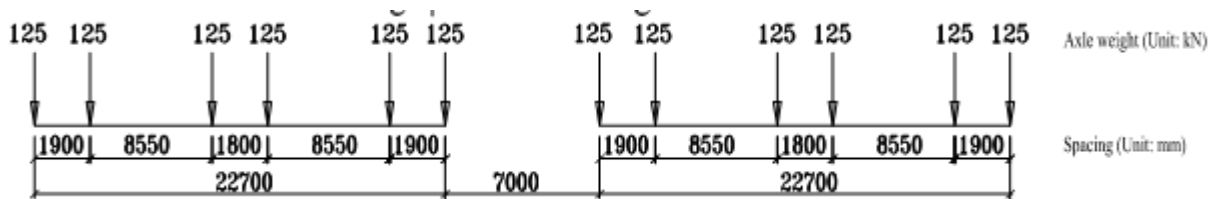
Category	Secondary Dead Load	Secondary Dead Load	Weight (t)
Single track(span)	25.5KN/m(Straight)	28.5KN/m(Curve)	98.1

**Table 3.2:**Dead weight of main girder

**b. Live load**

- i. Live load of train: loads of tram are shown in fig.....coefficient of dynamic force should be calculated based on the following:

$1+0.8*6*a/(30+L)$ ,where a is taken as 2 and L is the calculated span of girder.



**Figure 3.1:** Live load diagram

**c. Impact Load**

$I=19.2\%$

**d. Centrifugal force**

The centrifugal force on the bridge in curve equal to the vertical dead and live load of the train multiplying the eccentricity C.It acts horizontally outward onto the position 2.0m above the rail top. The eccentricity C should be calculated as per the following formula:

$C=v^2/(127R)$ ,where v is the design speed and R is the curve radius.

**e. Rail flexural force**

The stretching force and flexural force of long rail for jointless track should be calculated respectively based on the bridge position in the no breathing zone or breathing zone of the jointless track. For the breathing zone, only flexural force should be calculated.

**f. Additional force**

Braking force or tractive force: braking force and tractive force should be calculated based on 15% of vertical static live load or 10% of vertical static live load when they are calculated together with centrifugal force. For the section double-track bridge, the braking force or tractive force should be based on that of the single track; for the double-track bridge at an elevated station or with 100m of inbound and outbound tracks of station should be calculated based on 10% of the double track vertical static live load. In case of combination of the braking force or tractive force of double-track with other loads, it should be calculated respectively based on the same direction or reverse direction.

**g. Lateral swaying force:**

It should be calculated based on 15% of the total weight of 4 axles of adjacent two cars. It acts on the rail top in the form of concentrated force.

**h. Wind force:**

$$W=K1*K2*K3*W_0=1.3*1.0*1.0*1000Pa.$$

If advertising board is set, it should be calculated and checked separately based on the size of advertising board.

**i. Temperature load:**

Deformation and stress caused by even temperature difference and sunlight temperature difference should be calculated for bridge, culverts and their parts. Temperature difference should be determined according to local climate condition and building condition. Linear expansion coefficient (1/C) should be 0.0000118 for steel and 0.000010 for reinforced concrete. According

to the local environmental condition, overall temperature difference caused by temperature rising and falling concrete should be considered as  $\pm 20^{\circ}\text{C}$  for the system.

**j. Ballast less bridge deck:**

Ballast less bridge deck should be adopted and the clear width of bridge deck of single girder should be 3.47m. If the distance between centers of straight lines is 4m, the width of wet joint between two girders should be 0.2m. In case of curve girder, the clear width of bridge floor of two girders should be kept unvaried at 3.47m and the width of bridge deck should be adjusted by adjusting the width of wet joint of bridge deck board.

**k. Rail support:**

The longitudinal rail-support-stand short-sleeper-type monolithic track bed should be adopted for trackside equipment and 50Kg/m rail should be adopted. The design calculated height between rail top and girder top should be set to 500mm.

**l. Change in operation load:**

Where the operation load change, calculate and check should be carried out as per the actual load.

**Design parameters**

1. Special live load should be used to calculate the vertical deflection of girder body. The vertical deflection should be no more than  $1/2000$  of the calculated span.
2. Under the effect of swaying force of train, centrifugal force, wind force and temperature force, horizontal deflection of girder body should be less than or equal to  $1/4000$  of the calculated span of girder body.

**Structure:**

- I. In this design, post-tensioning prestressed concrete box girder are employed and one girder should be used each track. The two girders of the double track should be integrated in to one through wet joint of bridge deck board and cross slab connection.

- When located at curve and the distance between center of lines is more than 4m, this can be accomplished by adjusting wet joints of bridge deck board.
- II. The clear width of single girder bridge floor should be 3.47m
  - III. The calculated span of girder is 19.2m. The girder is 19.9m long in total and 1.5m high. The structure from the rail top to girder bottom should be 2.0m. For each pre-fabricated girder, its top width is 3.47m and width of lower edge is 1.7m.
  - IV. Each box girder should be provided with drainage slope on one side with a gradient of 2.0%. The drain pipe should be arranged downward. Additional drain pipe should be set at both ends to facilitate the slope drainage. In addition, continuous drip mold should be provided in the longitudinal direction at the lower edge outside bridge deck board so as to prevent water entry into girder body.[9]

### 3.1.3. Review of ADDIS ABABA LRT Project

#### 3.1.3.1. Evaluation on design and construction approaches of A.A LRT Bridge

##### Selection of beam shape and size

Shape refers to section geometry of the beam structure along the longitude and transverse direction. Currently the A.A LRT projects adopt adjacent single boxes section with constant depth along the span with a longitudinal cast-in-place concrete closure pour (wet joints).

The appearance and Visual impact of bridge construction in urban areas on the community must receive serious consideration. Moreover, knowing that Addis Ababa is becoming the host of many diplomatic and tourist destination, aesthetics of bridges design is a critical issue.

For a good design, there must be a rational balance between the overall width of the deck, and the width of the box as well as balance between overall width of deck and depth of the section along the span. Box sections provides higher structural efficiency but suffer from a certain blandness of appearance; the observer does not know whether the box is made of an assemblage of thin plates, or is solid concrete. Also, the large flat surfaces of concrete tend to show up any defects in the finish and any changes in color. Designs should consider these problems and do what can be done within the constraints of the project budget to alleviate them. Side cantilevers have an important effect on the appearance of the box. The thickness of the cantilever root and the shadow cast on the web mask the true depth of the deck. If the deck is of variable depth, the perceived variation will be accentuated by these two effects. In general, the cantilever should be made as wide as possible, that is some seven to eight times the depth of the root.

The visual impact of the depth of the box is reduced if it has a trapezoidal cross section. This inclination of the web makes it appear darker than a vertical surface, an impression that is heightened if the edge parapet of the deck is vertical.

The depth may be varied continuously along the length of the beam, adopting a circular, parabolic, elliptical or Islamic profile, Figure 13.4. Alternatively, the deck may be hunched. The decision on the soffit profile closely links aesthetic and technical criteria. Clearly, this decision also has an aesthetic component. When the change in the depth of the box is not too great,

haunched decks are often chosen for the precast segmental form of construction, as they reduce the number of times the formwork must be adjusted, assisting in keeping to the all-important daily cycle of production. However, here again there is a conflict between the technical optimization of the shape of the beam and aesthetic considerations. The beginning of the haunch is potentially a critical design section, both for shear and bending. This criticality is relieved if the haunch extends to some 25–30 per cent of the span length. However, the appearance of the beam is considerably improved if the haunch length is limited to 20 per cent of the span or less.

A.A LRT bridges lack aesthetics due to the adopted size and shape of the girder beam.

The box girder is also the most flexible bridge deck form. It can cover a range of spans from 25 m up to the largest non-suspended concrete decks built; of the order of 300 m. Single box girders may also carry decks up to 30 m wide. For the longer span beams, beyond about 50 m, they are practically the only feasible deck section. The A.A LRT Bridge adopts 30m adjacent single boxes section with constant depth along the span with a longitudinal cast-in-place concrete closure pour (wet joints) which increase additional pier support, workload in fabrication, prestressing and erection of two separate beams which can be avoided by adopting a single-cell-wide-deck-long-span box section.

### **Over bridge track selection and track configuration**

Slab track over bridge section is the most efficient design approach. Moreover, box section bridges offer a greater advantage in providing the top deck slab as track structure. Slab track have different forms based on the structural interaction between the slabs, the tie/sleepers and the rail. This are mainly categorized in to two groups.

1. Discrete rail support slab track system: this system contains the slab track with the rail supported discretely over the sleepers or blocks.
2. Continuously rail support slab track system: this system contains the slab track with the rail supported continuously in the form of embedded the rail inside the concrete slab while casting the track and by clamping the rail continuously over the slab top.

Practically, continuously supported rail system has higher track stability, efficient track/support structure interaction, simplified stress analysis and easily constructible with reduced cost over the bridge section.

Addis Ababa LRT project adopt twin-block discrete rail support placed on top of the bridge deck which increase the complexity of the bridge track interaction and workload in casting the blocks. However, discrete rail support has found to be efficient in accommodating hunting for coming from the train wheel by sliding laterally. But on the long run, lateral displacement of the blocks will cause miss- alignment of the track and may lead to derailment of the vehicle. Hunting movement could be addressed by using rail fasteners which tolerate elastic lateral rail displacement.

### **Design load consideration**

Design loads for different load effects are considered for the project. However some loads which have significant effect on the structure are neglected, underestimated or overestimated. These loads are discussed as follows:

#### **a. Live load**

Live load considered in the project consists only operational vehicle load.i.e live load specified under design specification. However during operational stage there are expected vehicle load which differ in axel load/configuration from the main live load. These are maintenance vehicle which offer puling or pushing purpose during damage.Therefore,design should be computed based on any combination of this vehicles(operation and maintenance) which produce the most critical loading conditions for bending, axial, shear and tensional stresses, as well as deflections, fatigue, and stability shall be used for the design.

#### **b. Rolling force (RF)**

Rolling/rocking for is neglected in the design specification for LRT bridge.

### **c. Wind load**

#### **Wind load on structure**

Wind load considered on the project expected to act on the structure is underestimated. i.e  $W=1300\text{Pa}$ , However, local code [ERA Bridge Design Manual] suggests the wind load expected to act to structures build around urban areas of the country should be considered to  $2400\text{Pa}$  [ $2.4\text{KPa}$ ] assumed to act on the vertical projection of the structure applied at the center of gravity of the vertical projection in any horizontal direction.

#### **Wind load on live load**

Wind load acting on the moving load is neglected for the project which has significant effect on the overall stability of the bridge.

### **d. Derailment load (DL)**

Since derailment of moving vehicle for railway bridges have significant safety and serviceability issue than highway bridges, serious consideration of this load must be addressed in the design process. This consideration should have two forms. First, derailment load effect on the deck structure and derailment load effect on the railing structure.

On the design specification of the Addis Ababa LRT project, there is depiction of derailment load consideration for both deck and railing structure which leads raise concern on the safety and serviceability of the bridge.

### **e. Construction load**

#### **Construction approach**

##### **Prestressed Precast**

Prestressed Precast girder fabrication is one of widely accepted bridge construction approach nowadays. In generally prestressed precast production gives benefits of good quality control, avoids construction of unnecessary supporting structures [formwork], give better performance and economic section due to prestressing technique. Addis Ababa LRT Bridge construction

prestressed precast method with internal post-tensioning. This approach is the latest achievement of the project and new advancement to the country's bridge construction development.

### **Full Span Precast**

Addis Ababa LRT Bridge constructions adopt full beam precast approach. Full span precast fabrication, however, has its own drawback. These are:

1. limited Quality control over the full span: bulk concrete casting creates huge heat of hydration and significant temperature variation along the length of the beam span. In addition, slight movement of the casting formwork due to the weight of bulk fresh concrete during vibration and floating, creates distortion along the span and cross section. This limits the quality control of full span precast process.
2. Erection difficulty: due to the large weight of full span precast, erection process requires high caution and special construction equipment. This extra workload makes the erection process difficult.
3. Requirement of special equipment: due to large weight of full span precast requires special large weight lifting and moving equipments.
4. Limited to production of constant cross section: due to difficulties in preparing casting formworks to variable cross section, constant depth casting formworks are widely used.

However, bridge construction experience in recent time achieved improved approach to the full span precast fabrication. Segmental prestressed/external post tensioned/ precast over head span by span construction approach are the current state of art in bridge construction and suitable to the current and future light rail transit bridges build in our country with similar economic investment.

### **Girder Erection/Construction Method**

The Addis Ababa LRT Bridge construction adopts full precast span by span erection scheme. This method is comparatively preferable construction approach for full beam precast mainly in the simplicity of the technique.

## Analysis and Design Method

The scope of the research was to evaluate the analysis and design methods of the project. However, due to difficulties finding these documents from the concerned office, i.e. Ethiopian Railway Corporation (ERC), Chinese Railway Engineering Corporation (CREC) and Sweden consulting firm (Swesroad), the research cannot incorporate this section.

### Section Adequacy for Flexure, Shear and Shear of A.A LRT bridge section

#### Comparative Section Modulus Analysis (Appendix B&C)

Section Modulus(Z)	Research Result	LRT Project Result
Top Section modulus ( $Z_t$ )	<u><math>2.325 \times 10^5 \text{ mm}^3</math></u>	<u><math>6.37 \times 10^5 \text{ mm}^3</math></u>
Bottom Section modulus ( $Z_b$ )	<u><math>2.203 \times 10^5 \text{ mm}^3</math></u>	<u><math>5.58 \times 10^5 \text{ mm}^3</math></u>

According to the research finding, the required flexural section modulus for the project is lightly lesser than the actual used section.

However, the section shows over stressed under tensile stress when checked for permissible for working load condition. See Appendix C

#### 3.1.4. Proposed Guideline for Design and Construction of Light Rail Transit Bridges

**Design Philosophy:** AASHTO LRFD

**Design approach:** Precast-Prestressed-Segmental Box section bridge girder according to AASTHO LRFD Bridge Design Specification for Segmental Bridges

**Design Procedure:** AASTHO-PCI-ASBI for span by span segmental bridge design

**Form of structure:** Simply supported Single cell concrete box girder

**Span Length:** Maximum span utilized for Addis Ababa Light Rail Bridges is 30m and the practical capacity of box section is much beyond 30m. Therefore, this proposition shall use 40m

length span. This figure is selected for the simple reason to show that on every 3 consecutive span a single pier would be reduced if 40m was used.

**Span Configuration and Typical Section:** The structure is a single-span bridge with span length of 40m. The Bridge carries double track light rail traffic of standard gauge (1435mm) and center to center distance between tracks equal to 4m. In two direction. Expansion bearings are placed at all piers.

The typical section selected is the AASHTO-PCI-ASBI Segmental Box Girder Standard Type 2700-2, a single-cell concrete box girder with 8.4m wide deck and 1.8m depth. Cantilevered overhangs are 1.4m each. Minimum top slab thickness is 350mm. The thickness of the bottom slab is 400 for four segments on both sides of each pier and 250mm thick elsewhere. The thickness of the webs is 300mm”, which are sloped at 2.5:1.

The top slab can accommodate 12 tendons in each half of the box girder, for a total of 24 tendons in the top slab. The bottom slab can accommodate 6 tendons in each half of the box, for a total of 12 tendons in the bottom slab. Additional tendons may still be accommodated either in the top or bottom slab.

When dealing with development of a cross-section, it is important to investigate the efficiency of the proposed cross-section. The section efficiency of the AASHTO-PCI-ASBI 2700-2 section can be computed using Guyon’s formula:

$$\rho = \frac{I_c}{A_c y_t y_b}$$

where,

$I_c$  = Moment of inertia of the section

$A_c$  = Area of the section

$y_t$  = Distance from the top fiber to the center of gravity of the section

$y_b$  = Distance from the bottom fiber to the center of gravity of the section

The efficiency of the cross-section,  $\rho$ , is 0.6 which is considered to be high. For the sake of comparison, the flat slab is the most inefficient section with a  $\rho$  value of 0.33.

This design example utilizes a 4m typical segment length, resulting in a maximum segment weight of  $\approx 615.35\text{kg}$ .

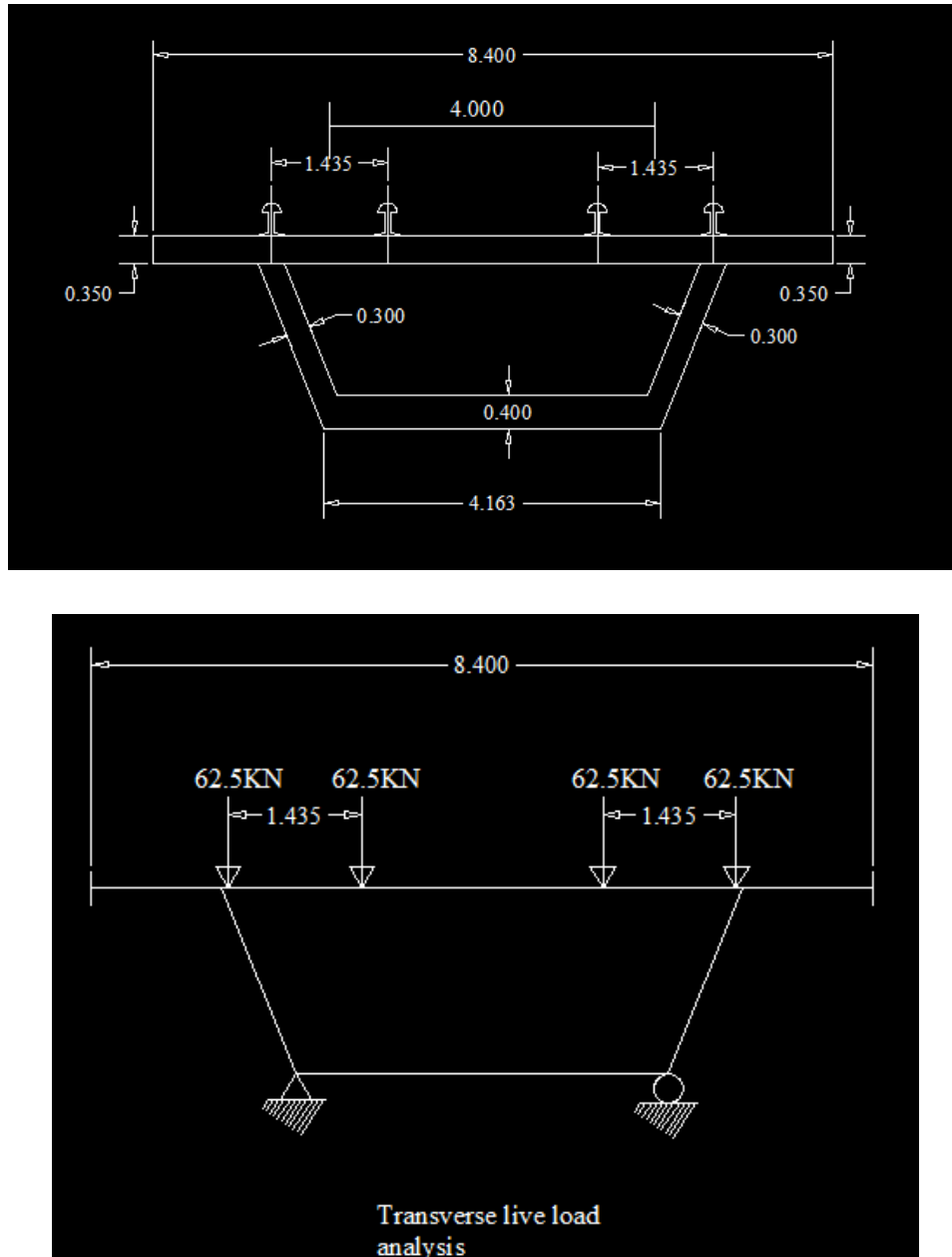


Figure 3.2 : Transverse live load

**Method of construction:**Overhead span by span precast segmental construction

**Track system:**Clamped continuous rail support Slab track system

**Minimum Dimensions:** Box Girder Cross-Section Dimension and Details [AASHTO & AREMA provision]

**Width of Bridge Deck:**The dimensions for a double track rail transit bridge are based on the center to center distance between the tracks which is 4m between the center lines of the track according to most code provision.

Therefore, accordingly the width of bridge deck is 8.4m.

**Minimum Flange Thickness:**Top and bottom flange thickness shall not be less than any of the following:

- 1/30 the clear span between webs or haunches. A lesser dimension will require transverse ribs at spacing equal to the clear span between webs or haunches.
- Top flange thickness shall not be less than 225 mm beyond anchorage zones or for pretensioned slabs.

Transverse post-tensioning or pretensioning shall be used where the clear span between webs or haunches is 4500mm or larger. Strands used for transverse pretensioning shall be 12.7mm diameter or less.

A top flang thickness of 225mm is preferable in area of anchorage for transverse post-tensioning tendons.A minimum flange thickness of 200mm is recommended.

**Minimum Web Thickness:**

The following minimum values shall apply,except as specified herein:

- Webs with no longitudinal or vertical post post-tensioning tendons-200mm
- Webs with only longitudinal (or vertical ) post-tensioning tendons-300mm
- Webs with both longitudinal and vertical tendons-375mm

**Length of Top Flange Cantilever :** The cantilever length of the top flange measured from the centerline of the webs should preferably not exceed 0.45 the interior span of the top flange measured between the centerline of the webs.

**Overall Cross-Section:** Overall dimensions of the box girder cross-section should preferably not be less than that required to limit live load plus impact deflection calculated using the gross section moment of inertia and the secant modulus of elasticity to 1/1000 of the span. The live loading shall be in accordance with the part "Live Load". The live loading shall be considered to be uniformly distributed to all longitudinal flexural member.

Girder depth and webs spacing determined in accordance with the following dimensional ranges will generally provide satisfactory deflection behavior:

- Constant depth girder

$$1/5 > d_0/L > 1/30$$

Optimum 1/18 to 1/20

Where:  $d_0$  = girder depth (mm)

L = span length between supports (mm)

- Variable depth girder with straight haunches at pier  $1/16 < d_0 > 1/28$  optimum 1/18  
At center of span  $1/22 < d_0/L > 1/28$  optimum 1/24  
A diaphragm will be required at the point where the bottom flange changes direction.
- Variable depth girder with circular or parabolic haunches at pier  $1/16 > d_0/L > 1/20$   
optimum 1/18  
At center of span  $1/30 > d_0/L > 1/50$   
Depth width ratio  
A single cell box should preferably be used when  $d_0/b \geq 1/6$  or  
Minimum Box Section Dimension and Detailing [AASHTO-API-ASBI provision]

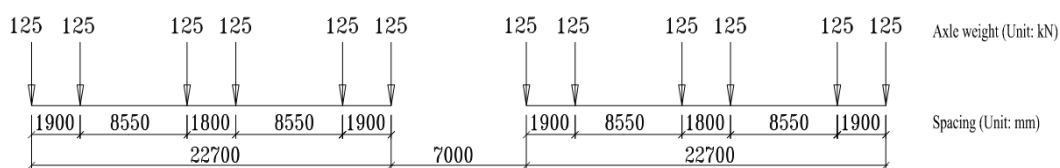
**Design Working Life:** The design service life is 100 years in normal operation condition.  
[According to ERA, AREMA and Chinese standard]

## Load Assumptions

### a. Live load (LL)

The Design Vehicle load should be based on the locomotive axle/load configuration proposed by the agency/company owning the railway transport. Since there is no standard live load for light rail vehicle incorporated in any of the latest design standards including AASHTO/AREMA (however AASHTO has already initiated a research in 2013 and expected to be finalized in 2016 to standardize live load for light rail vehicles) like AASHTO design vehicle load HS-15/HS-20/HS-25 etc.

However, in regards to A.A LRT bridges, the following live load is adopted and bridges should be designed and/or checked based on the given vehicle load.



**Figure 3.3:** Live load diagram

The live load should be considered a series of concentrated loads.

### b. Impact load

The impact force suggested by AREMA manual is proposed for ballasted-deck spans which account the dumping effect of ballast structure; which is not applicable for ballastless deck spans.

Moreover, the Chinese standard also suggests empirical approach to calculate impact load:

$U+1=1+0.8*6*a/(30+L)$ , where  $a=2$  and  $L$  is the calculated span of girder.

Based on this formula, the LRT project employed practically an impact load of  $\approx 19\%$  of LRV train load which assumed twin-mono block sleepers/ties on top of the deck structure helping to absorb some of the impact load coming from the LRV. However, for direct fixations of the rail

structure to the deck slab of the bridge render the bridge to an extra impact load effect. To account this effect, a reasonable value for impact load should be adopted. Besides, maintenance of rail irregularities and operation speed restriction could be adopted to minimize the impact effect on the bridge and based on these criteria, the Chinese provision for impact load is more or less reasonable value. This provision should be adapted for future LRT bridge projects.

### c. Longitudinal/traction or braking force

Longitudinal/breaking force mainly depends on the weight of the vehicle, acceleration or deceleration rate and rail/bridge interaction. Any provision for calculating/considering the value of this force should take in to account the above points.

Minnesota light rail transit design criteria provide with an empirical approach that considers weight of the vehicle and rate of acceleration or deceleration. The vehicular load and axle configuration is found to be similar to current LRV load adopted for the current A.A LRT project. This standard is with Chinese standard for longitudinal/breaking force is adapted for this research.

The longitudinal/breaking for suggested by AREMA manual is based on E-80(EM 360) loading. Hence provide unreasonably larger value of longitudinal or breaking force for the LRV loading.

Therefore, the Longitudinal/breaking force should be the greater on the two.

1. The magnitude of the longitudinal force shall be computed as follows:[According to Minnesota light rail criteria]

$$LF=0.046*W*A$$

Where W = vehicle weight

A = vehicle acceleration or deceleration rate in mph/sec

This force shall be applied to the rails and supporting structure as uniformly distributed load over the length of the train in a horizontal plane at the top of low rail.

The magnitude of the longitudinal force shall be computed as follows:[According to Chinese design specification for A.A LRT Project]

- Single track loaded; longitudinal force acting in anyone direction.
  - Both tracks loaded; one train accelerating while the other train is decelerating, or both trains traveling in the same direction. Both longitudinal forces acting in the same direction.
  - Both tracks loaded; both trains accelerating and decelerating. Longitudinal forces acting in opposite directions.
2. Breaking force or tractive force: braking force and tractive force should be calculated based on 15% of vertical static live load or 10% of vertical static live load when they are calculated together with centrifugal force. For the section double-track bridge, the braking force or tractive force should be based on that of the single track; for the double –track bridge at an elevated station or with 100m of inbound and outbound tracks of station should be calculated based on 10% of the double track vertical static live load. In case of combination of the breaking force or tractive force of double-track with other loads, it should be calculated respectively based on the same direction or reverse direction.[Chinese design specification for A.A LRT project]

#### **d. Rolling Force (RF)**

A force equal to 10% of the axle loading (20% of the wheel loading) per track shall be applied downward on one rail and upward on the other, on all tracks.

#### **e. Derailment load (DL)**

The derailment load shall be equivalent to a standard light rail transit vehicle axle load, plus an impact factor of 100 percent. For the derailment condition, a for the derailment condition, a derailment load from two adjacent axles (one truck) shall be simultaneously applied to the deck. The load from the remaining light rail vehicle axles shall be applied through the rail using a normal impact factor. The derailment load axles should be selected such that they generate the critical loading condition for the structure.

The derailment load shall be assumed to act anywhere on the deck not physically restricted. Where tracks are protected by guardrails, the maximum excursion due to derailment shall be limited to that allowed by the placement of guardrails.

A separate check for slab punching shear associated with the derailment shall be performed. The punching shear check shall consist of an equivalent static wheel load with applicable impact factor distributed over an area of one square foot.

When checking any component of the superstructure or substructure that supports two or more tracks, only one train on one track shall be considered to have derailed, with the other track(s) each loaded with a stationary train. No impact allowance shall be considered for the stationary train on the other track(s).

All elements of the structure shall be checked assuming simultaneous application of all derailed wheel loads. The reduction of positive moment in continuous slabs due to derailed wheel loads in adjacent spans shall not be allowed.

For prestressed concrete members, the steel stress shall not exceed 85 percent of the ultimate tensile strength ( $0.85 f_s$ ) and the concrete stress shall not exceed 60 percent of the 28-day compressive strength ( $0.60 f_c$ ). For load factor design, or load and resistance factor design, the live load factor may be reduced to 1.1. (Extreme Event). [According to Design standard for light rail structures of Massachusetts bay transport authority and Minnesota light rail transit design criteria]

#### **f. Wind Load (WL)**

Wind load consideration for LRT structure may be assumed to be similar to any highway bridges built in Ethiopia before. For this case, wind load on structure could be considered by the provision given on Ethiopian code of standard for bridge design (ERA, Bridge design manual-2002) which is intensively used for design of bridges structures around the country.

However, wind load on LRT vehicle differ from that of highway vehicles and this provision is not available in the design standard of ERA which only accounts roadway vehicle. Therefore, AREMA provision for wind load on vehicle is taken directly. Besides wind load consideration

on the current LRT project according to the Chinese standard is discussed under Design specification.

### **Wind load on Structures: WS [ERA, Bridge design manual-2002]**

Pressures specified herein shall be assumed to be caused by a base design wind velocity,  $V_B$ , of 160 km/h (= 45 m/s). Wind load shall be assumed to be uniformly distributed on the area exposed to the wind.

For large and/or light bridges the following shall apply. If justified by local conditions, a different base design wind velocity shall be selected for load combinations not involving wind on live load. The direction of the design wind shall be assumed horizontal, unless otherwise specified in the following subchapter Aeroelastic Instability. In the absence of more precise data, design wind pressure,  $P$  in kPa, shall be determined as:

$$P_D = P_B \left[ \frac{VDZ}{V_B} \right]^2 = P_B \frac{V^2 DZ}{25600}$$

Where  $P_B$  = base wind pressure specified in Table 3.3:

<b>Structural Component</b>	<b>Windward Load,Kpa</b>	<b>Leeward Load,Kpa</b>
Trusses,Columns,and Arches	2.4	1.2
Beams	2.4	Not applicable
Large Flat Surfaces	1.9	Not applicable

**Table 3.3:** Base Pressure,  $P_B$  corresponding to  $V_B = 160 \text{ km/h}$  (45 m/s)

The wind loading shall not be taken less than 4.4 kN/m<sup>2</sup> in the plane of a windward chord and 2.2 kN/m<sup>2</sup> in the plane of a leeward chord on truss and arch components, and not less than 4.4 kN/m<sup>2</sup> on beam or girder components. Wind tunnel tests shall be used to provide more precise estimates of wind pressures. Such testing should be considered where wind is a major design load.

Accordingly, wind load on the structure shall be taken as 2.4Kpa (2400Pa). The base wind load on the structure is assumed to act on the vertical projection of the structure applied at the center of gravity of the vertical projection in any horizontal direction. [According to ERA Bridge Design Manual – 2002.]

**Wind load on live load :(WL)[ . [According to AREMA, concrete structures, Vol2.]**

A wind load of 300 lb per linear foot (4.4 kN/m) on the train shall be applied 8 feet (2450 mm) above the top of rail in a horizontal direction perpendicular to the centerline of the track.

**g. Earthquake Load :( EQ)**

Subject to the Designers discretion, earthquake analysis will not be considered to have significant impact on any of the load cases and does not need to be considered for design.

Seismic analysis and design of LRT bridges shall be according to AREMA provision for Seismic design of railway structure along with ERA Bridge manual 2002 for Earthquake effect.

**Creep of concrete [AREMA, Prestressed concrete, vol 2]**

(a) For pretension and post-tensioned members:

$$CR_c = 12f_{cir} - 7f_{cds}$$

Where:

$f_{cds}$  = stress in concrete at centroid of prestressing reinforcement, due to all dead load not included in calculation of  $f_{cir}$ .

**Shrinkage of Concrete [AREMA, Prestressed concrete, vol 2]**

(a) For pretensioned members:

$$SH = 117 - 1.03 R$$

(b) For post-tensioned members:

$$SH = 0.8 (117 - 1.03 R)$$

where:

R = annual average ambient relative humidity in percent. The following map may be used to determine R.

**h. Rail Break Force (RB)**

For direct fixation track, provision shall be made for longitudinal forces due to a rail break. Forces from a single broken rail at any one time shall be applied to the structure.

Longitudinal break force shall be based on the maximum temperature differential in the rail. The mobilized forces in the structure are equal to the maximum restraint force in fasteners of the broken rail until the thermal force is equalized. The maximum allowable longitudinal gap in a rail due to rail break shall be 2 inches.

The structure shall be designed to include horizontal forces at the fixed bearing due to the summation of each rail fastener's longitudinal restraint. The structure shall also be designed to include a twisting moment in a horizontal plane at the height of the low rail due to opposing directions of forces in the broken and unbroken rails, [31].

**i. Construction loads [AASHTO LRFD Bridge Design Specification, Segmental Construction Load]**

Construction loads and conditions that are assumed in the design and that determine section dimensions, camber and reinforcing and/or prestressing requirement shall be shown as maxima allowed in the contract document. In addition erection load, any required temporary support or restraints shall be defined as to magnitude or included as part of the design. The acceptable closure force due to misalignment corrections shall be stated. Due allowance shall be made for all effects of any changes of the statical structural scheme during construction and the application, changes, or removal of the assumed temporary supports of special equipment, taking into account residual force effects, deformation and any strain-induced effects.

The following construction loads shall be considered for span by span construction:

- Weight of the supporting structure (DC)
- Superimposed load (N) or (N/mm) (DW)
- Distributed construction live load (CLL): an allowance for miscellaneous items of plant, machinery, and other equipment, apart from the major specialized erection equipment; taken as  $4.8 \times 10^{-4}$  MPa of deck area;
- Specialized construction equipment (CE): the load from segment delivery trucks and any special equipment, including a form traveler launching gantry, beam and winch, truss or similar major auxiliary structure and the maximum loads applied to the structure by equipment during the lifting of segments (N)

- Dynamic load from equipment (IE): determined according to the type of machinery anticipated (N)
- Longitudinal construction equipment load (CLE): the longitudinal load from construction equipment (N)
- Segment unbalance (U): the effect of any out-of-balance segments or other unusual condition as applicable: applies primarily to balance cantilever construction but may extend to include any unusual lifting sequence that may not be a primary feature of the generic construction system (N)
- Horizontal wind load on equipment (WE): taken as  $4.8 \times 10^{-3}$  MPa of exposed surface
- Static weight of precast segment being handled (N)
- Dynamic (AI): response due to accidental release or application of a precast segment load or other sudden application of an otherwise static load to be added to the dead load; taken as 100 percent of load A (N)
- CR=shrinkage effects in accordance with AASHTO LRFD Segmental Bridge Design provision
- SH=shrinkage in accordance with AASHTO LRFD Segmental Bridge Design provision
- Thermal (T): the sum of the effects due to the uniform temperature variation (TU) and temperature gradient (TG)<sup>(0)</sup>

### **Method of Analysis [According to ASSHTO LRFD and AREMA provision]**

#### **General**

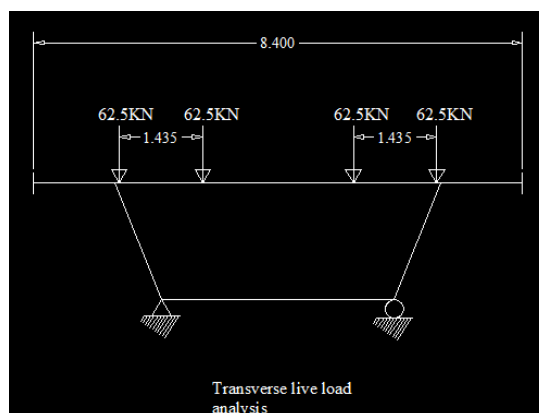
Elastic analysis and beam theory may be used to determine design moments, shears, and deflections. The effects of creep, shrinkage, and temperature differentials shall be considered, as well as the effects of shear lag.

#### **Transverse Analysis**

- a. The transverse design of box girder segments for flexure shall consider the segment as a rigid box frame. Flanges shall be analyzed as variable depth sections considering the fillets between the flange and webs. Combinations of track loads, if the structure may support more than one track, shall be positioned to provide maximum moments, and

elastic analysis shall be used to determine the effective longitudinal distribution of wheel loads for each load location. Tracks shall be positioned on the structure in accordance with clearance policies. Consideration shall be given to the increase in web shear and other effects on the cross-section resulting from eccentric loading or unsymmetrical structure geometry.

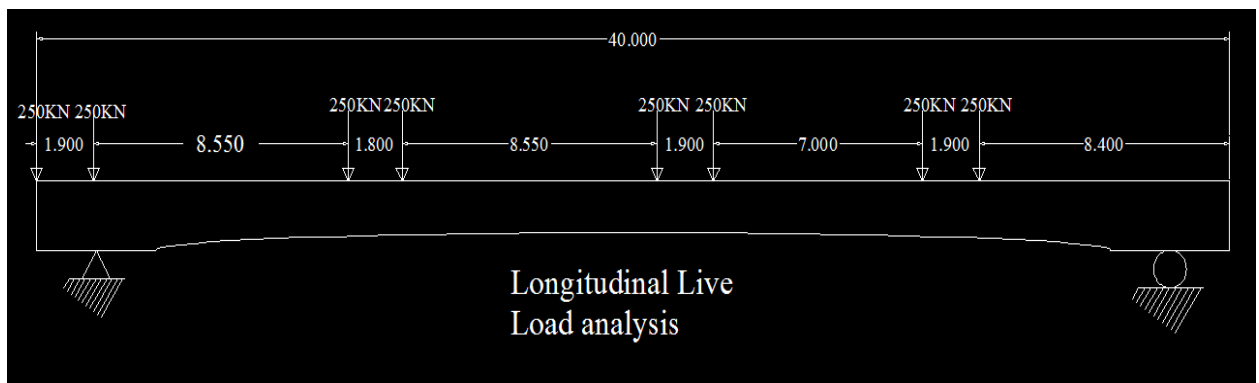
- b. Influence surfaces or other elastic analysis procedures may be used to evaluate live load plus impact moment effects in the top flange of the box section.
- c. Transverse elastic and creep shortening due to prestressing and shrinkage shall be considered in the transverse analysis.
- d. The secondary effects due to prestressing shall be included in stress calculations at working load. In calculating ultimate strength moment and shear requirements, the secondary moments or shears induced by prestressing (with a load factor of 1.0) shall be added algebraically to the moments and shears due to factored ultimate dead and live loads.
- e. The provision of influence surface such as those by Homberg (1968) and pucher (1964), or other elastic analysis procedures may be used to evaluate live load plus impact moment effects in the top flange of the box section.
- f. The effect of secondary moments due to prestressing shall be included in stress calculations at the service limit state and construction evaluation. At the strength limit state, the secondary force effects induced by prestressing, with a load factor of 1.0, shall be added algebraically to the force effects due to factored dead and live load and other applicable loads.[AASHTO LRFD Bridge 2005]



**Figure 3.4:** Transverse live load analysis

## Longitudinal Analysis

- a. Longitudinal analysis shall be in accordance with the provisions stated in “General Requirement”. Longitudinal analysis of segmental concrete bridges shall consider a specific construction method and construction schedule, as well as the time-related effects of concrete creep, shrinkage, and prestress losses.
- b. . The secondary effects due to prestressing shall be included in stress calculations at working load. In calculating ultimate moments and shear requirements, the secondary moments or shears induced by prestressing (with a load factor of 1.0) shall be added algebraically to moments and shears due to factored dead and live loads.
- c. Internal Tendons shall be designed and constructed as bonded tendons. Details of construction methods resulting in unbonded or partially unbonded internal tendons are not allowed.



**Figure 3.5:** Longitudinal live load analysis

## Deck Design

The top deck of a box girder is subjected to complex external forces, static and dynamic loads, thermal gradients, construction, creep and shrinkage effects. Proper consideration should be given to these effects to prevent cracking and deterioration. Studies have shown that transverse post-tensioning of top decks improves long-term deck durability and results in low life cycle cost. It is recommended that for all post-tensioned box girders the top deck be transversely post-tensioned, even for short overhangs. For bridges not subjected to freeze-thaw action and de-

icing chemicals, at least the deck should be partially prestressed. The top deck should be designed using elastic methods and then checked for ultimate limit states, not the other way around.

### Design Approach

To correctly represent the final system of the box girder, one would need to do a three dimensional analysis and incorporate all loads the box is subjected to along with proper boundary conditions. Due to complexity of this type of analysis, in particular the application of prestressing to three dimensional systems, this is seldom done. In lieu of this complex analysis, it is common practice to model the box as a 2-D (two dimensional) plane frame of unit length, as shown in Figure... If the thicknesses of the web and bottom slab vary along the length of the bridge, several 2-D frames may have to be analyzed in order to obtain a more representative interpretation of these varying cross-sectional properties. The 2-D frame model allows for load distribution to the webs and slab members relative to their stiffness.

A typical 2-D frame model is assumed to be supported at the lower end of the webs as shown in Figure While it could be argued that different boundary conditions exist for this model, this simplified assumption produces reasonable results.

The design loads considered in transverse design include, but are not limited to:

DC = Dead load of structural components and non-structural components, such as traffic barrier wall

DW = Dead load of wearing surface or future wearing surface and utilities if any

LL = Live load

IM = Dynamic load allowance

PT = Primary prestressing forces

EL = Accumulated locked-in force effects resulting from the construction process, including the secondary forces from post-tensioning

TG = Thermal gradient (+/-  $10^0$  F differential between the inside and outside of box girder)

Note: currently not required by AASHTO LRFD Design Specifications, but commonly done in standard practice

CR = Creep effect of concrete

SH = Shrinkage effect of concrete

Secondary forces of post-tensioning shall be included in ultimate limit state load combinations with a load factor of 1.0.

live Load configuration should be strategically placed in order to produce the worst condition  
Listed below are some common points where stresses are checked:

- Maximum negative bending moment at the root of deck overhang
- Maximum positive and negative bending moments at the center line between two webs
- Maximum negative bending moment in the top deck at the interior face of the webs
- Maximum negative and positive bending moments in the webs and bottom slab
- Maximum negative moment in the deck overhang where the taper begins

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## 4. Conclusion

Based on the discussion and case study under consideration which tries to assess current practices of design and construction approaches, the following conclusions are made:

1. In development of structural design of railway bridges, though Ethiopia has not practiced constructing railway bridges particularly light rail bridges, comprehensive standard for design of highway bridges is available basically similar to AASHTO LRFD design method with a slight deviation.
2. AREMA provision provides explicit guideline for design and construction of railway structures. AREMA provision is mainly developed for heavy vehicle railway structures which limit the applicable scope to light vehicle railway systems.
3. Railway bridges particularly light rail bridges design insist consideration of bridge- rail interaction which has a significant effect on the structural and operational efficiency of the bridge. Slab track are preferably over bridge section.
4. Jointed rail to rail connection has been practiced for long time and this experience shows that jointed rail-rail connection leads to frequent failure of the track and bridge structure due to the increased impact load at the discontinuities between consecutive rails and also cause passenger discomfort which raise serviceability issues.
5. Construction of LRT bridges especially around urban areas needs proper choice of construction methodology. Special features of constructing bridges around urban areas are: passing over congested traffic roads without interrupting or minimum disruption of traffic flow and reduced vibration and noise effect due to heavy machineries nearby buildings.
6. Cast-in situ construction scheme is not recommended for such condition since there is obvious consequence in this process.
7. Prestressed concrete has an advantage of predictable stress behavior than reinforced, much smaller section, high fatigue and impact resistance and greater crack control. Full span precast prestressed structures are usually not recommended for large project with many number of spans due to lesser construction efficiency.

8. Segmental bridge construction with its many advantages can be considered as the current state of art in bridge construction.
9. Segmental bridge constructions are easily applicable to box section and basically adapted to prestressed concrete with external post tensioning technique.
10. Selection of proper segment construction method is critical decision in bridge project build around urban areas.
11. Currently there are four techniques commonly practiced for the construction of segmental construction with their respective advantages and disadvantages. The criteria for selection of this method are mainly the length of the bridge, the individual span length, and height of superstructure along its length, access to the area below the superstructure, available construction equipments, available skilled personnel and total project cost.
12. The Addis Ababa Light Rail Transit (LRT) projects consist major of precast prestressing and span by span erection methods; a new achievement on bridge engineering has been made.
13. The section capacity of the A.A LRT Bridge for flexure is found to be adequate based on the comparative analysis computation (Appendix A) and stress condition of the section provided for the project.
14. The design section of the A.A LRT Bridge for flexure is shows excess according to this research result.

## 5. Recommendation

In due course of the research, in regards to the design and construction of light railway bridges in urban areas, valuable findings are able to be found which would help to improve our practices of bridge engineering in both design and construction. This part focus on suggesting improved design and construction approaches according to the current bridge engineering achievements in LRT Bridge construction which go with exclusive situation and capacity of our country. The guidelines are provided in the context of the research framework. Accordingly the following recommendations are made:

1. LRFD design philosophy adopted from AASHTO standard for design of highway bridges and this approach shall be adopted for railway particularly for light rail transit bridges too in accordance with the requirement of modifying the track-bridge interaction.
2. For design and construction of LRT bridges in Addis Ababa and other urban areas around the country in which the bridge section crosses over populated areas and congested roadways, **Precast-Prestressed-Segmental-Box section concrete** approach shall be adopted according to the scale of the bridge section from the overall project scope.
3. Design of Precast-Prestressed-Segmental bridge section shall be according to AASHTO segmental bridge specification along with AREMA for wind load and earthquake design provision. AREMA design standard is at most similar to AASHTO specification.
4. External post-tensioning method shall be adopted for segmental construction.
5. Live load and live load distribution shall be adopted according to the provision under Proposed Guideline for Design and Construction of Light Rail Transit Bridges. If any modification made to the Light vehicle type, live load consideration shall be according to the modified load type.
6. Track structure of the light rail transit over bridge section shall be direct fixation for Addis Ababa and other cities LRT system provided that slow speed vehicle is maintained.
7. Derailment of rail vehicle shall be considered and checked for critical condition.
8. Span-by-span segmental bridge construction is recommended for A.A LRT Bridges. This approach is recommended for the reason;(1) heavy load moving is not required for transporting segments from casting yard to final position unlike full span construction

method, simple lobed trucks may be employed;(2) overhead supporting gantries as truss form and lifting winchs can be manufactured locally or purchased in comparatively low cost;(3) Segment delivery is possible along completed deck to rear of gantry or at ground level;(4) Smaller crew size is required;(5) Good access provided within the gantry to all work fronts;(6) Fast rates of erection are possible due to use of external post-tensioning.

9. Due to time and resource constraints;dynamic coupling analysis, fatigue of prestressing strands, bearing designs, seismic response of segmental bridges, comparative design analysis for shear, life cost analysis for proposed design and construction approach and other alternative bridge section are not included in the study.However,the above points are vital and considerable area of future studies which would bear advanced findings in design and construction of bridges.

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