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Analysis and Design of Bridge Substructures in Eurocodes using Java Programming Language

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ABSTRACT

This research designed, analyzed and detailed concrete bridge substructures in Eurocodes using manual and computer methods. The design was done focusing only on the substructure being the lower part of a bridge. It is the one that supports the horizontal spans of the bridge by carrying all the weights of the superstructural members. In this work, a Java based computer program was developed to design, analyze and detail the bridge substructures. The Java computer program focused on the design and analysis of the abutments and pier caps using Courbon's method. The program made use of algorithms and visualization techniques. The bridge spans considered, ranged from 23.2m to 40m. The maximum design bending moments, maximum shear forces and the axial forces were 935.90kNm, 366.69kN and 682.48kN respectively. The manual analyses and designs were done for 23.2m, 25m, 30m, 35m and 40m bridge spans. These results were then compared to those obtained from the java computer program. It was seen that the results of both analyses and designs from manual procedure and java computer program, were relatively the same. The percentage difference results showed that, there were very little differences between the various results obtained from the manual and the computer program analyses, <<1%. This showed that, the developed Java computer program have been validated with Eurocodes BS EN 1992-1-1, BS EN 1991-1-4, and BS EN 1991-2, and can serve as a reliable and handy tool for the analyses and designs of bridge substructures. Therefore, BS EN 1991-1-4, BS EN 1992-1-1, and BS EN 1991-2, can easily be applied by using this Java computer program to design bridge substructures based on actor of safety, serviceability and economy. The Java computer program will also contribute to the performance of bridge structure in terms of suitability and reliability. It also gives quick and accurate analysis and design of bridge substructures.

Keywords:- *1. Analysis, 2. Design, 3. Bridge Substructures, 4. Eurocodes, 5. Java Programming*

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CHAPTER ONE
INTRODUCTION

A. Background of Study

According to Prayerful and Hanumant (2015), a bridge is a structure that permits passage over an obstacle without obstructing the path below. The obstacle to be crossed could be a river, a road, a railway, pedestrians, a canal, or a pipeline, while the passage could be used to build a road, a railway, or a pipeline. From a few meters to several kilometers, bridges can be found. They are among man's largest constructions. Material and design requirements are extremely high. A bridge needs to be able to withstand both the weight of the vehicles and people who use it and its own weight. Additionally, the structure must withstand a variety of natural occurrences, including temperature changes, strong winds, and earthquakes. The majority of bridges are constructed with a concrete, steel, or wood framework and an asphalt or concrete roadway for vehicular and pedestrian traffic. A regular bridge comprises of an upper part (the superstructure), that contains the slab, floor framework, and essential truss or girders, and a lower part (the substructure), that are towers, footings, abutments, columns, piers, and piles. The superstructure effectively carries traffic loads and provides horizontal spans, such as deck and girders. The substructure, lifting over the ground surface, ordinarily carry the horizontal spans, (Soumya and Umadevi, 2015). The type of bridge to be constructed, the scour depth, the kind of obstacle that the bridge is expected to avoid, and the nature of the subsoil all play a role in determining the foundation type for a particular bridge work. Well type or pile type of foundation can be used for any bridge. Al-Sarraf, et al., (2005), performed the analysis of composite bridge superstructures using Modified Grillage Method. Their analysis revealed that the Modified Grillage Method is simpler and provides adequate results in comparison to the Finite Element Method or Orthotropic Plate Theory solved using the Finite Difference Method. A PC program was made for developing slant bridge by Mahadevan, (2005), He improved on the operation of a Semi Continuum Method and determined the moments and shear forces in a right bridge as a precursor to his program. Kesarwani, et al., (2018), focused on load distribution on the support by utilizing the Courbon Method and other diverse methods. They decided how the live loads would be distributed among the longitudinal supports, also live loads for a girder and slab bridge bending moments were as well evaluated. Manohar et al., (2018), focused on the study of a single span, two-path T-beam span by altering the ranges of the bridges and deck slab depth with SAP 2000 software. Patel (2008) created a VB.Net-based software package for the analysis and design of concrete bridge substructures with simply supported spans. His analysis was based on the Class AA and Class A IRC loading categories.

A computer software package for the analysis and design of substructures for concrete bridges with simply supported spans based on Euro codes will be developed using the Java programming language in this research work.

B. Statement of the Problem

The manual analysis and design of all bridge components, be it superstructure or substructure can be very tedious and laborious. The structural engineer would be interested in trying the various sizes, forms and configurations of major components of the bridge system before concluding on most optimal combinations based on safety, economics and aesthetics of the bridge. Moreover, uncertainty always exists with respect to the subsoil conditions of the pier and foundation locations. These conditions may require a total change or considerable redesign of the foundation type. It is therefore necessary that a speedy, easy to reach and a dependable tool should be available to the structural engineer for the analysis and design of bridge substructures.

C. Aim and Objectives

➢ *Aim*

Design and analyze reinforced concrete bridge substructures on Eurocodes 2, using manual and Java programming language. This study aims to create interactional, easy computer software for the analysis and design of reinforced concrete bridge substructures that has simply supported spans.

➢ *Objectives*

The objectives of this study are:

- Manual design and analysis of bridge substructures using Eurocodes 2.
- Development of a computer package using Java Programming Language based on Eurocodes 2.
- Comparison of the results from the manual method with the developed computer method.

D. Scope of Study

This study will design and analyze simply supported bridge substructures based on eurocodes 2, using manual and computer methods. Two types of piers will be considered namely: Wall and hammer head type with round shaft. Wind, earthquakes, as well as hydrodynamic effects will all have an impact on gravity and lateral loads. At both the ultimate and elastic states, the well foundation analysis is conducted. For both vertical and lateral loads, only under-reamed and drilled cast-in-place piles in frictional and cohesive soils are used in the analysis and design of the pile foundation. Structural design of pile and pile-cap will be included in this work. Analysis and design of bridge superstructural components are not considered here.

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E. Justification of Study

Analysis and design of bridge substructure components were usually done using the provisions outlined by the various relevant design codes to prevent structural failure and collapse. Unlike IRC and LFRD, Euro codes are used across Europe and other English colonized countries like Nigeria. Analysis and Eurocodes in Nigeria are performed using BS 5400 or Eurocodes. Microsoft VB.Net and Java Programming language are two leading technologies intended for development of desktop and server side applications. Java Programming Language offers greater advantage to VB.Net because it's platform is independent which makes it easy to be run on any device. Hence, in order to provide a handy computer program that can analyse and design bridge substructure systems, it is very imperative to consider a programming language which will suit many users and various platforms.

CHAPTER TWO
LITERATURE REVIEW

❖ *Types Bridge Foundation*

There are various types of bridge foundations available to be used. The determinant of the type of foundation is driven by the subsoil condition and the success of the previous work done- in the locality. However, the type of superstructure will determine the nature of foundation to be used (Patel, 2008).

A. Shallow Foundations (Spread Footings)

As seen in all foundations, when designing spread footing geotechnical and structural issues and criteria are both considered (FHWA, 2012). In the design of spread footing, strength, movement and serviceability must to be taken in account in the calculations. Moreover, there are site-specific and construction related cases to be taken into account.

➢ *Bearing Capacity:*

Compare the maximum bearing pressure that can be applied to the maximum bearing pressure that is allowed.

➢ *Overturning:*

This check is exceptionally, significant for spread base abutments that regularly needs to endure horizontal soil strain on the backwall and for spread footing piers which will have enormous tipping over (overturning) moments, in particular, assuming they have tall piers.

➢ *Sliding:*

Spread base abutments, will generally have to defy lateral soil pressure action on backwall, require this check to be performed very carefully.

➢ *Settlement:*

While considering spread bases in soil, the normal settlement of the structure will have to be within the tolerable settlement calculated.

➢ *Horizontal Movement and Rotation:*

The anticipated horizontal movements (sliding displacements) and overturning rotations, especially for spread footings in soil, should be determined and equated to tolerable movements for the structure, even if the calculations for sliding and overturning indicate an adequate safety factor.

Spread footings are mostly used when solid rock is found at shallow depths, but they can be used to build foundations on soil where scour is not a factor.

The recommended values of the resistance factors for spread foundations is given in Fig 2.1

{See Fig 2.1 Partial Resistance Factors (gR) for Spread Foundations (Fig A.5 of EN 1997-1), page 1 Tables for Research work}.

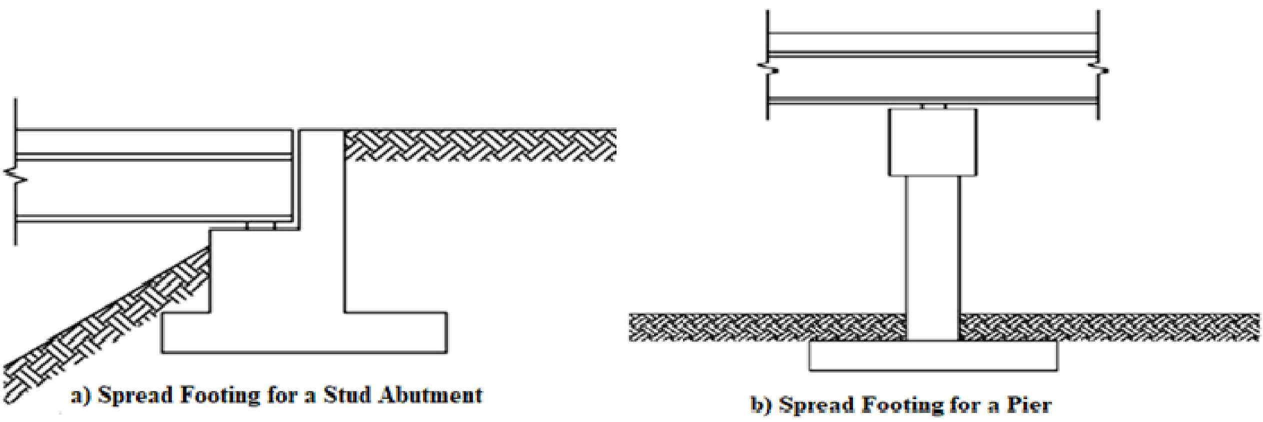

Fig 2.1 Spread Footing Applications (FHWA, 2012)

B. Deep Foundations

Deep foundations give backup to foundations, in manners that essentially not the same as spread footings. Driven piles or bored shafts, known as bored piers or bored caissons, are the two most common types of deep foundations. Deep foundations like piles and bored shafts are by and large lengthy, column-type elements which accomplish vertical capacity through end bearing, in a relatively deep bearing layer, side detrition through part or along their profundity, or having both together. The majority of the time, drilled shafts and piles achieve lateral capacity through embedment rather than sliding friction. Group action, in which the overturning moments are solved for axial force couples that are administered among grouped piles or bored shafts, or individual action, acting as flexural elements, is typically the method by which piles and drilled shafts acquire overturning capacity (FHWA, 2012).

C. Structural Design Considerations

Additionally, before determining its geotechnical capacity, the pile or bored shaft itself has to be assessed for its structural ability, which is dependent on the type of loads and the foundation's structural configuration. As part of structural design, axial, bending, and shear capacities, or a combination of all three, may need to be evaluated. According to FHWA, (2007), the required structural analyses are essentially straightforward and identical to that for column design. However, determining boundary consideration for structural analysis for deep foundations can be challenging. The boundary considerations for lateral support and stability are chiefly determined by the subgrade modulus of the surrounding soil, but the constraint applied at the top of the foundation element as well plays a significant role (FHWA, 2012). A less intricate or more mind boggling investigation can be utilized to quantify these limit conditions. Based on the structural setting, the pile can be modeled as a column with no other vertical support along its path and a fixed base, with boundary considerations at its top. Pile's depth from the assumed fixed base is referred to as the "point of fixity" (FHWA, 2007). Most times, piles are installed with sufficient soil embedment to provide adequate fixity. The fundamental question is how deep that fixity achieved. The fixity depth can be determined by either rule of thumb (for estimated or fundamental analysis) or a more exhaustive horizontally stacked pile analysis can be utilized to decide this. The pile is modeled with vertical spring supports whose spring constants are determined by the subgrade modulus of the soil layers surrounding it for a more comprehensive analysis (FHWA, 2012). This kind of analysis can be carried out with either standard software for finite element analysis or specialized software for modeling the interaction between soil and structures. In all circumstances, designers should also carefully look at the boundary considerations at the top of the piles or drilled shafts (FHWA, 2012). The characteristics of the substructure and its link to the foundation, also the characteristics of the superstructure and its link to the substructure, have a direct impact on the support provided to the foundation elements (FHWA, 2007). For instance, a link to the superstructure that is fully integrated can subsequently contribute to support a substructure/foundation system. Avoiding the urge to simplify the analysis is crucial. Some simplifying assumptions might not be taken into account at all, depending on the loading nature.

Values of the resistance factors for retaining structures recommendations are given in Fig 2.2

{See Fig 2 Partial Resistance Factors (gR) for Retaining Structures (Fig A.13 of EN 1997-1) page 1 Tables for Research work}.

D. Choosing Between Piles and Drilled Shafts

According to FHWA (2012), there are numerous circumstances when selecting between pile foundations and bored shaft foundations, including:

- *Subsurface Considerations:*

Relying on the situation of the subsurface conditions, drilled shafts and piles each have likes and dislikes. These circumstances frequently clearly favour one over the other.

- *Local Economic Considerations:*

Depending on their experience with on either of the deep foundations, the availability of workers with the necessary special skills, the accessibility and cost of specialty heavy equipment, and other factors, local contractors frequently prefer piles or drilled(bored) shafts.

- *Structural Conditions:*

Piles or drilled shafts may be selected based on the loading conditions and structural configurations. Individual driven piles for a smaller size may not be as effective at conveying lateral loads and moments as individual drilled shafts of a larger diameter.

- *Considerations for Constructability:*

Piles or drilled shafts may be chosen depending on access to equipment, site conditions, or structural configuration.

- *Environmental Factors:*

Environmental degradation is more severe with some foundation construction methods and types than with others. In order to access the environmental impacts and environmental commitments that is likely to influence the selection of the chosen type of foundation and construction method, the contractor will collaborate with specialists in environmental permitting.

- *Piles*

One broad class of deep foundations is driven piles. Driven piles are predominantly characterized by the fact that driving operations represent either a large part or the entirety of the field construction operations. Piles are also typically, but not always, characterized by relatively slender cross-sectional dimensions compared to drilled shafts. Barker, et al., (1991) and Hannigan, et al., (2007), provide detailed discussions of driven pile foundations.

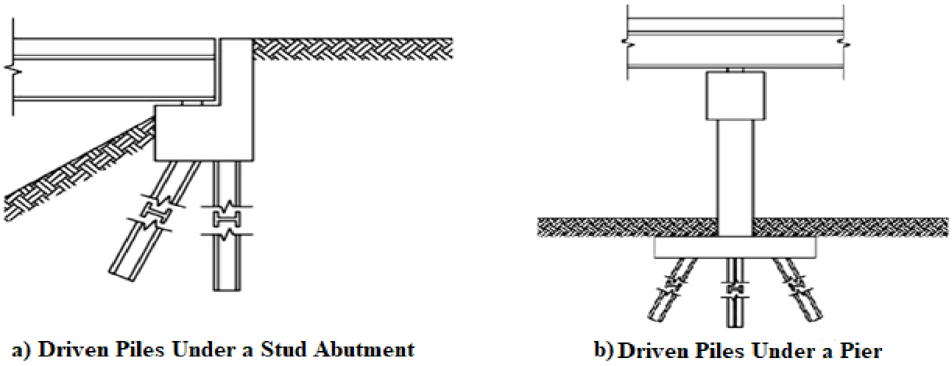

Fig 2.2 Driven Pile Applications (FHWA, 2012)

➤ *Pile Types*

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➤ *Factors Affecting the Choice of Pile Type*

According to FHWA, a variety of factors influence the choice of pile type. Though, examples were listed below. Designers should bear this in mind while choosing type of foundation, that, it could be complicated and a lot of difficult problems. Precast concrete piles are a better choice when steel pipes are likely to be severely corroded. Nevertheless, it would be hard driving solid or closed-end piles on dense soil made of precast concrete piles (with driving plate). Steel pipe and precast concrete piles may be chosen over steel H-piles, when lengthy unbraced pile are required and the greater buckling and flexural capacity of steel pipe and precast concrete piles are desired. At the point when the necessary pile lengths are adequately lengthy and require joining, owing to the expenses and challenges related with joining steel piles, concrete piles will be used. Steel piles can be joined fairly easily. Piles made of timber is typically restricted to structures with light loads when there is limited access to the site and there is local access to high-quality timbers. When there is a top-of-the-line bearing capacity in soil or rock, steel H-piles are absolutely necessary because the cross sectional area required is small for pile to support the needed end bearing capacity. Piles that has bigger cross sections were appealing owing to their larger perimeter (more area available for generating side friction resistance) in situations where side friction is the primary source of vertical capacity.

➤ *Specific Design Considerations for Piles*

Moreover, all the acknowledged design conditions concerning deep foundations as aforementioned, it is frequently the notice of designers by FHWA (2012), that driven piles are comparatively slender member. In pile bent, bending capacity and shear buckling are considered as well as the second-order slenderness effect when doing structural analysis of the pile. Group piles are used in many bridge foundations. Group pile geometry and improper spacing of piles in a group are the dangers to look out for by designers, also to be aware of these likely adverse effects as:

- Group effects that result in a decrease in lateral and/or vertical capacity.
- Possibilities for uplift as a result of overturning moments.
- Hindrances that could happen between next piles when more than one piles are battered.

Caution has to be applied as battered (or "brace") piles are utilized. As long as the lateral capacity is meticulously evaluated, giving that battered piles tolerate lateral capacity of the pile foundations, there is increase in axial load as regard the effect of lateral load. Hence, check for possible pile hindrances. Additionally, "too many battered piles" are brought to the notice of designers. In some circumstances, mainly in abutments, where piles may be battered in one direction alone to withstand lateral soil pressures, the substructure may "walk backward" into the retained fill if the longitudinal component of the battered pile axial loads is bigger than the passive pressure of the retained fill of the substructure. The idea was that the out-of-plumbness and out-of-position top of pile tolerances during construction should be considered by designers. The pile design ought to consider these axial load application due to eccentricities allowance. It is essential to take note of that relying upon how the contract was written his capacity to manage piling the impacts of out-of-plumbness and top of pile out of position might be must do. A pile might, for example, be notably out of position at its lower end while, yet fulfilling the out-of-plumbness necessity, while at the same time, fulfilling the out-of-position prerequisites by being out of position at opposite direction, at top of the pile. In these circumstances, the combination of the two effects results in the net effective eccentricity.

- *Design of Pile Foundation*

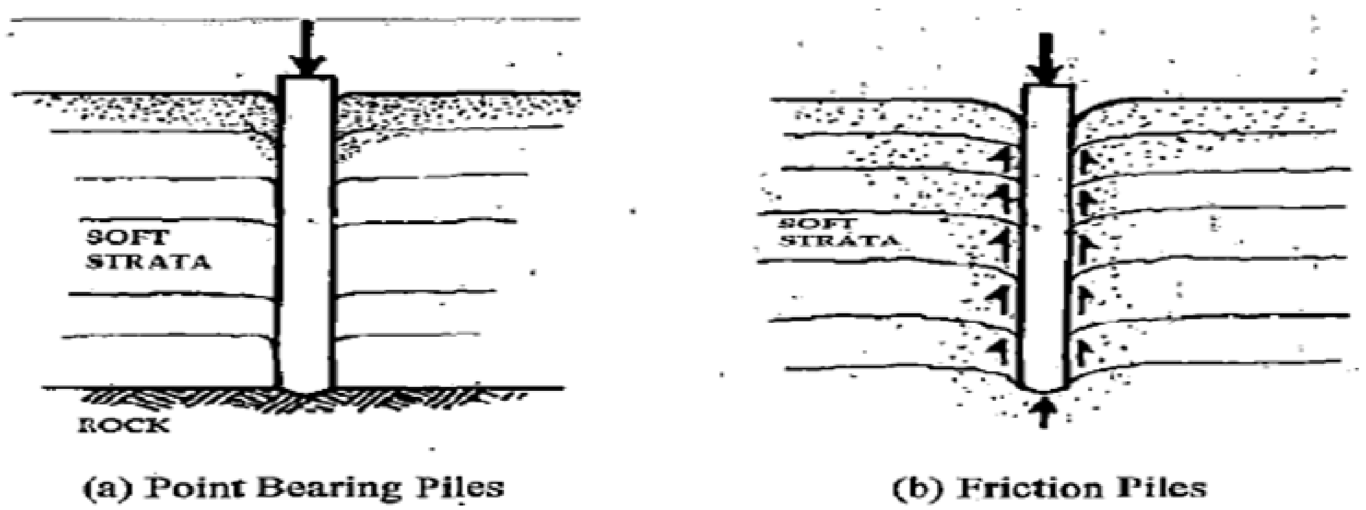

Fig 2.3 Pile Classifications on the Basis of Load Transfer Mechanism (Patel, 2008).

When loaded, a pile transfers the load through point bearing at the pile's tip and skin friction along its length. As a result, the assumption that a pile's ultimate bearing capacity is the sum of its total ultimate point or end bearing resistance q_p and its total ultimate skin friction f_s can be used to calculate its ultimate capacity Q_u (Patel, 2008).

$$Q_u = Q_s + Q_{p,} = f_s A_s + q_p A_{p,} \qquad (2.1)$$

Where

Q_s = Total skin frictional resistance
Q_p = Total point bearing resistance
f_s = Unit skin frictional resistance
A_s = lateral surface area of the pile
q_p = Unit point resistance and
$A_{p,}$ = area of the pile tip

For piles in Cohesionless soil, the unit point bearing resistance is given as by (Patel, 2008):

$$q_p = \overline{p_v}(N_q - 1) + \lambda_s \gamma B N_\gamma \qquad (2.2)$$

Where

$\overline{p_v}$ = effective overburden stress at the level of the pile tip
B = diameter or width of pile
γ = density of the soil
λ_s = shape factor, 0.4 for square or rectangular piles, and 0.3 fo rcircular piles
$N_q$andN_γ = bearing capacity factors

For piles in cohesive soils, Patel (2008) gave the point bearing resistance and the skin frictional resistance as follows:

The unit point bearing resistance:

$$q_p = c_u N_c \tag{2.3}$$

c_u = undrained cohesion at the pile tip and
N_c = bearing capacity factor generally taken as 9

The unit skin resistance:

$$f_s = \alpha c_u \tag{2.4}$$

Where

α = adhesionfactor

- *Distribution of Load between Vertical Piles of Pile Group*

The load acting on an individual pile is obtained from the elastic theory as:

$$Q_i = \frac{Q}{n} + \frac{M_{yy} x_i}{\sum x^2} + \frac{M_{xx} y_i}{\sum y^2} \tag{2.5}$$

Where

Q_i = load on the pile
Q = Total vertical load acting on the foundation
n = Total number of piles in group
M_{yy} = Moment acting at the soffit of pile cap about longitudinal axis of bridge
M_{yy} = Moment acting at the soffit of pile cap about transverse axis of bridge
x_i = distance of the centre of the pile from the centre of gravity of pile group measured parallel to transverse axis of bridge,
y_i = distance of the centre of the pile from the centre of gravity of pile group measured parallel to longitudinal axis of bridge,
$\sum x^2$ = summation of squares of distances of the centres of all the piles from the centre of gravity of pile group measured parallel to transverse axis to bridge,
$\sum y^2$ = summation of squares of distances of the centres of all the piles from the entre of gravity of pile group measured parallel to longitudinal axis to bridge.

If the calculated load on a pile exceeds its safe bearing capacity, then the piles are required to be redesigned. The option is that, the number of piles or the spacing between piles can also be increased to reduce the maximum load acting on the pile.

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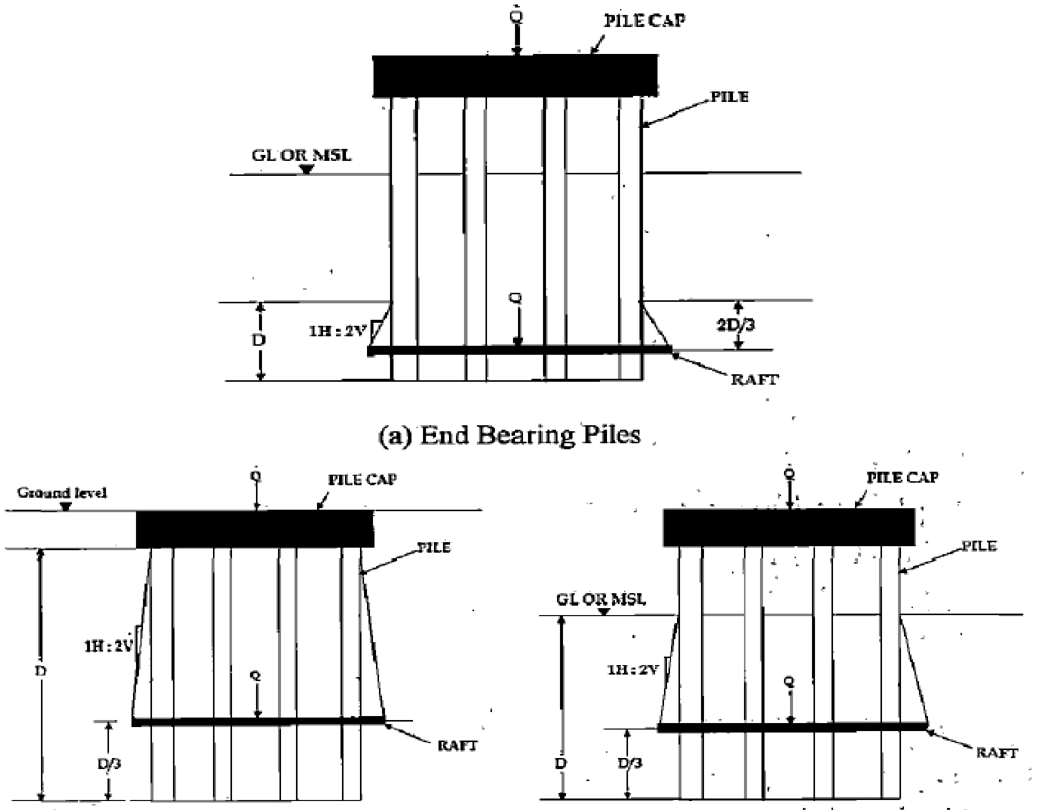

(a) End Bearing Piles

(b) Friction pile, with pile cap embedded into the ground

(c) Partially embedded friction piles

Fig 2.4 Computation of Settlement for end bearing Piles & Friction Piles (Terzaghi and Peck (1967)

- *Settlement of Pile Group in Cohesionless Soils*

The settlement S_i of the pile group in a soil layer can be estimated from the following equation proposed by De-Beer and Martens method (1957):

$$S_i = 2.303 \frac{H}{C} \log_{10}\left(\frac{\overline{p_v} + \Delta p}{\overline{p_v}}\right) \tag{2.6}$$

Where

S_i = settlement of the layer considered,
H = height of layer,
$\overline{p_v}$ = mean effective overburden pressure for the layer,
Δp = average increase in vertical stress in the layer due to footing load.

This may be obtained assuming 1H:2V load dispersion

$$C = \text{constant of compressibility} = \frac{1.5 q_c}{\Delta p} \tag{2.7}$$

Where, q_c = average static cone penetration resistance for the layer considered

- *Settlement of Pile Group in Cohesive Soils*

For piles in normal consolidated clays, the settlement S_i is given by

$$S_i = \frac{C_c H}{1 + C_0} \log_{10}\left(\frac{\overline{p_v} + \Delta p}{\overline{p_v}}\right) \tag{2.8}$$

Where

S_i = settlement of the layer considered,

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$\mathrm{H} = \text{heigh to flayer},$
$\overline{\mathrm{p_v}} = \text{mean effective overburden pressure for the layer},$
$\Delta\mathrm{p} = \text{average increase in vertical stress in the layer due to footing load.}$
This may be obtained assuming 1H:2V load dispersion

Where

$\mathrm{C_c} = \text{compression index and}$
$\mathrm{e_0} = \text{void ratio in the clay layer corresponding to the effective in} - \text{situ over burden}$
Pressure.

For piles in pre-consolidated clays, the settlement $\mathrm{S_i}$ is given by

$$\mathrm{S_i} = \frac{\mathrm{C_c H}}{1 + \mathrm{C_0}} \log_{10}\left(\frac{\overline{\mathrm{p_v}} + \Delta\mathrm{p}}{\mathrm{pc}}\right) \tag{2.9}$$

$\text{Where, pc} = \text{pre} - \text{consolidated pressure of the layer considered.}$

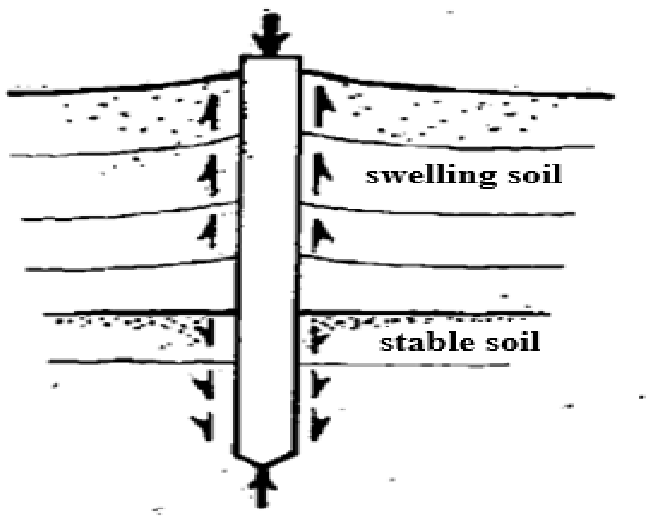


Fig 2.5 Piles in Expansive Soils can Control Seasonal Movements (Patel, 2012)

➢ *Drilled Shafts*

The variety of deep foundation types is somewhat more limited when it comes to drilled shafts. Drilled shafts most likely share a few characteristics in common: a round, relatively deep hole that was bored in the ground and filled with reinforced concrete. O'Neil, et al., contains two useful references on the construction and design of drilled shafts: (1999) as well as Barkar et al., (1991). Penetrated shafts are some of the time classified according to wellspring of their lateral capacity as either end bearing bored shafts, side friction bored shafts, or a mix of the pair.

According to FHWA (2012), several additional characteristics vary from application to application for drilled shafts:

- *Casing*:

Steel casing is frequently used to hold the drilled hole open till concrete is casted when shafts are drilled through caving-prone soils. The casing may then be removed in some instances, while in other instances, it may remain in place.

- *Bottom Configuration:*

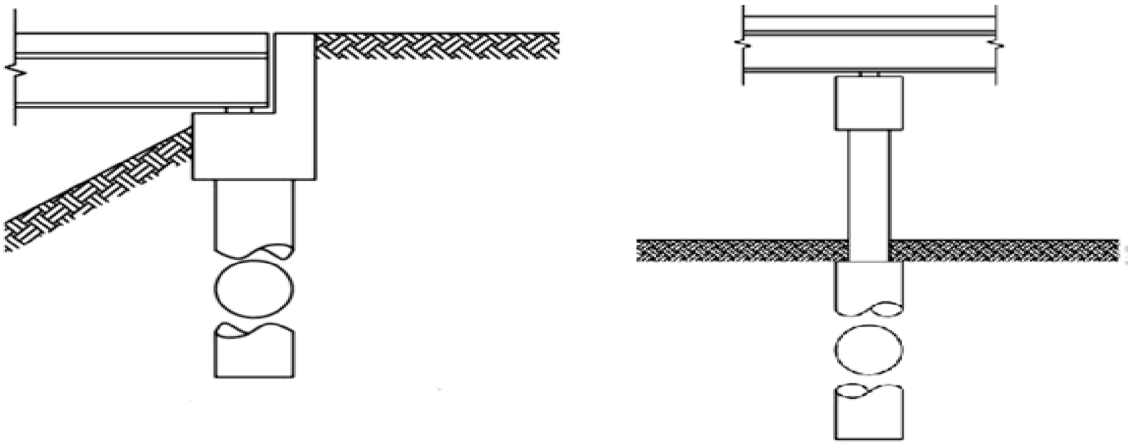
For the increment of the end bearing area, the bottom of a drilled shaft may be expanded in some instances. Due to the bell-shaped shape at the shaft's tip, these are referred to as "belled" drilled shafts.

- *Rock Attachments:*

At times where penetrated shafts are introduced in rock with soil overburden, the shafts are bored a short profundity into rock to get solid end bearing or to accomplish lateral fixity. At times, measurement of these stone attachments is somewhat not exactly same as that of the remainder of the penetrated shaft.

- *Concrete Placement Method:*

The different approaches to pouring/casting of concrete in bored shafts. The situation inside the shaft during placement, the presence of formwork, and/or the presence of water or slurry in the shaft all play a role in determining which option is chosen. Concrete can be dropped into clean, dry shafts, at least up to a chosen maximum drop height. Concrete is sometimes poured under water using a tremie tube, which is kept inserted in the fresh concrete, if the shaft contains ground water. In different occurrences, slurry is utilized to remove the water, then the tremie poured/casting the concrete, which thus removes the slurry. The utilization of the slurry-deracination method gives relief from casing in order to protect the unstable earth as since the slurry can be utilized to forestall and hole the soil. However, the slurry prevents inspection of the shaft excavation's bottom prior to concrete placement.



a) Drilled Shafts for a Conventional Stub Abutment **b) Drilled Shafts used for a Pier**

Fig 2.6 Drilled Shaft Applications (FHWA, 2012)

➢ *Factors Affecting Choice of Drilled Shaft Type*

Subsurface conditions and constructability issues are typically the primary factors that influence the selection of drilled shaft types. The choice of concrete placement method and whether the shaft will be cased are directly influenced by the presence of groundwater and/or unstable, caving soils. The utilization of belled penetrated shafts is no longer in use nowadays, essentially because of the muddles related with their application and furthermore to guarantee a perfect base for end bearing. The required vertical capacity and lateral fixity, as well as the means by which these can be achieved, typically determine the need for rock sockets. Rock sockets may not be necessary if side friction through soil alone provides sufficient vertical capacity and lateral fixity.

➢ *Specific Design Considerations for Drilled Shafts*

Furthermore, designers are reminded that, despite the fact that drilled shafts are typically relatively stocky members, care should be taken when performing the structural analysis, evaluating the axial load and bending capacity, typically through an axial-bending interaction analysis. This is in addition to the general design conditions for deep foundations that were discussed earlier. Drilled shaft structural design should be approached in the same manner as regular reinforced concrete column design.

E. Abutments (End Bents)

Abutments, also known as End bents, provide support for the superstructure at bridge's ends (FHWA, 2012). Abutments typically have to withstand loads from the superstructure as well as soil pressure loads in order to maintain the approach roadway embankments (note that soil pressures can rise during seismic events). There are several fundamental kinds of abutments, that can comprehensively be grouped for discussion purposes as customary, semi-essential, and fundamental abutments. The criteria for the selection of these three abutment types are, with a few exceptions, as follows: geometry of the bridge, such as its length and skew, as well as any additional geometric constraints, such as the necessary longitudinal clearances, anticipated loads, problems with future maintenance, as well as preferences for local owners and agencies.

F. Conventional Abutments

These attributes recognize regular abutments, otherwise called seat-type abutments: a joint that isolates the bridge deck from the abutments backwall and front slab/front pathway, as well as a member that isolates the superstructure from the abutment, bearings. The regular design and construction of abutments can be improved on account of these divisions. Consequently, the interface between the superstructure and substructure is distinct and well-defined. Contrarily, conventional abutments call for the use of expansion joints, which demand an upkeep and have the possibility to jam or leak, putting the girder, bearing, or abutment at greater risk of deterioration. Furthermore, integral or semi-integral abutments are more robust and redundant than conventional abutments.

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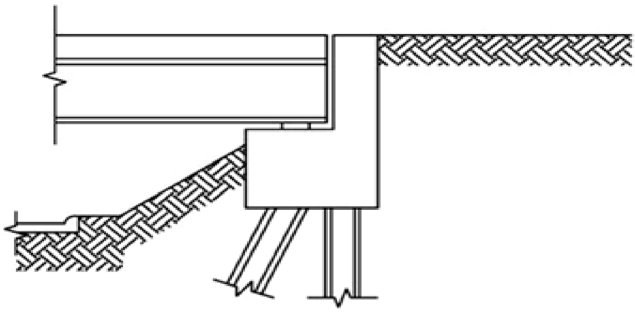
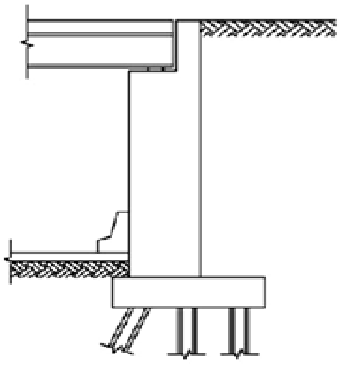
G. Design Considerations for Conventional Abutments

In the design and layout of conventional abutments, many issues are to be looked into. Many are outlined below (FHWA, 2012):

- *Height:*

Conventional abutments can be broadly categorized in terms of height as either stub abutments or tall abutments. In a stub abutment, the depth of the abutment cap is set at a nominal, and usually fairly shallow, depth, typically not much deeper than the cap width, or less. Stub abutment caps depths are often standardized as an owner-agency preference. Stub abutments usually also feature a header slope in front of the abutment cap. The grade of the header slope can be as shallow as 4:1 or shallower or as steep as 1:1 or steeper, depending on owner-agency typical preferences, aesthetic considerations, clearance considerations, and notably, slope stability considerations, including consideration of the type of slope protection used, if any. Tall abutments, on the other hand, function as retaining walls as well as supports. Tall abutments are often used when horizontal clearance requirements below the bridge prohibit the use of a header slope, or where superstructure span lengths restrict the location of the abutment. Sometimes, a stub abutment is used in conjunction with a separate retaining wall in front of the abutment. This can be done to achieve similar geometric goals as a tall abutment in terms of maintaining horizontal clearances. Any of a number of retaining wall types can be used: Mechanically Stabilized Earth (MSE) walls, soil nail walls, drilled shaft walls, etc. There are a few caveats, however. First of all, careful coordination is required to ensure the abutment foundation elements do not interfere with any of the wall elements, especially when the wall uses straps, tie-backs, soil nails, etc. Also, the wall designer needs to be fully informed of the abutment configuration when designing the wall to make sure that all loads are correctly quantified in the wall analysis. At a minimum the abutment and its backfill represent a surcharge loading on the wall, and depending on the abutment and the wall configurations, additional loads may exist as well.





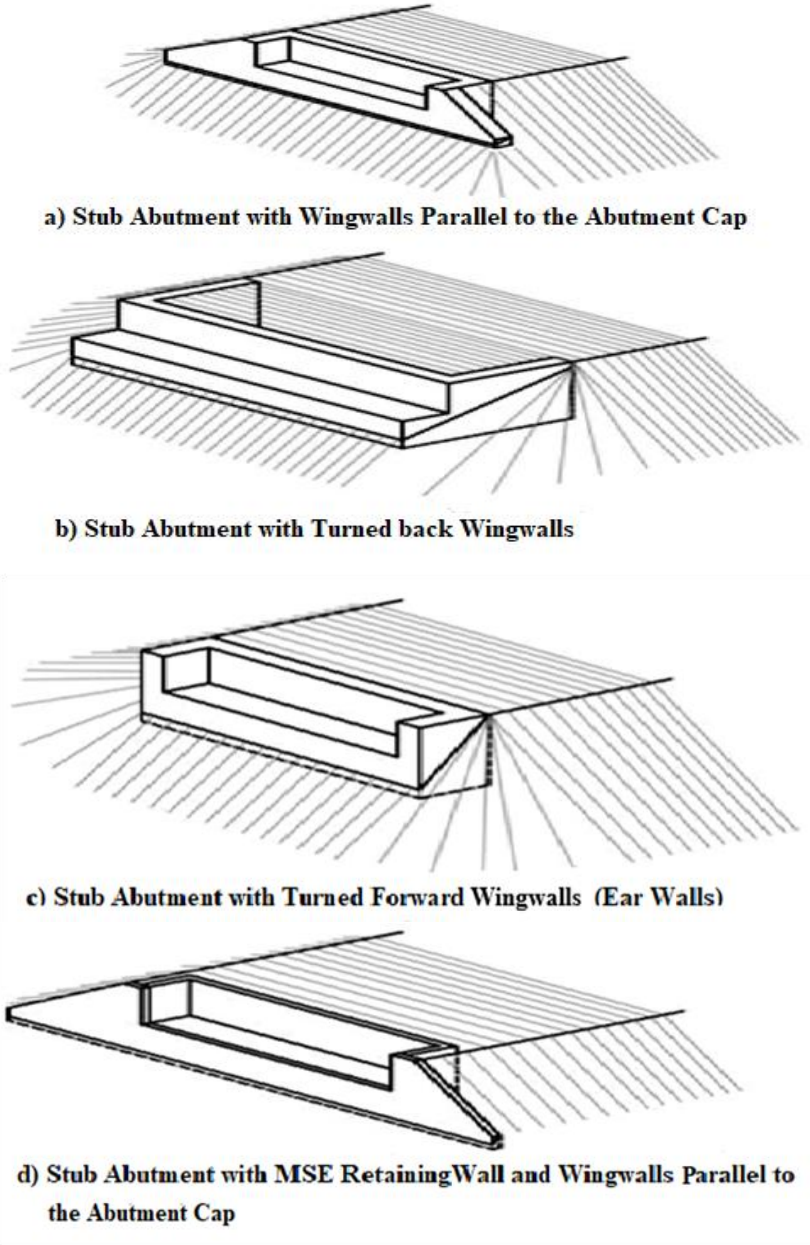
a) Conventional Pile Founded, Stub Abutment **b) Conventional Pile Founded, Tall Abutment**

Fig 2.7 Conventional Abutments of Various Heights (FHWA, 2012)

- *Width:*

The width of an abutment cap is being controlled by many conditions, include:

- ✓ The demand to fit bearings and anchor bolts with adequate edge distances.
- ✓ The demand to fit one or more rows of piles or drilled shafts with enough spacing and edge distance.
- ✓ The demand to meet seismic detailing guidelines related to required seat widths.



a) Stub Abutment with Wingwalls Parallel to the Abutment Cap

b) Stub Abutment with Turned back Wingwalls

c) Stub Abutment with Turned Forward Wingwalls (Ear Walls)

d) Stub Abutment with MSE RetainingWall and Wingwalls Parallel to the Abutment Cap

Fig 2.8 Abutment Wingwall Configurations (FHWA, 2012)

- *Wingwall Configuration:*

Wingwalls are made to retain the backfill which would otherwise "spill around" the ends of the abutment backwall and cap. Wingwalls can be oriented in a number of directions including parallel to the cap, angled at some angle (e.g., 30 deg., 45 deg.), turned back (parallel to the roadway, pointed away from the span), or turned forward (parallel to the roadway, pointed toward the span). The preferred orientation and layout of wingwalls is usually determined by owner-agency preference or local practice.

- *Expansion Joints:*

The design of expansion is done in such a way it allows for free anticipated movements of the superstructure relative to the abutment. Care should be taken in calculating these movements to account for all potential sources of movement. Thermal expansion and contraction are typically the primary sources of these movements but other sources may exist, particularly for longer structures or structures subjected to seismic events. Expansion joints also need to be designed structurally for anticipated vehicle loading, although in many cases this boils down to selecting an appropriate joint from a selection of standard owner-agency or vendor designs based simply on the anticipated traffic. Many different joint types exist, satisfying a wide range of design criteria. Since joints require maintenance, it is generally advisable to have significant input from the owner-agency regarding choice of the best joint type, so that, many owner-agencies have very explicit rules on aforementioned topic.

- *Backfill:*

The selection of the appropriate backfill material for abutments has received a lot of attention. Backfill material demands for abutments typically vary greatly from state to state, and occasionally from county to county or city to city. Larger part of proprietor organizations has normalized subtleties and determinations concerning backfill material for abutments. Some backfill materials need to be cement-stabilized, while others need to be free-draining granular backfill materials that can be reinforced with geotextile fabric, for example. The requirements for the backfill material, which also include provisions to make drainage easier, will further be discussed. Backfill material provisions for abutments should be discussed with the appropriate owner-agency, according to the instruction given to designers.

- *Approach Slabs:*

Serviceability, design, and detailing of approach slabs vary significantly between owner-agencies and designers, as does the shapes of the structures (integral versus non-integral abutments, for example) and regions of the country. The length of approach slab, the design methods used, the preferences for the detailing. How to deal with skews, how to connect to the abutment backwall, and even whether or not to provide approach slabs are all questions to which the responses vary greatly from owner agency to owner agency. Approach slab guidelines should be discussed with the appropriate owner-agency by designers.

- *Drainage:*

Drainage issues fall into two categories: top surfaces drainage detailing of the abutment as well as the backfill material behind the abutment. To keep water from amassing or ponding on the highest point of the abutment, presence of mind rules ought to be observed to make slants that appropriate drainage channel. Owner-agencies frequently combine their chosen details for the abutment or their chosen details for the backfill material with standard drainage details for the drainage of the backfill material behind the abutment. These will consolidate the usage of free-draining granular top off materials, drainage strip materials set against the abutment backwall, weep openings are furthermore given in the abutment, underdrain structures, etc.

According to FHWA (2012), the conventional abutments elements require some degree of design as follow:

- *Abutment Caps:*

Designing the cap for lateral loading, the beams length runs between the foundation elements. This might the point where the meet abutments on spread footings. The design should involve conditions for lateral moment and shear. The design may need to treat torsion as well, especially if the abutment or its backwall are tall in particular, or if there is noticeable eccentricity between the centerline of application of superstructure reactions (centerline of bearing) as well as the centerline of the foundations.

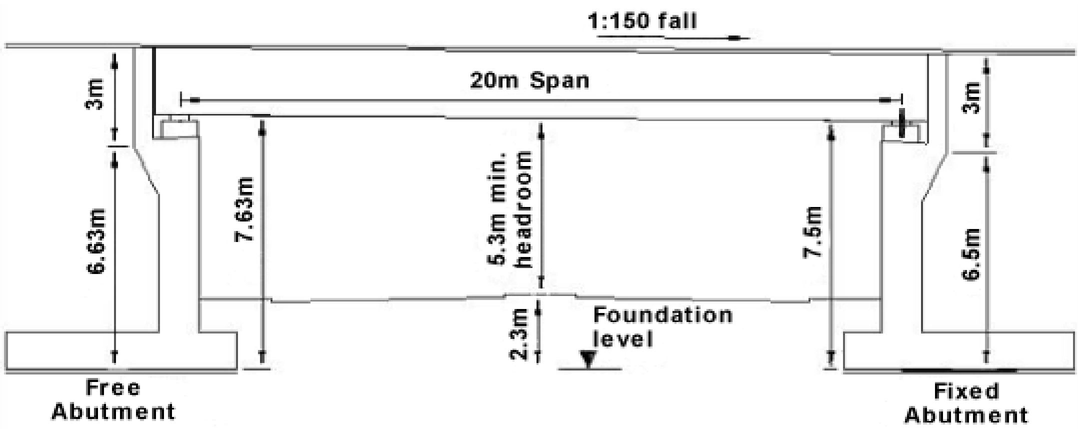


Fig 2.9 Abutment Design (Childs, 2018)

- *Abutment Backwalls:*

Backwalls are generally constructed as cantilever retaining walls that transmit backfill material-induced lateral soil pressure. Tractive forces must also be applied to the backwall's top as demanded by some agencies.

- *Abutment Wingwalls:*

Almost all designers practice some discretion in the design of wingwalls due to the complex nature of their support and connection to the rest of the abutment. Designers are encouraged to develop rational design procedures based on careful and realistic consideration of the particular detailing of the wall. Some wingwalls, particularly longer walls, rest on pile or drilled shaft foundations, some use spread footing foundations, some are cantilevered off the abutment cap. Some are square and some are tapered. Most wingwalls will generally behave in a manner that can be adequately captured by a conservative, simplified cantilever beam analyses. In other cases, an analysis based on plate theory may be more appropriate.

- *Abutment Foundations:*

Foundation system should be taken into account during the abutment cap analysis, as well as the calculations did for foundation loads, for use in the foundation design. Various ways of calculating the abutment foundation loads, as taken by designers and owner-agencies. It is sometimes assumed that the abutment cap functions as a fixed body that evenly distributes all vertical loads among all piles. In other instances, the load distribution to each pile is calculated using standard beam theory, and the abutment cap is assumed to function as a continuous beam on pin supports.

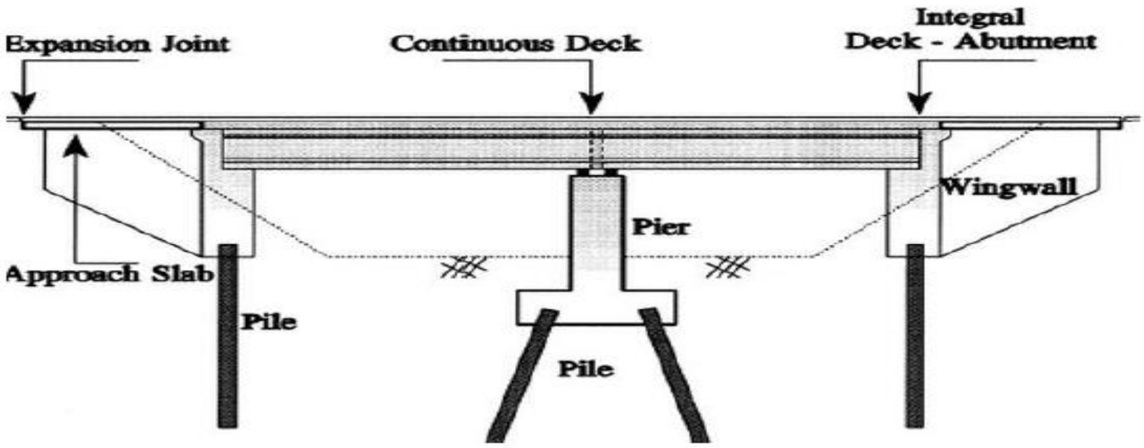


Fig 2.10 Abutment Foundations (Anwar, 2015)

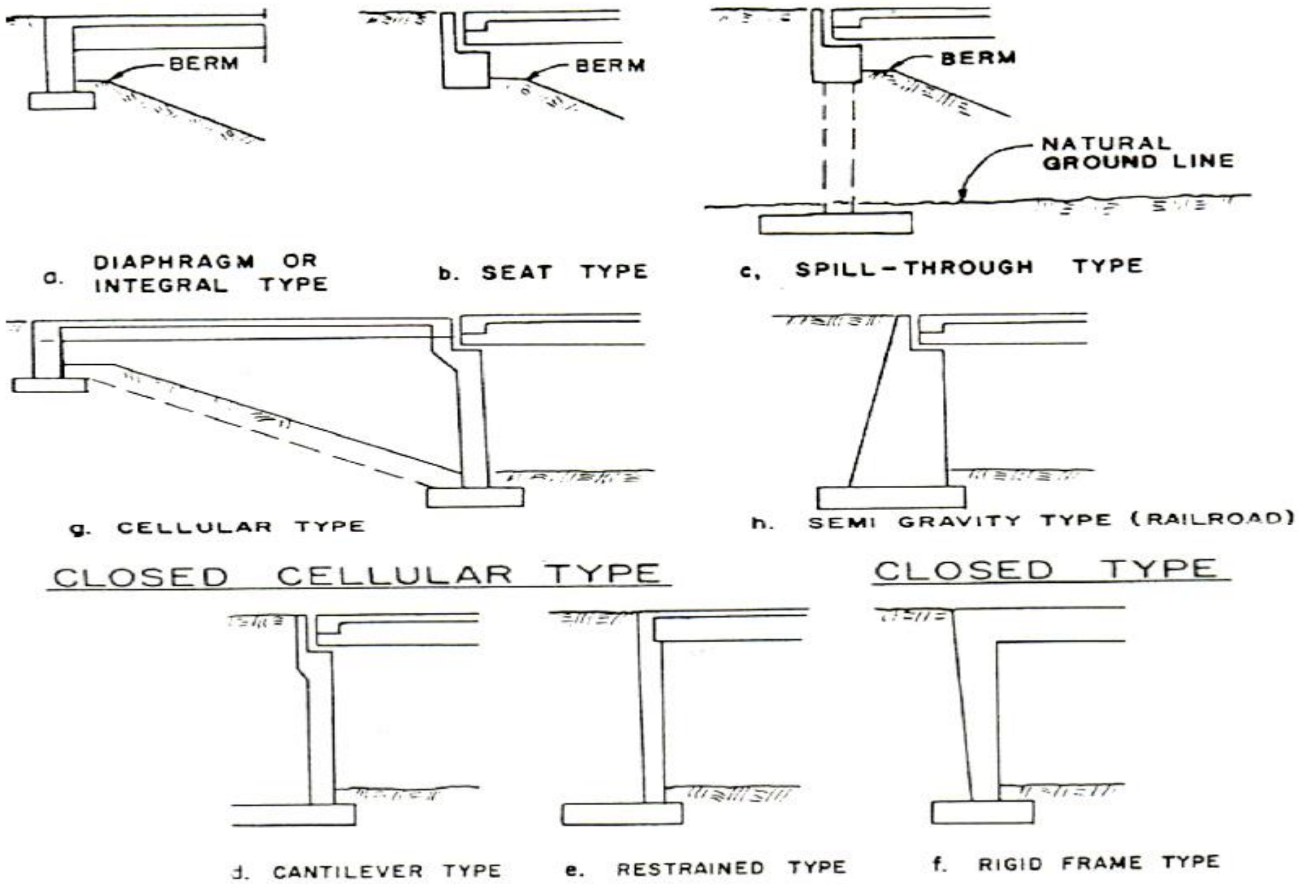


Fig 2.11 Typical Abutments (Anwar, 2015)

H. Forces on Conventional Abutments

According to Anwar (2015), conventional abutment design should take these forces into account:

- Abutment cap self-weight
- Abutment backwall self-weight
- Wingwall of Abutment self-weight
- Different dead loads (bearing seats, horizontal limitations, and so forth.)
- Dead load on the upper structure (girders, cross frames, deck, barrier rails, medians, overlays, and provisions for future overlays, among other things)
- Approach slab dead load.
- The live load on the structure.
- Approach slab with live load.
- Pressure exerted by the lateral soil on the backwall
- pressure exerted by lateral soil on the wingwalls
- live load surcharge.
- Longitudinal forces (in some instances, based on the bearings' nature or the integral connection between the superstructure and the abutment).
- Seismic loads

I. Abutment Design to BD 30 and EN 1997-1

➢ *Earth Pressures*

- Active earth pressures ($K_a\gamma h$) are considered to ensure that the abutment is stable.
- At rest earth pressures ($K_o\gamma h$) are considered to ensure that the structural elements are adequate.
- Passive earth pressures ($K_p\gamma h$) are only considered for integral abutments or where shear keys are provided.

According to Childs (2015), tensions begin to develop on the back of the abutment wall at the beginning of development and while the refill material is compacting. Therefore, the structural parts must be designed to withstand these forces. Any rotational movements or deflection of the structure caused by the pressure at rest will reduce the pressure on the back of the wall. A state of equilibrium is reached when the passive pressure drops to the value of the active earth pressure. As a result, the structure's stability can be evaluated using active earth pressures. As the structure rests up against the soil, passive pressures are initiated, the abutment foundation is designed not to slide, and for this to take place, active pressure will be more than the passive pressure. Naturally, dynamics needed to initiate passive pressure to occur is greater than active pressure. Hence, abutment will be designed to attain stability on its own without having earth in front of its footing. If shear keys are required to prevent sliding, they are used with a factored value of passive pressure, under the base's rear half, the key should be. According to Childs (2015), integral bridges are subjected to passive pressures on the back of the abutment wall when the deck expands.

➢ *Abutment Construction*

The rules of backfilled cantilever retaining walls with spread footings or piled foundations is outlined in Departmental Standard BD 30. The format of the abutment will have suggestions on the design which ought to be thought of.

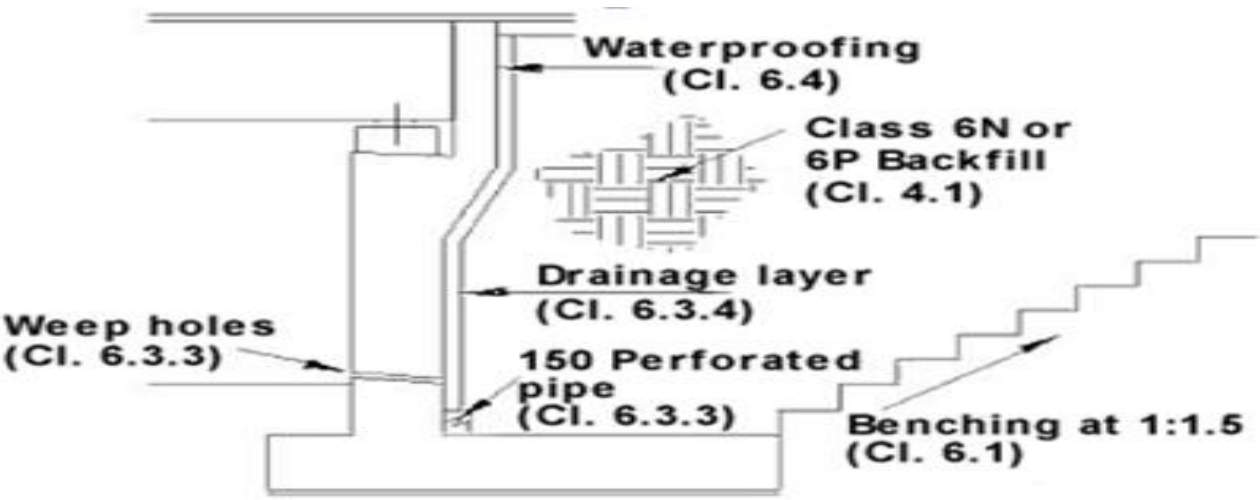


Fig 2.12 Abutment Construction (Departmental Standard BD 30 & Childs, 2015)

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By providing a drainage layer, pore-water pressures can be ignored unless a significant water main rupture is a possibility. However, the drainage layer separates the backfill soil from the wall, there should not be friction at the back of the wall. Any longitudinal friction consequences for the rear of the wall will likewise be impacted by traffic vibration. To keep ice activity from weakening the establishment material, the level of the foundation is normally set somewhere around one meter beneath the soil level. If administrations, such as power lines, water mains, gas pipelines, and so on, may be put in front of the wall of abutment. However, the depth to the foundation level may need to be increased in order for the services to be installed above the concrete footing.

➢ *Abutment Loading*

Through the bearings, load from the deck is transferred to the abutment. Based on the deck analysis, the maximum vertical bearing loads and the kind of restraint required to support the deck will determine the bearing type.

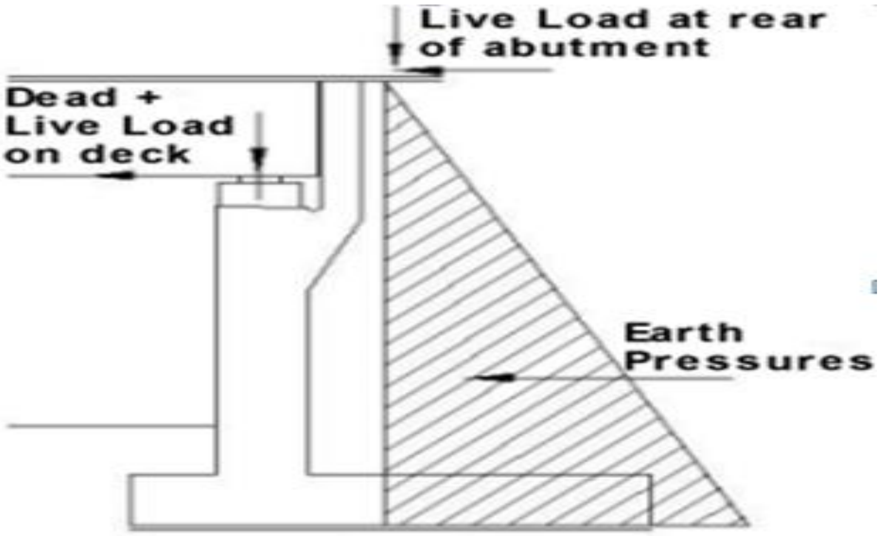

Fig 2.13 Abutment Loading (Childs, 2015)

The sources of horizontal loads that come from the deck are wind load, temperature effects, creep movements, traction, braking and skidding loads, collision loads when high level containment parapets are used, centrifugal loads when using BS 5400-2 if the horizontal radius of curvature of the carriageway is less than 1000 metres, and centrifugal loads carriageway is less than 1500 metres when using EN 1991-2. The deck will be subjected to temperature-induced longitudinal loads depending on the bearing used. Elastomeric bearings are effectively 'stuck' set up between the deck soffit and the abutment bearing plinth, with the objective that the bearing requirements to bend when the deck expand and contracts. The longitudinal force of the deformation is influenced by both the magnitude of the movement and the shear stiffness of the bearing. Sliding bearings, on the other hand, produce a longitudinal load that is proportional to the dead (permanent) load reaction and the sliding surface friction coefficient. Contingent upon the bearing's sort and stress, the coefficient of friction (μ) can go from 0.01 to 0.08 (BS 5400 Section 9:1, Tables 2 and 3).

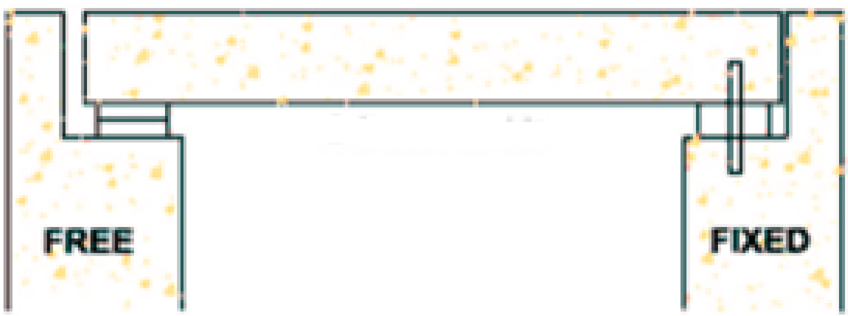

Fig 2.14 Free Abutment with Sliding Bearings (Childs, 2015)

Fig 2.15 Both Abutments with Elastomeric Bearings only (Childs, 2015)

Fig 2.16 Free Abutment with Elastomeric Bearings (Childs, 2015)

Both abutments will be affected equally by the temperature effect's longitudinal load. The friction at the bearing under dead (permanent actions) and superimposed dead loads (variable actions) equals the load transmitted when sliding bearings are used. According to Childs (2015), when elastomeric bearings are utilized, the load carried is proportional to the force required to distort the bearing by the length of time the deck expands or contracts.

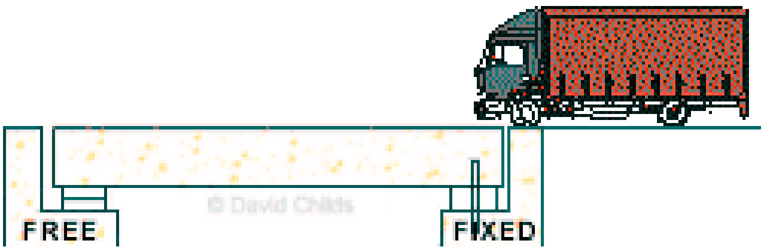

Figure 2.17 Free Abutment with Sliding or Elastomeric Bearings (Childs, 2015)

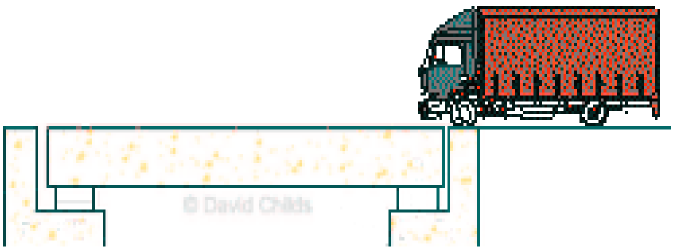

Fig 2.18 Both Abutments with Elastomeric Bearings only (Childs, 2015)

Because the deck is very stiff in the axial direction, horizontal loads won't affect how long the deck is. As a result, it is assumed that the fixed abutment alone carries longitudinal loads caused by skidding, traction, and braking. According to Childs (2012), the loads caused by traction, braking, and skidding are shared between the two abutments when only elastomeric bearings are utilized instead of a fixed one. Only the fixed and sliding-guided bearings will be used to transfer transverse deck loads to the abutment. Although it is unlikely that these loads will affect a full-height abutment's stability, the bearing plinths must be analyzed to withstand the loads. Small abutments like bank seats' stability may need to be checked for these loads. A surcharge load is a representation of the live load at the back of the abutment (PD 6694-1:2011 clause 7.6) or BS 5400 Part 2:2006 clause 5.8.2. When utilizing EN 1991-2:2003, traction, braking, and skidding loads at the rear of the abutment are not required to be taken into consideration (clause 4.9.2). The curtain wall (likewise called up stand wall) does anyway expect to be intended for slowing down forces. Because they are assumed to have sufficient mass to resist the collision loads for global purposes (BD 60/04 clause 2.2 or NA to BS EN 1991-1-7:2006 clause NA.2.13), vehicle collisions on abutments do not normally require consideration.

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➢ *Abutment Stability*

Stability of the abutment is determined by considering:

- Sliding
- Overturning
- Failure of the foundation soil
- Slip failure of the surrounding soil.

A comprehensive Ground Investigation Report is required for the structural design of the bridge. Expected to give data with respect to the events happening underneath the ground of the foundations, holes are bored. In addition, adequate sampling and testing were required to obtain the foundation-level friction and cohesion values of the soil as well as the design parameters for the allowable bearing pressures. Utilizing BD 30, the effects of sliding and overturning are determined using nominal load and dynamic earth pressures. A factor of safety of 2.0 is used to guarantee the abutment's stability against sliding and overturning. When adhering to EN 1997-1:2004, serviceability and ultimate limit states prerequisite for stability. A couple of load cases hope to be considered to ensure all loading conditions are obliged. Abutment wall will be backfilled to the bearing shelf level when constructed, this will give access for pavement construction, while compacting the backfill materials by the heavy compacting equipment, a surcharge load is subjected on its wall. this surcharge load will cause critical load case, due to the fact that the abutment active earth pressure initiated by the action of the machinery compacting the backfill material, has no counter load which should have been provided by the pavement permanent load. The ground survey gives the allowable bearing pressure of the soil, which help to evaluate the settlement limit of the structure, which is between 20 to 25mm. According to PD 6694-1:2011 Condition 5.2.2, the maximum SLS pressure under the foundation can be limited to a small portion (one third) of the ground strength. It is advisable to determine the allowable pressure which will assist in depth and size of the footings. It is imperative to assume loads and footing sizes, to know if there will be redesigning until a stable value for the allowable pressure, loads and footing size is achieved (Childs, 2015). According to BS 8002, the following soil locations may experience a slip failure:

- The wall is constructed on sloping ground that is close to limiting equilibrium in and of itself or,
- The structure is underlain by strata within which high pore-water pressures may develop from natural or artificial sources, or,
- The strata are founded on a relatively strong stratum underlain by weaker strata, or
- The structure is underlain by a significant depth of clay whose undrained strength increases only gradually with depth.

If none of these considerations are present, then a slip failure analysis will not be necessary.

J. Piers (Bents)

The term "pier" or "bent" refers to interior span that are located away from the end upholds. To maintain consistency, the term "pier" will be used throughout this module. It is expected to concern any structural member that carries the superstructure at moderate intervals between end upholds. According to Patel's definition, piers are substructural components that are situated at the ends of bridge spans at intermediate points between abutments. The usage of piers is to transmit the superstructure vertical weights to the foundation and to go against all longitudinal and lateral forces on the bridge. Piers are typically built with reinforced or masonry concrete. Contingent upon the kind, size, and aspects of the superstructure, they take on different structures and add to the extension construction's stylish allure.

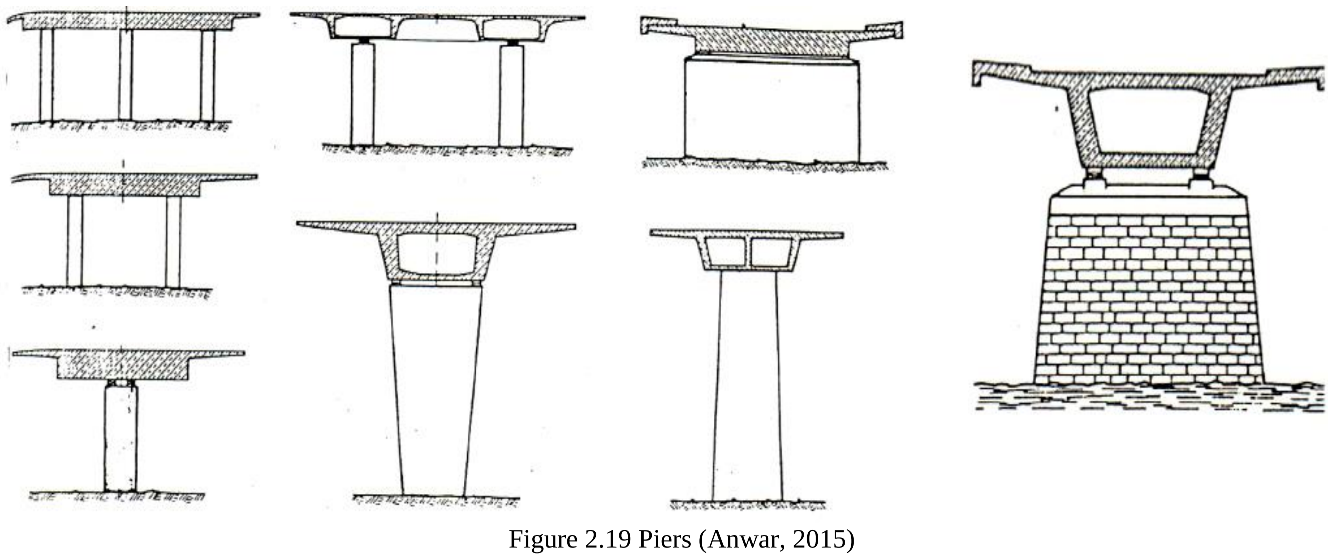

Figure 2.19 Piers (Anwar, 2015)

K. Pier Types

Piers come in all shapes and sizes. Instead of trying to list each possible combination individually, it is easier to classify different types of piers using a few fundamental pier elements. The following are the fundamental components of a pier:

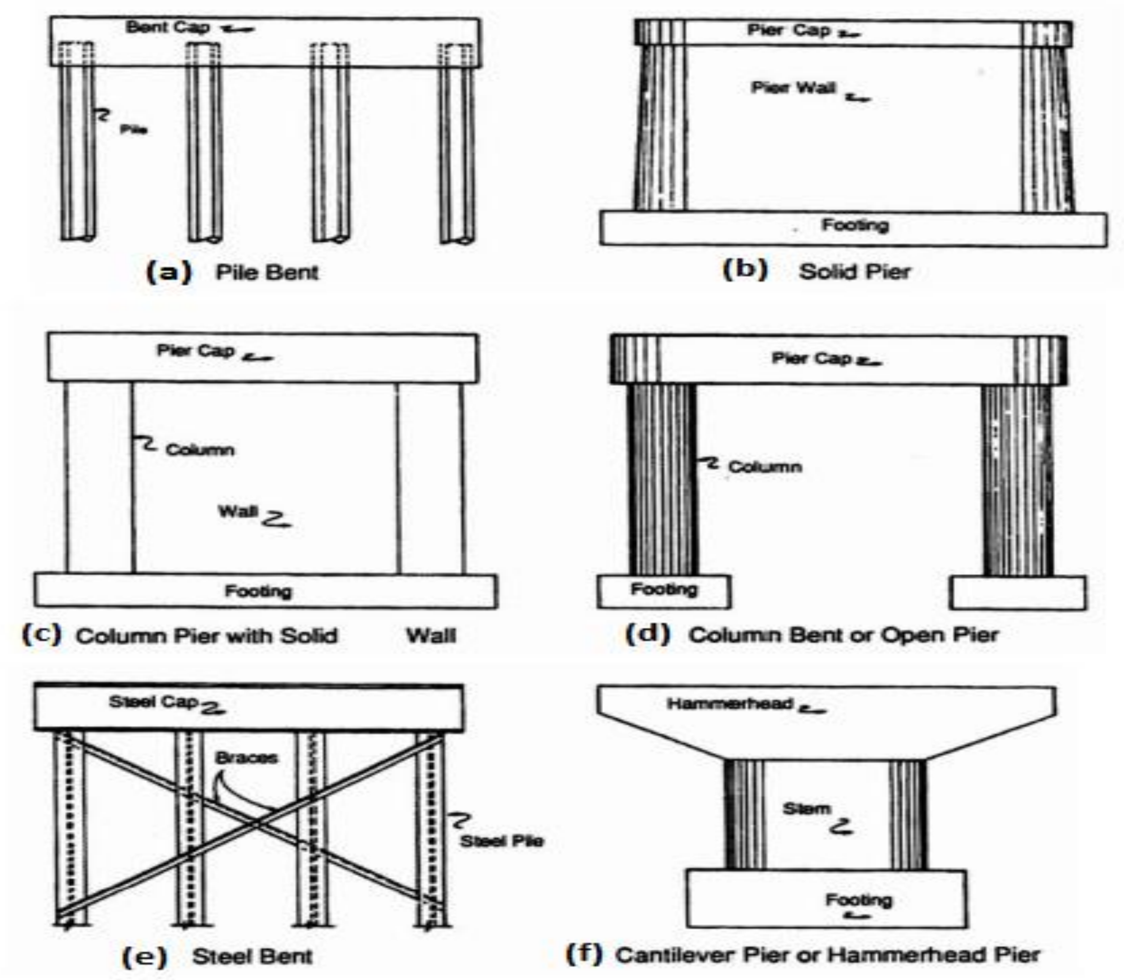

Fig 2.20 Pier Bent types (Anwar, 2015)

- *Pier Caps:*

A member that is roughly horizontal and supports the superstructure. The pier cap and the pier lateral support(s) are practically identical in some instances, such as wall piers. The cross sections of pier caps can be square, rectangular, "inverted T," or any other shape. They could be stepped, prism like, etc., yet most of them are concrete cast-in-situ, however, utilized and are determining more extensive acknowledgment all through the US (30), is the precast piers caps.

- *Pier Vertical Supports:*

Any part that is pretty much lateral and lays on the pier footing and supports the pier cap. Columns, which are lateral members whose cross-sectional dimensions are significantly smaller than the longitudinal dimensions of the pier cap, make up the majority of the pier's lateral supports. At times, lateral support for the pier comprises of a solitary "wall," or part with cross-sectional aspects that are almost indistinguishable from the pier cap's level aspects. For pier vertical supports, whether they are walls or columns, cross-sectional shapes of round columns, square columns, rectangular columns, and so on are all possible. Prismatic, tapered, stepped, or flared columns can be either solid or hollow.

- *Pier Intermediate Struts:*

The buckling capacity of the columns are strengthened by longitudinal individual supports.

- *Pier Intermediate Bracing:*

Any kind of bracing, including X-bracing, web walls, which are concrete shear walls between columns, that accomplishes both of the following: a) bracing the columns to make them more capable to resist buckling, and b) making a better shear load path for carrying longitudinal forces through the pier.

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- Steel, concrete, or, less common cases, timber or masonry can be used to make any of the aforementioned components. The elements in steel can be in rolled sections, built-up open plate sections, built-up closed box sections, trusses, latticework, or any other shape. On account of concrete, the components might be cast-in-situ, precast, prestressed or post-tensioned, or both. Additionally, on account of concrete, the components might be either projected set up, or precast.

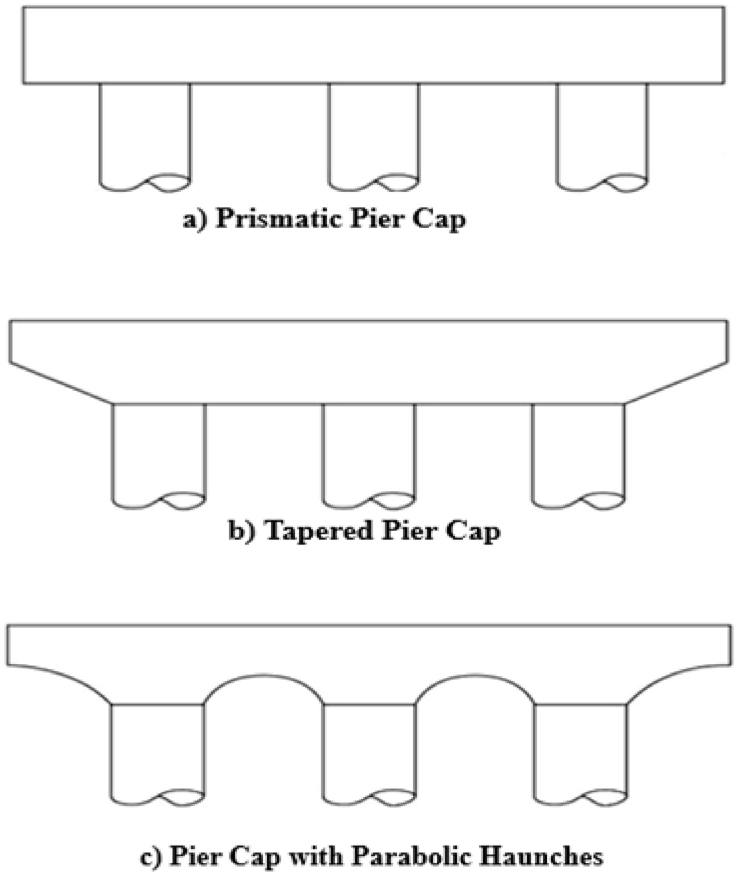

Fig 2.21 Different Types of Pier Caps for Multi-Column Piers (FHWA, 2012)

The following is a list of some of the most common combinations of these elements, but this list should not be considered exhaustive or restrictive (FHWA, 2012):

- *Reinforced Concrete Multi-Column Piers:*

A reinforced concrete pier cap is supported by two or more reinforced concrete columns, making this type of pier possibly the most prevalent. For the most part, the pier cap is just expectedly built up, yet periodically post-tensioning is utilized too. For the most part, the section not entirely settled to fulfill a harmony between sparing design of the element and of the segments, though decorative restraints might control the system in particular cases.

- *Reinforced Concrete Single Column Piers:*

Due to its resemblance to a hammer, this type sometimes goes by the name "hammerhead pier." It has a reinforced concrete pier cap called a "hammer head" that is supported by a single reinforced concrete column. Post-tensioning is used more frequently in hammerhead pier caps than in multi-column pier caps. This type of pier is particularly popular for tall piers where a single, much larger column may provide a more effective means of resisting column buckling and narrow bridges where there is not enough room for two or more columns.

- *Pile Bents:*

A "pile bent" is a pier cap supported on multiple steel or precast concrete piles. In most cases, there is no clear difference between the "columns" and "foundations" in a pile bent; the foundations are simply extensions of the piles that are supporting the cap.

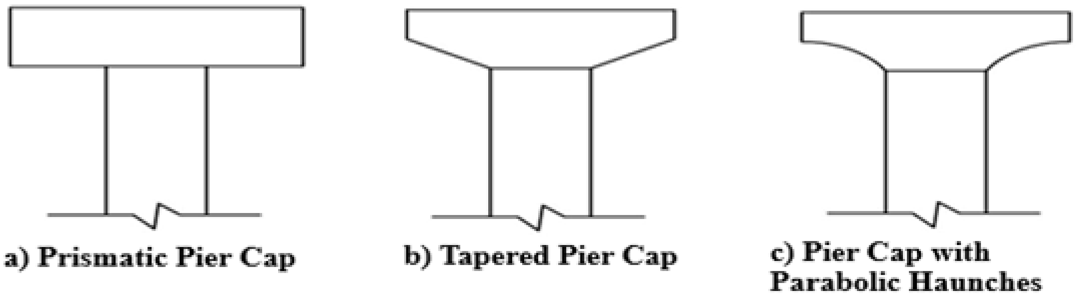

Fig 2.22 Different Types of Pier Caps for Single Column Piers (FHWA, 2012)

- *Straddle Bents:*

Straddle bent is a pier with many columns, that the spacing is majorly wide which allowed for passage of a roadway directly below the pier, such that the pier is "straddling" the roadway below. Due to the remarkably wide column spacing and the long span of the pier cap, straddle bent pier caps are often special structures such as steel box beam caps or post tensioned concrete caps.

- *Integral Piers:*

The construction of pier caps that are integrated with the superstructure is sometimes desirable. Sometimes this gives advantages in terms of structural efficiency, sometimes it gives aesthetic benefits, and sometimes it assists to reduce structure depth and improve lateral clearances. Integral pier caps for steel girder bridges have been constructed using both steel (Wassef, 2004) and concrete (FHWA, 1997).

- *Steel Piers:*

Even though most of the piers are made of reinforced, prestressed or post-tensioned concrete, there are still cases demanding the use of steel for part or all of a pier. One common chance to make use of steel piers is for temporary bridge structures, such as temporary construction sites bridges. In these instances, contractors often prefer using steel substructures =because they have light weight and easy to manipulate, comparatively quick to install, and possibly reusable. Steel elements can also be used in straddle bent caps and integral pier caps owing to its long span, Steel piers are perhaps used less frequently, but can render good solutions in the right context. For instance, various steel girder bridges have been built with integral slant-leg steel piers or steel delta-piers to provide solution to tough design problems in challenging sites. Furthermore, steel pipe piles filled with concrete can be used to give many values for both systems when rightfully applied.

L. Procedure for Analysis of Pier

The pier is analyzed taking into account a number of factors, including loads transferred by the superstructure and forces acting directly on the pier. A pier must resist the following loads and forces, according to Patel (2008):

- Dead Load: Dead load of superstructure and substructure above the base level of pier.
- Live load: This consist of live load of traffic passing over the bridge
- Buoyancy
- Wind Load
- Horizontal forces due to water current: Horizontal forces due to water current are considered on that part of substructure that lies between the water level and the base of pier. The water current pressure is given as

$$P = KV^2 \qquad (2.10)$$

Where

$P =$ intensity of pressure in kN/m^2 due to water level and the base of the pier
$K =$ a constant having different values for different shapes of piers

The values of this constant for different pier shapes are present in Fig 2.3

V is the current velocity in meters per second at the location where the pressure intensity is calculated. It is accepted that the speed circulation in stream is to such an extent that V2 is greatest at the free surface of water, zero at the most profound scour level and fluctuates in the middle between them. Additionally, it is assumed that the flow's maximum velocity is $\sqrt{2}$ times the current's velocity.

{See Fig 2.3 Value of constant k for pressure Intensity due to water current (Patel, 2008) page 1 Tables for Research work}.

The angle that the current has with the pier's axis should be taken into consideration. Typically, the maximum angle variation of water current, to the transverse axis of the bridge is taken as 20^0

Hence, the pressure along the pivot of the pier and transverse to it is respectively given by,

$$P_1 = KV^2\cos^2 20^0 \tag{2.11}$$

$P_2 = KV^2\sin^2 20^0$(2.12

M. Bridge Bearings

Bearings connect the superstructure with the substructure allowing for movements, yet transfer forces

- **Elastomeric:** Made from rubber and steel plates
- **Guided Sliding:** Steel on steel, Teflon coated (polytetrafluoroethylene PTFE)
- **Pot:** Made from steel. Rubber filled
- **Rocker:** Steel arm for large movement
- **Pin or Hinge:** Mechanical Pin made in steel
- **Roller:** Mechanical Roller

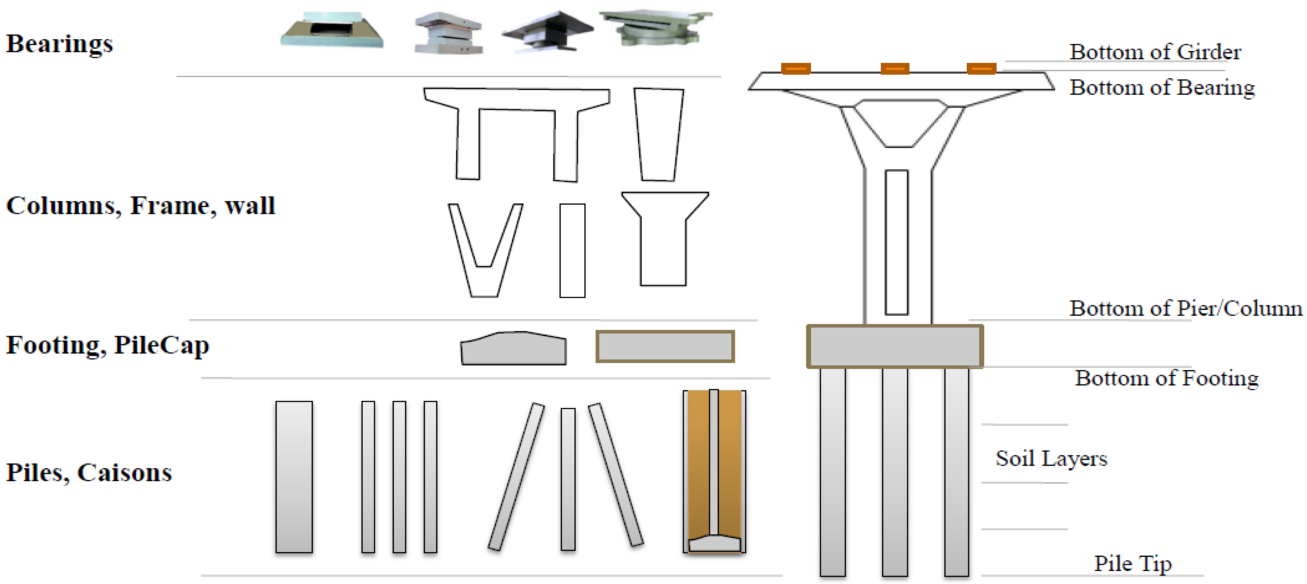

Fig 2.25 Subcomponents of Typical Bents (Anwar, 2015)

N. Software Availability

There are many different kinds of software that can design some or all of a substructure. The design of a bridge substructure can be completed in its entirety with the help of some commercial software packages. These kinds of programs can fabricate the base shapes, assemble the superstructure calculation, ascertain loads on the base, perform internal load stresses of the pier caps, columns, and foundations, and afterward sizes of various components, as well carryout detailed design analysis. Nevertheless, mostly, these programs have some limitations on the complexity of designs they can handle, such as complex loading, superstructure types, and complex geometry etc. Designers are encouraged to fully comprehend these "all in one" substructure design programs' capabilities and, more importantly, their limitations, according to FHWA (2012). A combination of general FEM analysis models and specific design software can also be used to design substructures. Regularly, designers will do the load calculations manually, then input these load values into frame model of the abutment or pier (or into a more complicated model if necessary), run the model to decide the internal load stresses, distribute loads and do detailed design checks making use of commercial or home-made software, coding etc. or blending of more than one.

CHAPTER THREE
METHODOLOGY

A. Design Information and Assumptions

The typical bridge section is shown in Figure 3.1

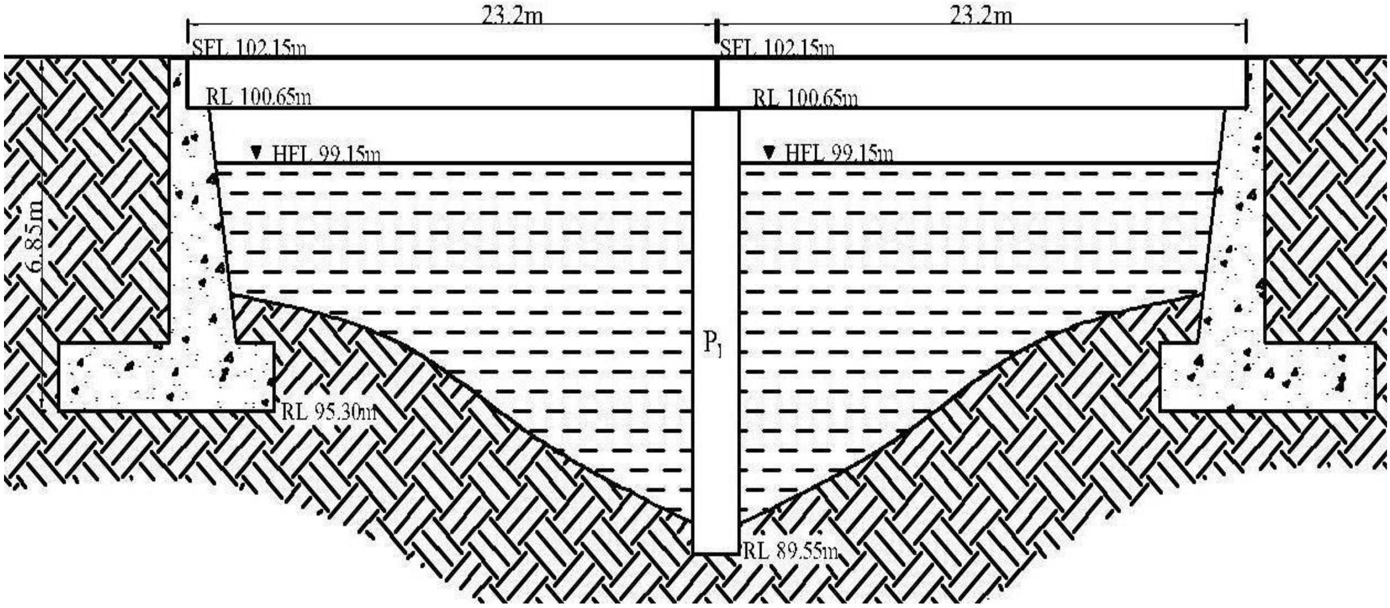

Fig 3.1 Typical Bridge Section

Design Code	: BS EN 1991-2-2003
Bearing pressure under abutment	: 670 kN/m^2
Bearing pressure under pier, P_1	: 510 kN/m^2
Deck slab	: 100 mm
Wearing course	:50 mm

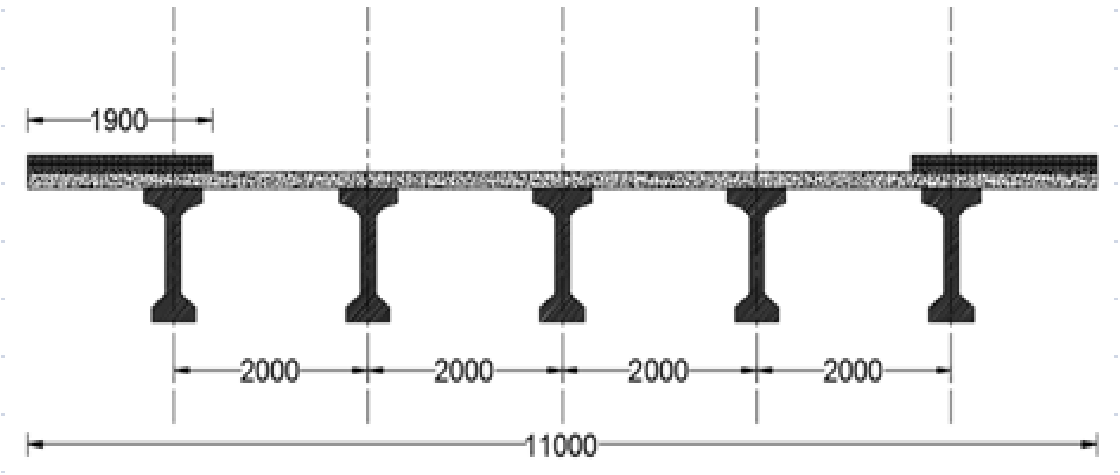

Fig 3.2 Cross Section of Bridge Deck Diaphragms (Cross-Girders) Arrangement

- Assume walk way kerb height as 150mm
- Side railing weight as 1kN/m
- Adjustment factors of α consider as 1
- Density of concrete is 25kN/m^3
- Density of wearing surface and pavement materials as 24kN/m^3

Thermal and wind actions along with creep & shrinkage of PC girders are not considered for load combinations. Thus, effects of displacement and deformations are assumed to be negligible.

➢ *Load Calculations*

- *Permanent Actions*

Prestressed concrete(PC) Girderfrom section area $= 0.3619348 \times 25 = 9.05\text{kN/m}$
Cross Diaphragm $= 0.6 \times 1.2 \times 25 = 18\text{kN/m}$
Deck Slab $= 25 \times 0.1 = 2.5\text{ kN/m}$
Wearing surface $= 24 \times 0.05 = 1.2\text{ kN/m}$
Walkway finishes including kerbs were assume = 175 mm height
Walkway finishes $= 24 \times 0.175 = 4.2\text{ kN/m}$
Handrail $= 1\text{ kN/m}$

- *Variable Actions*

Walk way $= 3\text{ KN/m}^2$ (BS EN 1991 − 2 − 2003)
Carriage way(BS EN 1991 − 2 − 2003)
Total width $= 11\text{m}$
Carriage way width $= 7.2\text{ m}$

Number of notional lane, $n_1 = Int\left(\frac{w}{3}\right) = 2$ m,

The width remaining area of the carriage way $= 7.2 - 2 \times 3 = 1.2$ m (BS EN 1991-2-2003).

{See Table 3.1 Load Model 1: Characteristic Values (BS EN 1991-2-2003) page 1 Tables for Research work}.

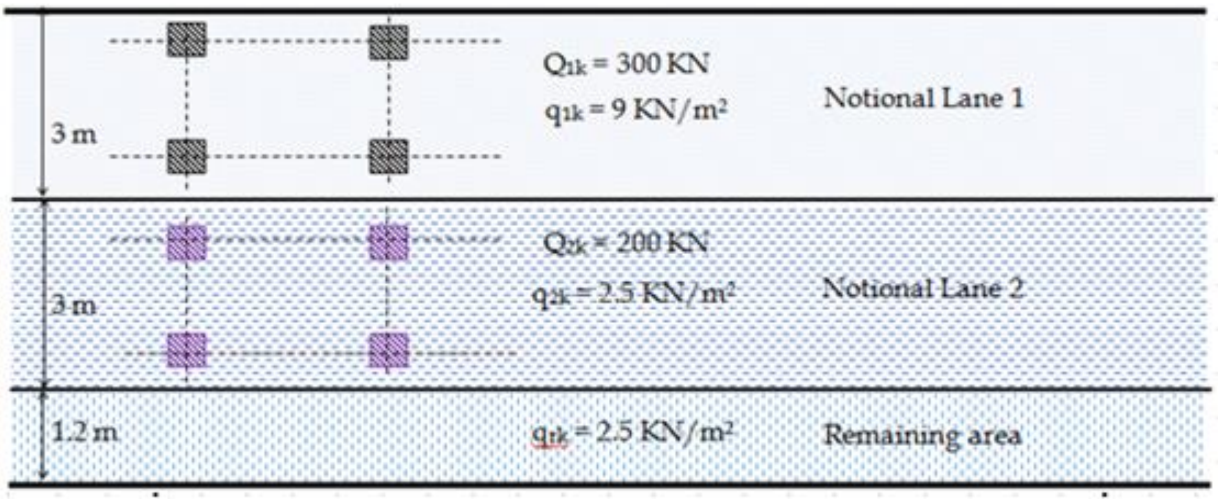

Fig 3.3 Application of Load Model 1, on a Deck with Two Notional Lanes

➢ *Load Calculations of Girders of Span 23.2m*
Consider the applicability of Courbon's Method

$\frac{\text{Span}}{\text{width}} = \frac{23.2}{11} = 2.11 > 2$. Hence, it is adequate.

Depth of longitudinal girders = 1362.5 mm.
Depth of cross girders $= 1050 > 0.75 \times$ depth of longitudinal girders. Hence, it is adequate.

Longitudinal beams are interconnected by symmetrically spaced cross girders of adequate stiffness, according to Courbon's Method, getting higher results on most end girders. So, girder "A" was selected for critical loading condition (Fig 3.4).

$$R_i = \frac{P}{n} \pm \frac{P_e}{I} \tag{3.1}$$

Where;

R_i = Reaction factor; P = Applied load; n = Number of girders;
e = Eccentricity of load; x = Distance under consideration.

But, $I = \frac{\sum x^2}{x}$

$$\text{Then, } R_i = P\left(\frac{1}{n} \pm \frac{ex}{\sum x^2}\right) \tag{3.2}$$

where; $\sum X^2 = (2^2 + 4^2) \times 2 = 40$
and, $n = 5$

Therefore, $R_i = P(0.2 + 0.1e)$. (from Equation 3.2)

X is positive for the considered girder, with the direction from center line. Assume unit load, P = 1, by substituting above values with different x values for Girder "A" (Fig 3.4).

At $e = 0$, $R_i = 0.2$ (from Equation 3.2)
At $e = 3.6$, $R_i = 0.56$ (from Equation 3.2)
At $e = -2$, $R_i = 0$ (from Equation 3.2)
At $e = -3.6$, $R_i = -0.16$ (from Equation 3.2)

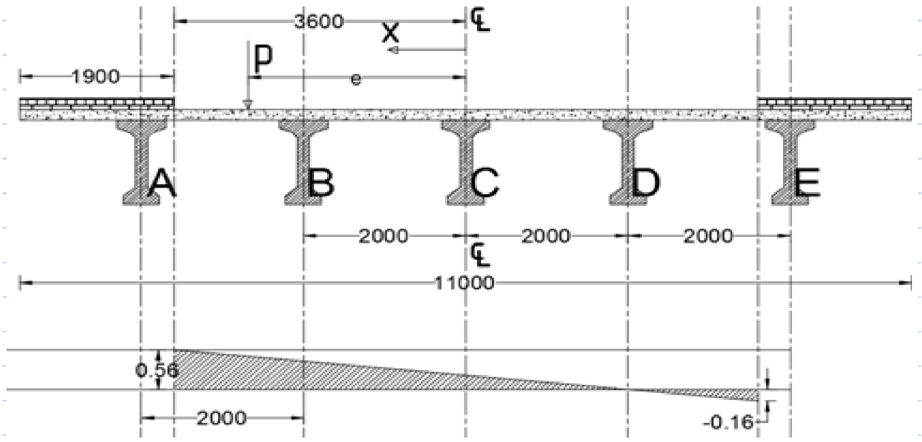

Fig 3.4 Influence Line Diagram of Diaphragms (Cross girders)

As indicated by the code, pair frameworks (TS) on contiguous notional paths are considered, they might be brought nearer, with a distance between wheel axles not underneath 0.5m (BS EN 1991-2:2003 Condition 4.3.2 (5)). Each tandem system is replaced in each lane by a one-axle concentrated load of equal weight to the combined weight of the two axles if the span is greater than 10 meters (BS EN 1991-2:2003 Clause 4.3.2(6)).

The following loading arrangement shall be considered for the Courbon's Method(Fig 3.5).

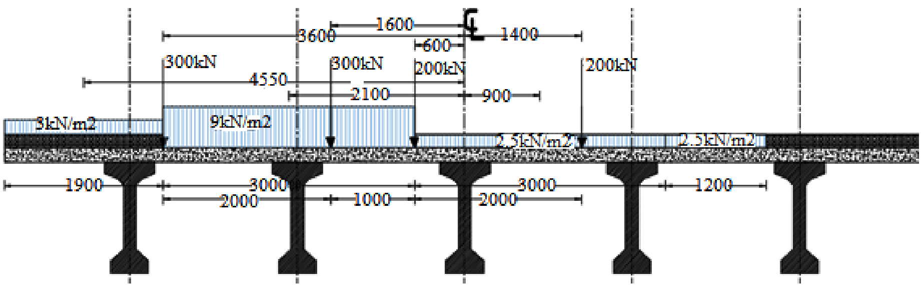

Fig 3.5 Load Model 1: Diaphragms (Cross Girders) Arrangement

With reference to Figure 3.5, the load center distance shall be govern effectively byloading arrangement on girder "A"(Fig 3.1). {See Table 3.2 Resultant variable actions on selected girder 'A', page 2 Tables for Research work}. From Table 3.2, total resultant variable action on selected Girder "A" is as follow;

Point Load $= 168 + 108 + 52 + 12 = 340$ kN (Fig 3.2)
Uniformly distributed load (UDL) $= 11.07 + 0.825 + 3.7335 = 15.6285$ kN/m (Fig 3.2)

Other actions on selected beam with self weight of the structures are going to uniformly distributedon the beam which is proprotion to effective depth. (Fig 4.1).

B. *Structural Components*

The components to be designed include the following; (1) Abutment, (2) Pier.

➢ *Abutment of Span 23.2m*

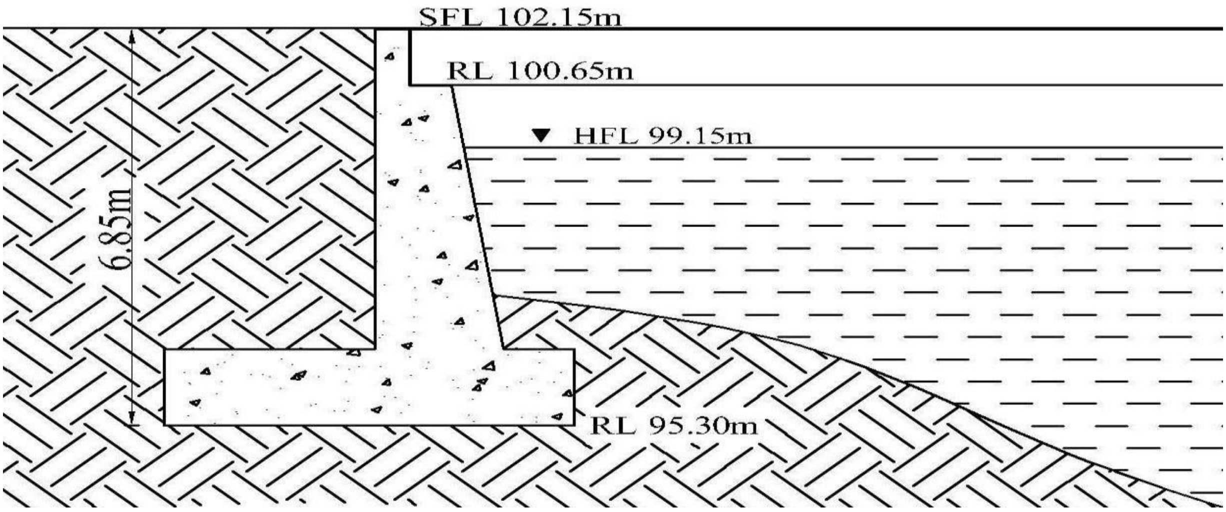

Fig 3.6 Schematic Diagram of the Abutment

- **Legend:** RL = Reduced level; HFL = Hydraulic flow level; SFL = Structural finished level.

Calculation of abutment height is based on the provided reduce levels with manipulated structural finish level based on the girder and slab finish details.

(RL) of the base = 95.30m
(RL) of the girder bottom = 100.65m
Girder height = 1.35m
Slab and wearing corse = 0.15m
(SFL) of road surface = 102.15m
Abutment height = 6.85m
Width of abutement = 11.6m

➢ *Abutment Load Calculations of Span 23.2m*

- *Vertical Load on Abutment from Superstructure of Span 23.2m*

Here, we assume higher loading value for a more safe design on abutment by considering the design load value calculated for the edge beam by Courbon's Method.

Assume pore water presure dissipate along the provided weep holes.

Assume abutment is RCC foundation and place at the rock level with scouring being considered up to the bed.

From shear force calculation of beam:

Self weight of the superstructure = 290 kN/Bearing pad
Self weight of the diapragm structure = 54 kN/ Bearing pad
Total dead Load on the abutment = 344 kN/Bearing pad
Uniform dead load on abutment = 172 kN/m
Load from the pedastrian (Imposed) = 43.31 kN/Bearing pad
Load from the traffic (Imposed) =137.982 kN/Bearing pad
Load from the TS (Imposed) = 170 kN/Bearing pad
Total imposed load on the abutment = 351.292 kN/Bearing pad
Uniform imposed load on abutment = 175.646 kN/m

- *Braking and Acceleration Forces (BS EN 1991-2-2003,Cl 4.4.1(2))*

$Q_{1k} = 0.6 \times \alpha Q_{1_k} \times (2Q_{1k}) + 0.1 \times \alpha q_1 \times q_{1k} \times W_1 \times L$(3.3)

Where;

180 x αQ_1 (kN) $\leq Q_1 k \leq 900$(kN),αQ_1 and $\alpha q_1$are equal to 1.
$Q_{1k} = 0.6 \times 300 \times 2 + 0.1 \times 9 \times 3 \times 23.2 = 422.64$ kN (from Equation 3.3)
$Q_{2k} = 0.6 \times 200 \times 2 + 0.1 \times 2.5 \times 3 \times 23.2 = 257.4$ kN (from Equation 3.3)

Therefore, use $Q_{1k} = 422.64$kN.
Distributing along the abutment, uniform force on abutment $= 36.4$ kN/m.

C. Lateral Pressure on the Wall

Abutment height = 6.85m

$$\text{Lateral pressure coefficient } K_a = \frac{(1 - \text{Sin}\varphi)}{(1 + \text{Sin}\varphi)} \quad (3.4)$$

Assume soil friction angle of $\varphi = 30$(BS EN 1991-2-2003,Cl. 4.9.1)

$$K_a = \frac{(1 - \text{Sin}30)}{(1 + \text{Sin}30)} = 0.33 \qquad \text{(from Equation 3.4)}$$

Base friction coefficient $\mu = \tan 30 = 0.58$

Assume soil density $= 19\ \text{kN/m}^3$ (BS EN 1991-2-2003)

Surcharge, due to non availability of data were considered 20 kN/m^2 (Cl.5.7.1 CIRA).
So, Surcharge acting on wall $= \frac{20 \times 3 \times 2}{11.6} = 10.3\ \text{kN/m}^2$ (PD6694 − 1: 2011).
Line load from surcharge actingon Wall, $H_S = 2 \times 330 \times 0.33 \times \frac{2}{11.6} = 37.6$ kN/m

$$FKN_{lane}/W_{abutment} \quad (3.5)$$

Surcharge pressure on wall, $P_s = 10.3 \times 0.33 \times 6.85 = 23.6$ kN/m
Soil pressure on wall, $P_{soil} = 19 \times 0.33 \times 6.85^2 \times 0.5 = 148.6$ kN/m

Pore water pressure on wall (wasnot required due provision of weep holes).

The following preliminary section parameters were considered for the abutment, (Fig 3.7).

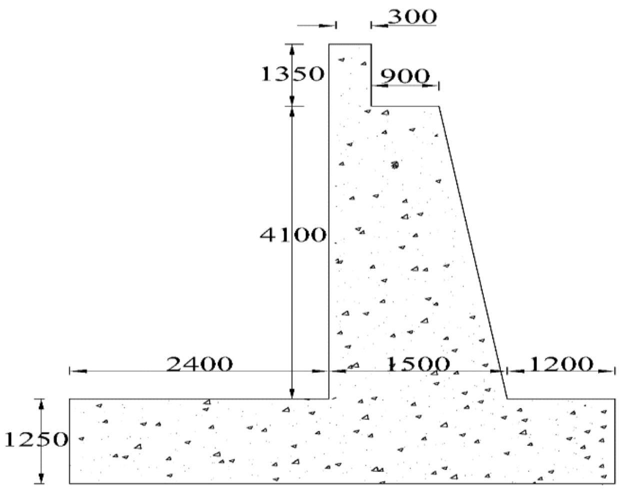

Fig 3.7 Cross Section of Abutment (All dimensions in mm)

Density of concrete = 25 kN/m^3

Abutment weight, $W_1 = (1.25 \times 5.1 + 1.2 \times 4.1 + 4.1 \times 0.5 \times 0.3 + 1.35 \times 0.3) \times 25 = 307.875$ kN

{See Fig 3.2 Center of Gravity, Area of the Abutment, page 2 in Figures for Research work}.

COG of abutment $= \dfrac{34.3861}{12.315} = 2.792$ m from Toe.(Table 3.3)

Earth fill on abutment base $= 2.4 \times 5.45 \times 19 = 248.52$ kN/m

So, with the wall and load configurations, the folllowing dimensional arrangementswere considered for the stability evaluation (Fig3.8).

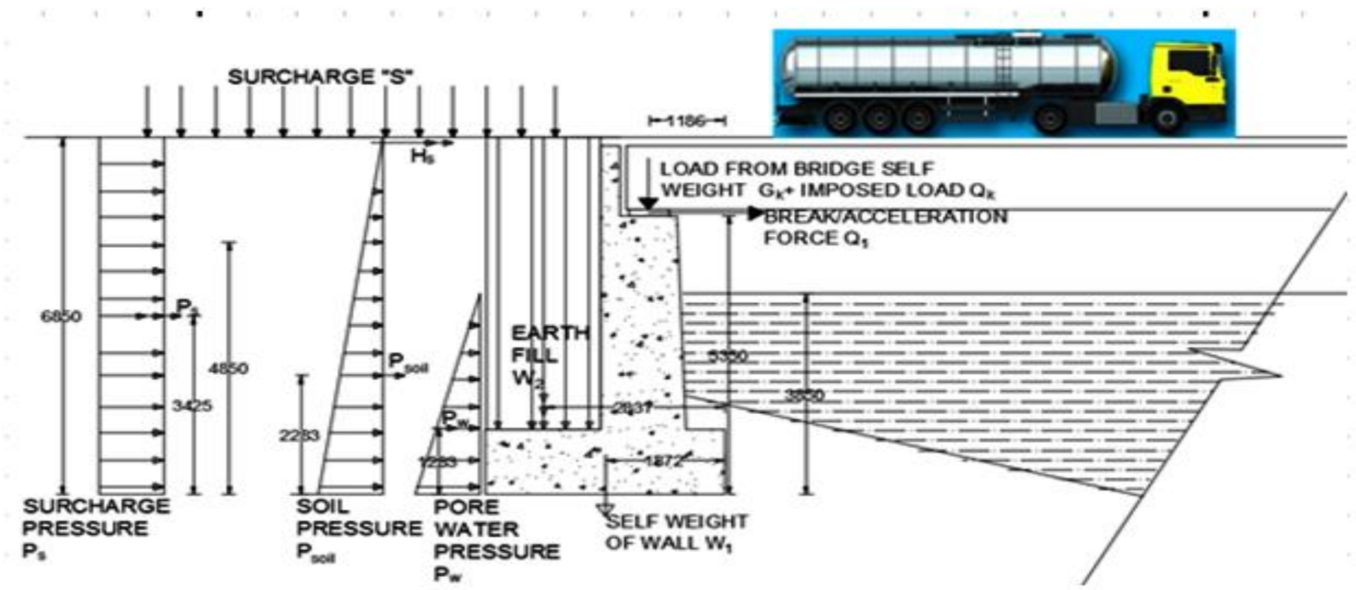

Fig 3.8 Active Forces on the Earth Retaining Wall side of the Abutment.

{See Fig 3.4 Restoring Moment and Stabilizing Forces on the Abutment wall around the Toe of Span 23.2m, page 2 in Fig for Research work}. Fig 3.4 shows the analysis of figures 3.7 and 3.8.

{See Fig 3.5 Overturning Moment and Destabilizing Forces on the Abutment wall around the Toe of Span 23.2m, page 3 in figures for Research work}. Fig 3.5, was gotten from the result analysis of sections 3.5, 3.5.1 and figure 3.8.

{See Fig 3.6 Axial Forces on the Abutment of Span 23.2m, page 3 in figures for Research work}. For axal forces on the abutment wall, lever arm is measured from wall center to load center, horizontally (lateral force on wall).

{See Fig 3.7 Shear Force on Abutment of Span 23.2m, page 3 in figures for Research work}. Shear forces on the abutment wall,lever arm is measured from base top level to load center vertically.

- *Pier on Span of 23.2m*

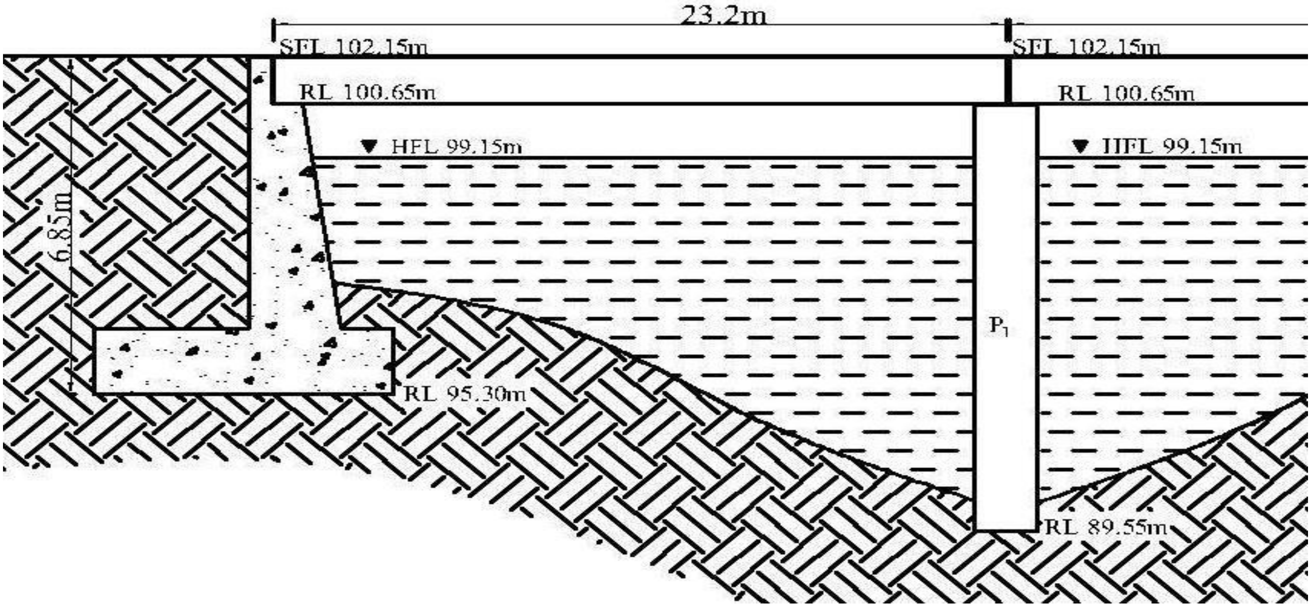

Fig 3.9 Pier Position on Bridge Span of 23.2m

Calculation of pier height is based on the provided reduce levels with maniputlated structural finish level based on the girder and slab finishing details (Fig 3.10).

RL of the base = 89.55 m
RL of girder bottom = 100.65 m
Girder height = 1.35 m
Slab and wearing course =0.15 m
SFL of road surface =102.15 m
Pier height = 11.1m
Width of pier cap = 11.6 m

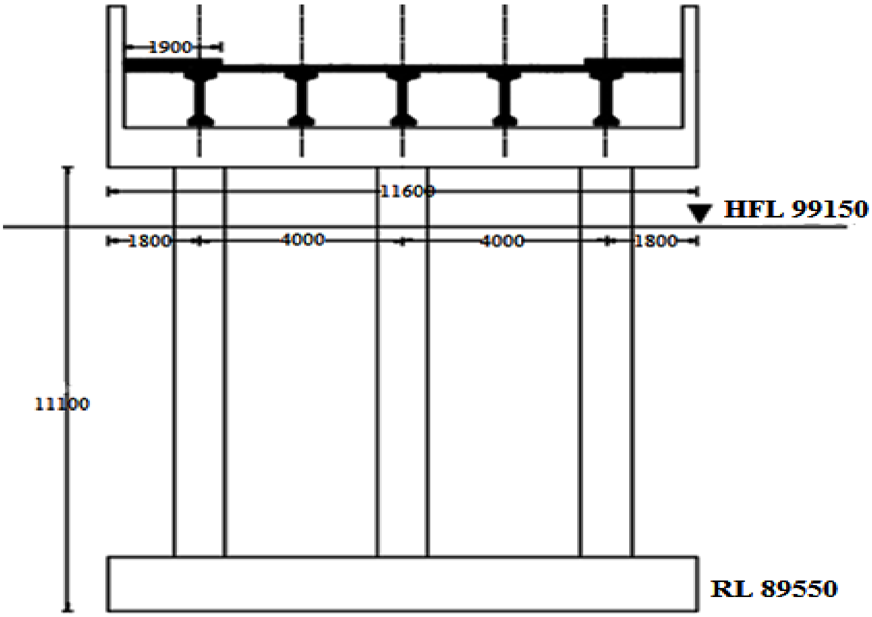

Fig 3.10 Typical Pier Cross Section (All dimensions in mm)

ISSN No:-2456-2165

➢ *Load Calculation for Pier of Span 23.2m*

- *Vertical Load on Pier from Superstructure of Span 23.2m*

Here, assume higher loading value for a more safe design by considering the design load value calculated for the edge beam by Courbon's Method.

Considering traction and breaking forces on piers,water current and log impacts on pier and wind force on the bridge and assumming pier foundation is RCC pad placed at the rock level with scouring considered up to the rock.

From the shear force calculations of beam of bridge deck;

Self weight of the superstructure =508 kN/ Bearing Pad
Self weight of the diaphragm structure =108 kN/ Bearing Pad
Total dead load on the abutment = 688 kN/Bearing pad
Uniform dead load on abutment =344 kN/m
Load from the pedestrain (Imposed) = 86.62 kN/Bearing pad
Load from the traffic (Imposed) = 275.96 kN/Bearing pad
Load from the TS (Imposed) = 340 kN/Bearing pad
Load imposed oad on the abutment = 702.582 kN/Bearing pad
Uniform imposed load on abutment = 351.292 kN/m

D. Braking and Acceleration Forces

Since one end of the girders dowel on the pier and other side girders act as rollers on the pier. So, traction and breaking forces will be applicable only at one side.

$$Q_{1k} = 0.6 \times 300 \times 2 + 0.1 \times 9 \times 3 \times 23.2 = 422.64 \text{ kN} \quad \text{(from Equation 3.3)}$$

$$Q_{2k} = 0.6 \times 200 \times 2 + 0.1 \times 2.5 \times 3 \times 23.2 = 257.4 \text{ kN} \quad \text{(from Equation 3.3)}$$

Therefore, use $Q_{1k} = 422.64$ kN and distribute along the pier beam.

Thus, uniform force on abutement=36.4 kN/m.

Collision force on the pier = 1000 kN along the bridge or 500 kN perpendicular to the bridge(BS EN 1991-2-2003, Cl. 4.7.2.1).For the calculation, Load Case 2 full load condition was considered.

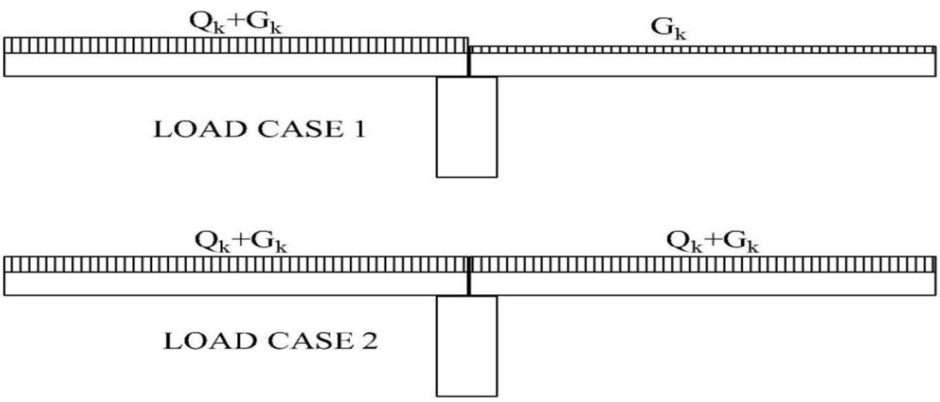

Fig 3.11 Collision Forces on Bridge Deck

➢ *Wind Load Calculation for the Bridge (EN 1991-1-4:2005+A1:2010)*

Wind presure calcualtion according to the "Eurocode 1: Actions on structures - General actions - Part 1-4: Wind actions"

Assume wind speed is $V_{b,0} = 22 \text{ ms}^{-1}$ for Zone 3 (National Annex (Cl: 4.3))

- *Basic Wind Speed*

$$V_b = C_{dir} \times C_{season} \times V_{b,0} \text{ (EN 1991-1-4:2005+A1:2010, Cl: 4.2.2)} \quad (3.7)$$

ISSN No:-2456-2165

Where:

C_{dir} Directional Factor $= 1$
C_{season} Seasonal Factor $= 1$
$\therefore V_b = 22\ ms^{-1}$

- *Mean Wind Velocity*

$$V_{m(z)} = C_{r(z)} \times C_{0(z)} \times V_b \text{(EN 1991-1-4:2005+A1:2010, Cl: 4.3.1)} \quad (3.8)$$

(EN 1991-1-4:2005+A1:2010)

Assume terrain category is 0

Roughness length, $Z_0 = 0.003$ m
Minimum height, $Z_{min} = 1.0$ m
Maximum height, $Z_{max} = 200.0$ m
Orography factor, $C_{0\,(z)} = 1$ due flat terrain
Roughness factor, $C_{r\,(z)} = k_r \text{In}\,(Z/Z_0)$
$k_r = 0.19 \text{In}\,(Z_0/Z_{0,ii})^{0.07} = 0.15604$

- *Peak Velocity Pressure*

$$q_{p(z)} = C_{e(z)} \times q_b \quad \text{(EN 1991-1-4:2005+A1:2010, Cl: 4.5.1)} \quad (3.9)$$

$$q_b = \frac{1}{2} \rho \times V_b^2 \quad \text{(EN 1991-1-4:2005+A1:2010, Cl: 4.5.1)} \quad (3.10)$$

$\rho = 1.25\ kg/m^2$ (EN 1991-1-4:2005+A1:2010, Cl: 4.5.1)
$q_b = 0.5 \times 1.25 \times 484 = 3.025\ kN/m^2$ (from Equation 3.10)
(EN 1991-1-4:2005+A1:2010, Cl: 4.4.1)
Standard deviation of turbulance, $\sigma_v = K_r \times V_b \times K_t$ (3.11)

Where $K_t = 1$

$\sigma_v = 0.156 \times 22 \times 1 = 3.43\ ms^{-1}$ (from Equation 3.11)

$$\text{Turbulance Intensity}, I_{v(z)} = \sigma_v / V_{m(z)} \text{(EN 1991-1-4:2005+A1:2010, Cl: 4.4.1)} \quad (3.12)$$

$$\text{Wind pressure of structure,}\ q_{p(z)} = \left(1 + 7I_{v(z)}\right) 1/2 \times \rho \times V_{m(z)^2} \quad (3.13)$$

At deck level $Z = 13.6$m

{See Fig 3.8 Bridge Traffic Load, page 3 in figures for Research work}.

- *Wind Force Acting on the Structure*

$$F_{w(z)} = C_s C_d . C_f . q_p . (Z_e) . A_{ref} \text{(EN 1991-1-4:2005+A1:2010, Cl:5.3.1)} \quad (3.14)$$

Where: $C_s C_d$= structural factor, C_f = force coefficient, q_p = peak velocity pressure,
A_{ref}= reference area of the structure, Z_e = reference height.
$Z_e = 11.1 + d_2$ or d_3

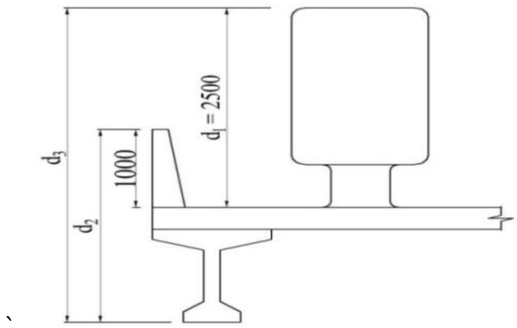

Fig 3.12 Wind Force Acting on the Bridge Deck (All dimensions in mm)

$d_2 = 2.5$ m
$d_3 = 3.9$ m
$d_2 =$ without traffic
$d_3 =$ with traffic
Without traffic, $\frac{b}{d_{tot}} = \frac{11.6}{2.5} = 4.6 \qquad S_{O,}C_f = 1.15$
With traffic, $\frac{b}{d_{tot}} = \frac{11.6}{3.9} = 3.0 \qquad S_{O,}C_f = 1.5$
Consider C_sC_d factor =1 (EN 1991-1-4:2005+A1:2010, Cl:6.2.1a)
So, wind force effective on deck to pier without traffic condition $= 23.2 \times 1.09953 \times 2.5$ $= 63.77$ kN(Table 3.8), and wind force effective on deck to pier with traffic condition = 23.2x1.09953 = 99.49 kN (Fig 3.8).

E. Horizontal Load Due to Water Current on Pier

$$P = 52kV^2 \tag{3.15}$$

Assume square ended pier, $k = 1.5$
Assume river velocity, $v = 1.1$ m/s
$\therefore P = 52 \times 1.5 \times 1.1^2 = 94.38$ kg/m^2 $= 0.9438$ kN/m^2(from Equation 3.15)
Force due to the debris, $P = 52kV^2 = 0.63 N/m^2$

Where; $k = 1$, and $P = 0.63 N/m^2$

Assume length of 11.6 m with depth of 1.2 m debris.
So, maximum debris force on pier $= 0.63 \times 1.2 \times 11.6 = 8.77$ kN

- *Bridge Span of 25m*

  Figure 3.13 shows the bridge with a span of 25m

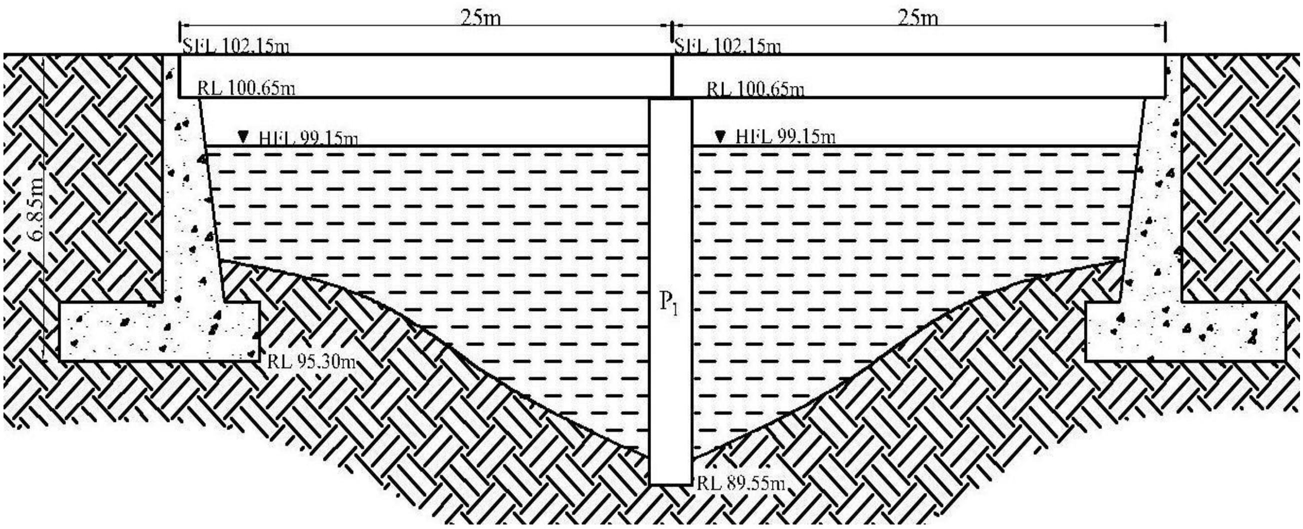

Fig 3.13 Typical Bridge Section with the Span of 25m

➢ *Load Calculations for Girders of Span 25m*
Considering the applicability of Courbon's Method

$\text{Span/width} = 25/11 = 2.27 > 2.$ Hence, it is adequate.
Depth of longitudinal girders = 1362.5 mm.
Depth of cross girders $= 1050 > 0.75 \times$ depth of longitudinal girders
Hence, adequate.

➢ *Load on Abutment Calculations of Span 25m*

- *Vertical Load on Abutment from Superstructure of Span 25m*
From shear force calculations of beam:

Self weight of the superstructure = 312.5 kN/Bearing pad
Self weight of the diapragm structure = 54 kN/ Bearing pad
Total dead load on the abutment = 366.5 kN/Bearing pad
Uniform dead load on abutment = 183.25 kN/m
Load from the pedastrian (Imposed) = 46.669 kN/ Bearing pad
Load from the traffic (Imposed) =148.688 kN/ Bearing pad
Load from the TS (Imposed) = 170 kN/Bearing pad
Total imposed load on the abutment = 365.357 kN/ Bearing pad
Uniform imposed load on abutment = 182.679 kN/m

{See Fig 3.9 Restoring Moment and Stabilizing Forces on the Abutment Wall around the Toe of Span 25m, page 4 in Figures for Research work}. Fig 3.9 shows the analysis of figures 3.7 and 3.8.

{See Fig 3.10 Overturning Moment and Destabilizing Forces on the Abutment Wall around the Toe of Span 25m, page 4 in Figures for Research work}. Fig 3.10, was gotten from the result analysis of sections 3.5, 3.5.1 and figure 3.8.

{See Fig 3.11 Axial Forces on Abutment Walls of Span 25m, page 4 in Figures for Research work}. For axal forces on the abutment wall, lever arm is measured from wall center to load center, horizontally (lateral force on wall).

{See Fig 3.12 Shear Forces on Abutment of Span 25m, page 5 in Figures for Research work}. Shear forces on the abutment wall,lever arm is measured from base top level to load center vertically.

➢ *Pier on Span of 25m*
The bridge section showing the components and their dimensions in Fig 3.14.

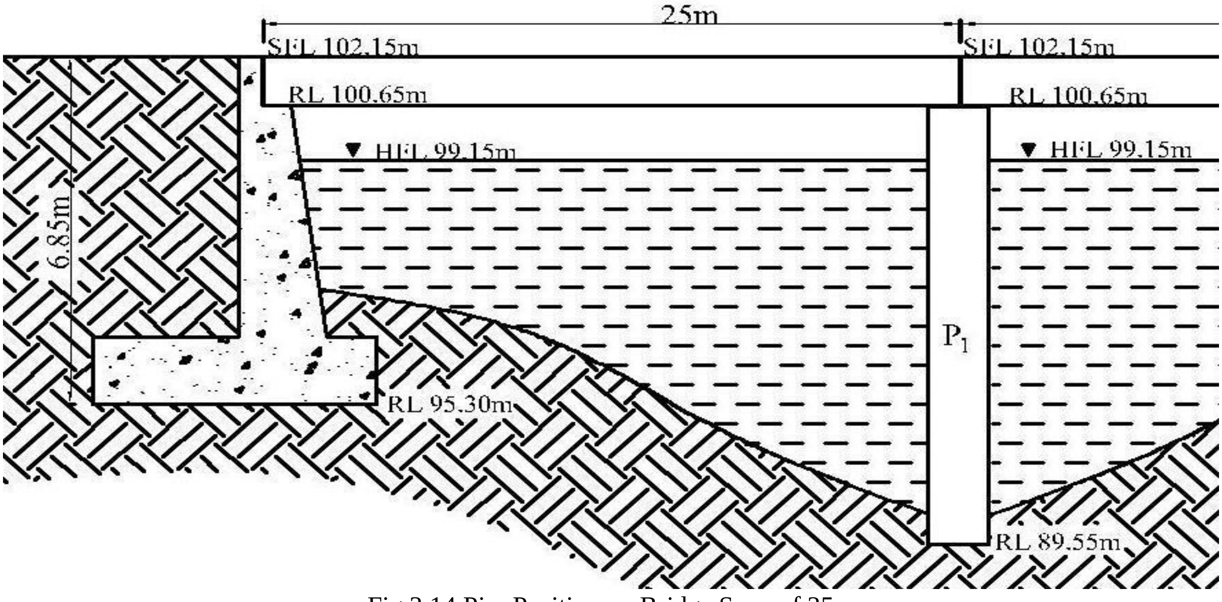

Fig 3.14 Pier Position on Bridge Span of 25m

➢ *Load Calculation for Pier of Span 25m*

- *Vertical Load on Pier from Superstructure of Span 25m*
  From the shear force calculations of beam of bridge deck:

Self weight of the superstructure = 625 kN/ Bearing Pad
Self weight of the diaphragm structure = 108 kN/ Bearing Pad
Total dead load on the abutment = 733 kN/Bearing pad
Uniform dead load on abutment = 366.5 kN/m
Load from the pedestrain (Imposed) = 93.338 kN/Bearing pad
Load from the traffic (Imposed) = 297.376 kN/Bearing pad
Load from the tandem system(TS) (Imposed) = 340 kN/Bearing pad
Load imposed load on the abutment = 730.714 kN/Bearing pad
Uniform imposed load on abutment = 365.357 kN/m

➢ *Bridge Span of 30m*

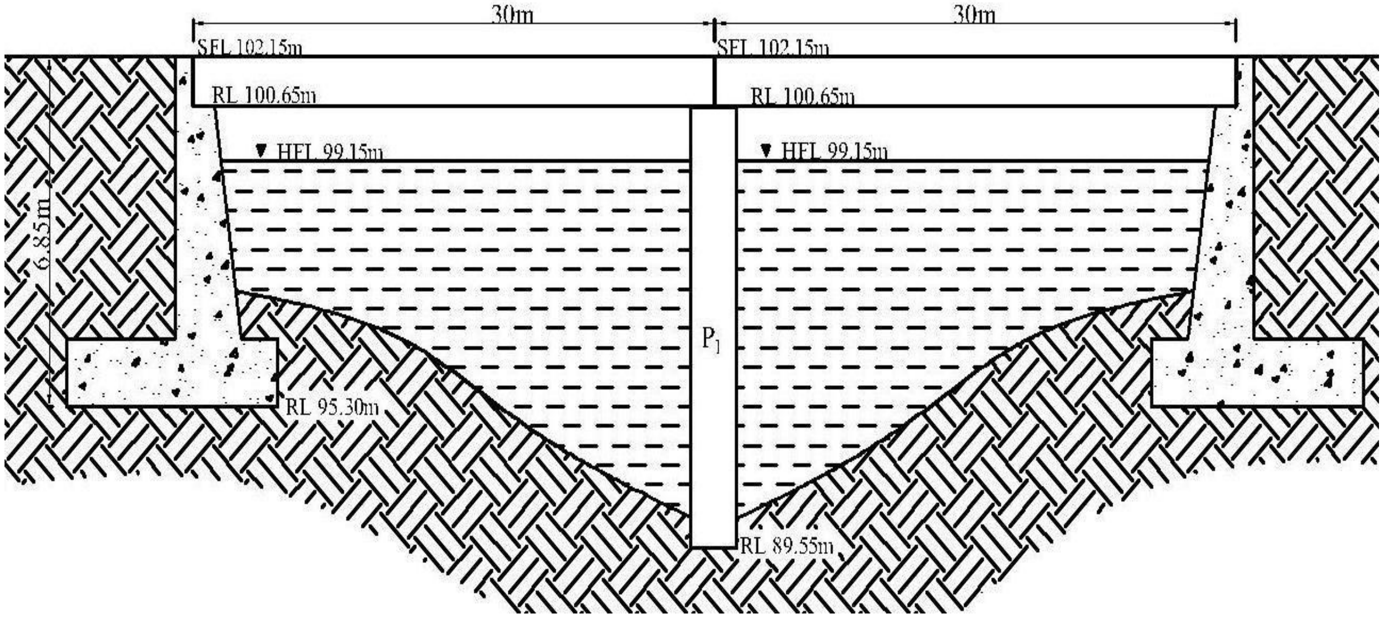

Fig 3.15 Typical Bridge Section with the Span of 30m

➢ *Load Calculations for Girders of Span 30m*
Span/width $= 30/11 = 2.727 > 2$. Hence, it is adequate.
Depth of longitudinal girders = 1362.5 mm
Depth of cross girders $= 1050 > 0.75 \times$ depth of longitudinal girders
Hence, adequate.

➢ *Load on Abutment Calculations of Span 30m*

- *Vertical Load on Abutment from Superstructure of Span 30m*
  From shear force calculations of the beam:

Self weight of the superstructure= 375 kN/Bearing pad
Self weight of the diapragm structure = 54 kN/ Bearing pad
Total dead load on the abutment = 429kN/Bearing pad
Uniform dead load on abutment = 214.5 kN/m
Load from the pedastrian (Imposed) = 56.003 kN/ Bearing pad
Load from the traffic (Imposed) =178.425 kN/ Bearing pad
Load from the tandem system(TS) (Imposed) = 170 kN/Bearing pad
Total imposed load on the abutment = 404.428 kN/ Bearing pad
Uniform imposed load on abutment = 202.214 kN/m

{See Fig 3.13 Restoring Moment and Stabilizing Forces on the Abutment Wall around the Toe of Span 30m, page 5 in figures for Research work}. Fig 3.13 shows the analysis of figures 3.7 and 3.8.

{See Fig 3.14 Overturning Moment and Destabilizing Forces on the Abutment Wall around the Toe of Span 30m, page 5 in figures for Research work}. Fig 3.14, was gotten from the result analysis of sections 3.5, 3.5.1 and figure 3.8.

{See Fig 3.15 Axial Forces on Abutment Walls of Span 30m, page 6 in Tables for Research work}. For axal forces on the abutment wall, lever arm is measured from wall center to load center, horizontally (lateral force on wall).

{See Fig 3.16 Shear Forces on Abutment of Span 30m, page 6 in Tables for Research work}. Shear forces on the abutment wall, lever arm is measured from base top level to load center vertically.

- *Pier on Span of 30m*

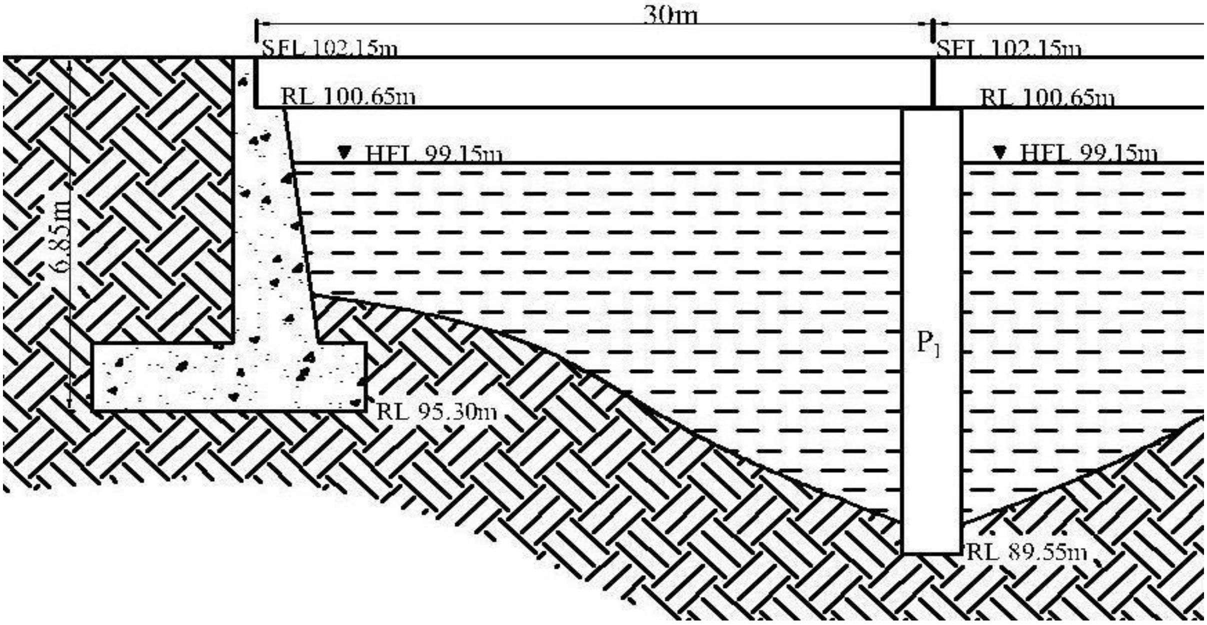

Fig 3.16 Pier Position on Bridge Span of 30m

- *Load Calculation for Pier of Span 30m*

- *Vertical Load on Pier from Superstructure of Span 30m*
  From shear foce calculation of beam of bridge deck analysis:

Self weight of the superstructure = 750 kN/ Bearing Pad
Self weight of the diaphragm structure = 108 kN/ Bearing Pad
Total dead load on the abutment = 858 kN/Bearing pad
Uniform dead load on abutment = 429 kN/m
Load from the pedestrain (Imposed) = 112.01 kN/Bearing pad
Load from the traffic (Imposed) = 356.85 kN/Bearing pad
Load from the TS (Imposed) = 340 kN/Bearing pad
Load imposed load on the abutment = 808.86 kN/Bearing pad
Uniform imposed load on abutment = 404.43 kN/m

- *Bridge Span of 35m*

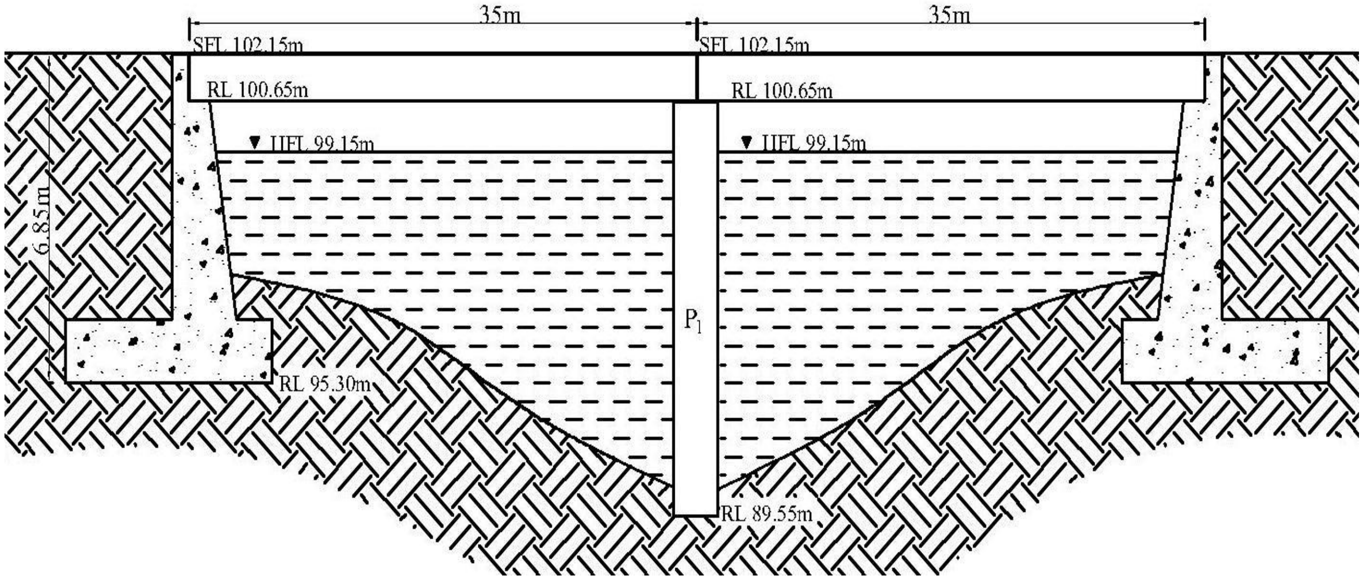

Fig 3.17 Typical Bridge Section with the Span of 35m

- *Load Calculations for Girders of Span 35m*

Considering the applicability of Courbon's Method
$\text{Span/width} = 35/11 = 3.18 > 2$. Hence, it is adequate.
Depth of longitudinal girders = 1362.5 mm
Depth of cross girders $= 1050 > 0.75 \times$ depth of longitudinal girders
Hence, adequate.

- *Load on Abutment Calculations of Span 35m*

- *Vertical Load on Abutment from Superstructure of Span 35m*
  From shear foce calculation of beam:

Self weight of the superstructure = 437.5 kN/Bearing pad
Self weight of the diapragm structure = 54 kN/ Bearing pad
Total dead load on the abutment = 491.5 kN/Bearing pad
Uniform dead load on abutment = 245.75 kN/m
Load from the pedastrian (Imposed) = 65.336 kN/ Bearing pad
Load from the trafic (Imposed) =208.163 kN/ Bearing pad
Load from the TS (Imposed) = 170 kN/Bearing pad
Total imposed load on the abutment = 443.499 kN/ Bearing pad
Uniform imposed load on abutment = 221.750 kN/m

{See Fig 3.17 Restoring Moment and Stabilizing Forces on the Abutment Wall around the Toe of Span 35m, page 6 in figures for Research work}. Fig 3.17 shows the analysis of figures 3.7 and 3.8.

{See Fig 3.18 Overturning Moment and Destabilizing Forces on the Abutment Wall around the Toe of Span 35m, page 7 in figures for Research work}. Fig 3.18, was gotten from the result analysis of sections 3.5, 3.5.1 and figure 3.8.

{See Fig 3.19 Axial Forces on Abutment Walls of Span 35m, page 7 in figures for Research work}. For axal forces on the abutment wall, lever arm is measured from wall center to load center, horizontally (lateral force on wall).

{See Fig 3.20 Shear Forces on Abutment of Span 35m, page 7 in figures for Research work}. Shear forces on the abutment wall,lever arm is measured from base top level to load center vertically.

➢ *Pier on Span of 35*

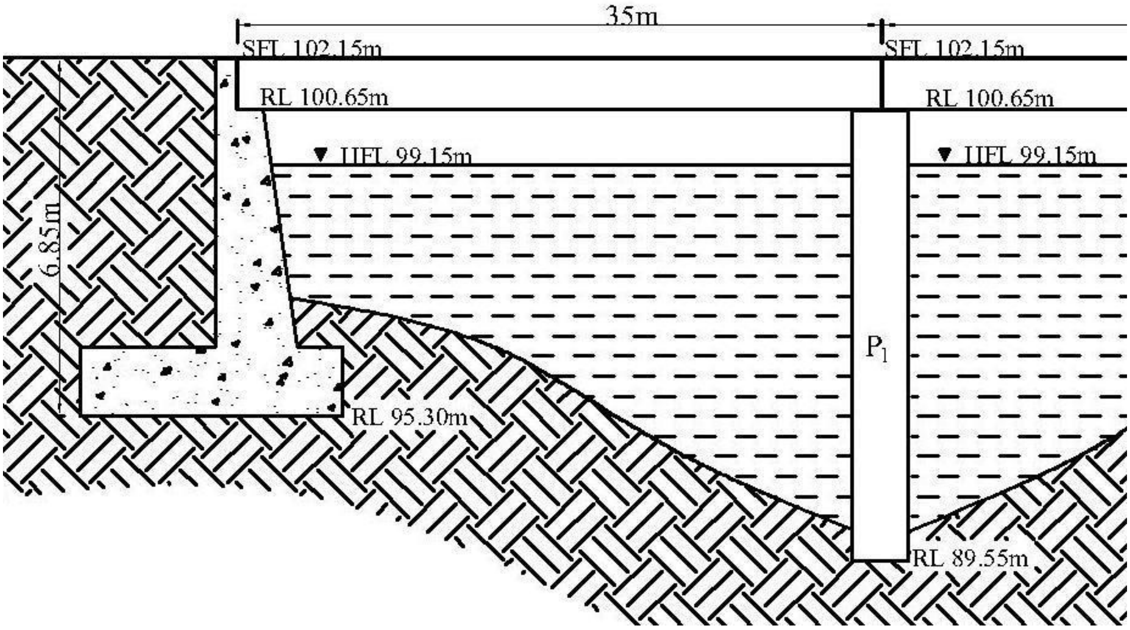

Fig 3.18 Pier Position on Bridge Span of 35m

➢ *Load Calculations for Pier of Span 35m*

- *Vertical Load on Pier from Superstructure of Span 35m*
  From shear force calculation of beam of bridge deck analysis:

Self weight of the superstructure =875 kN/ Bearing Pad
Self weight of the diaphragm structure =108 kN/ Bearing Pad
Total dead load on the abutment = 983 kN/Bearing pad
Uniform dead load on abutment = 491.5 kN/m
Load from the pedestrain (Imposed) = 130.672 kN/Bearing pad
Load from the traffic (Imposed) = 416.326 kN/Bearing pad
Load from the TS (Imposed) = 340 kN/Bearing pad
Load imposed load on the abutment = 886.998 kN/Bearing pad
Uniform imposed load on abutment = 443.499 kN/m

➢ *Bridge Span of 40m*

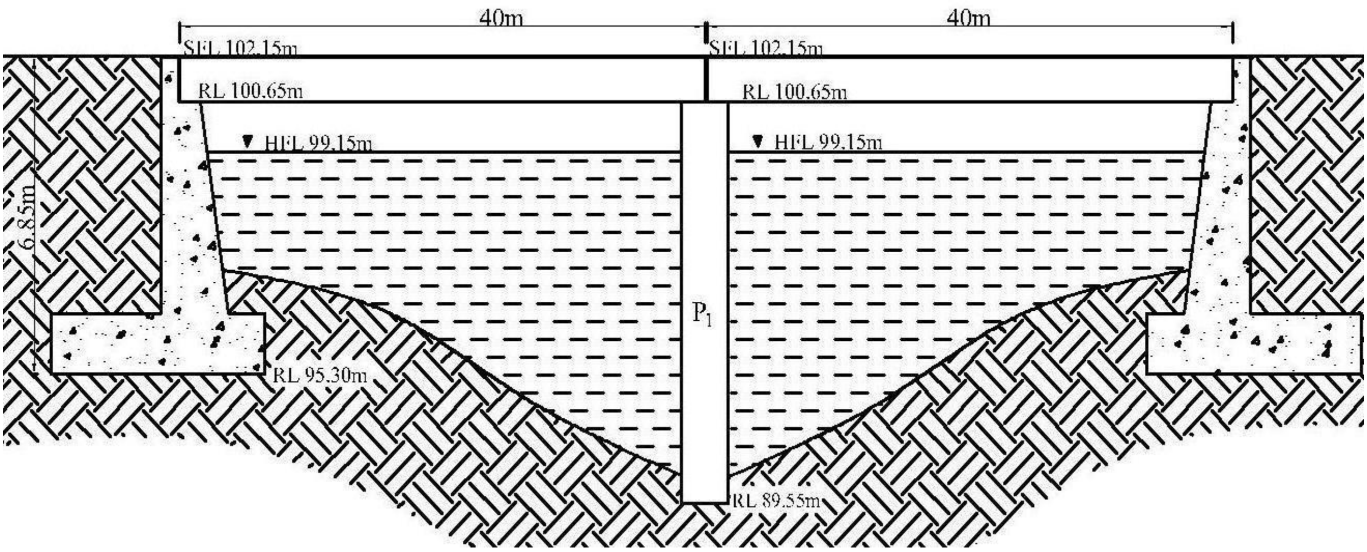

Fig 3.19 Typical Bridge Section with the Span of 40m

ISSN No:-2456-2165

➢ *Load Calculations for Girders of Span 40m*

Considering the applicability of Courbon's Method
$\text{Span/width} = 40/11 = 3.64 > 2.$ Hence, it is adequate.
Depth of longitudinal girders = 1362.5 mm
Depth of cross girders $= 1050 > 0.75 \times$ depth of longitudinal girders
Hence, adequate.

➢ *Load Calculations for Abutment of Span 40m*

- *Vertical Load on Abutment from Superstructure of Span 40m*
  From shear foce calculation of beam:

Self weight of the superstructure = 500 kN/Bearing pad
Self weight of the diapragm structure = 54 kN/ Bearing pad
Total dead load on the abutment = 554 kN/Bearing pad
Uniform dead load on abutment = 277 kN/m
Load from the pedastrian (Imposed) = 74.67 kN/ Bearing pad
Load from the trafic (Imposed) =237.9 kN/ Bearing pad
Load from the TS (Imposed) = 170 kN/Bearing pad
Total imposed doad on the abutment = 482.57 kN/ Bearing pad
Uniform imposed load on abutment = 241.285 kN/m

{See Fig 3.21 Restoring Moment and Stabilizing Forces on the Abutment Wall around the Toe of Span 40m, page 8 in Fig for Research work}. Fig 3.20 shows the analysis of figures 3.7 and 3.8.

{See Fig 3.22 Overturning Moment and Destabilizing Forces on the Abutment Wall around the Toe of Span 40m, page 8 in Figures for Research work}. Fig 3.18, was gotten from the result analysis of sections 3.5, 3.5.1 and figure 3.8.

{See Fig 3.23 Axial Forces on Abutment Walls of Span 40m, page 8 in Figures for Research work**}.** For axal forces on the abutment wall, lever arm is measured from wall center to load center, horizontally (lateral force on wall).

{See Fig 3.24 Shear Forces on Abutment of Span 40m, page 9 in Figures for Research work}. Shear forces on the abutment wall,lever arm is measured from base top level to load center vertically.

➢ *Pier on Span of 40m*

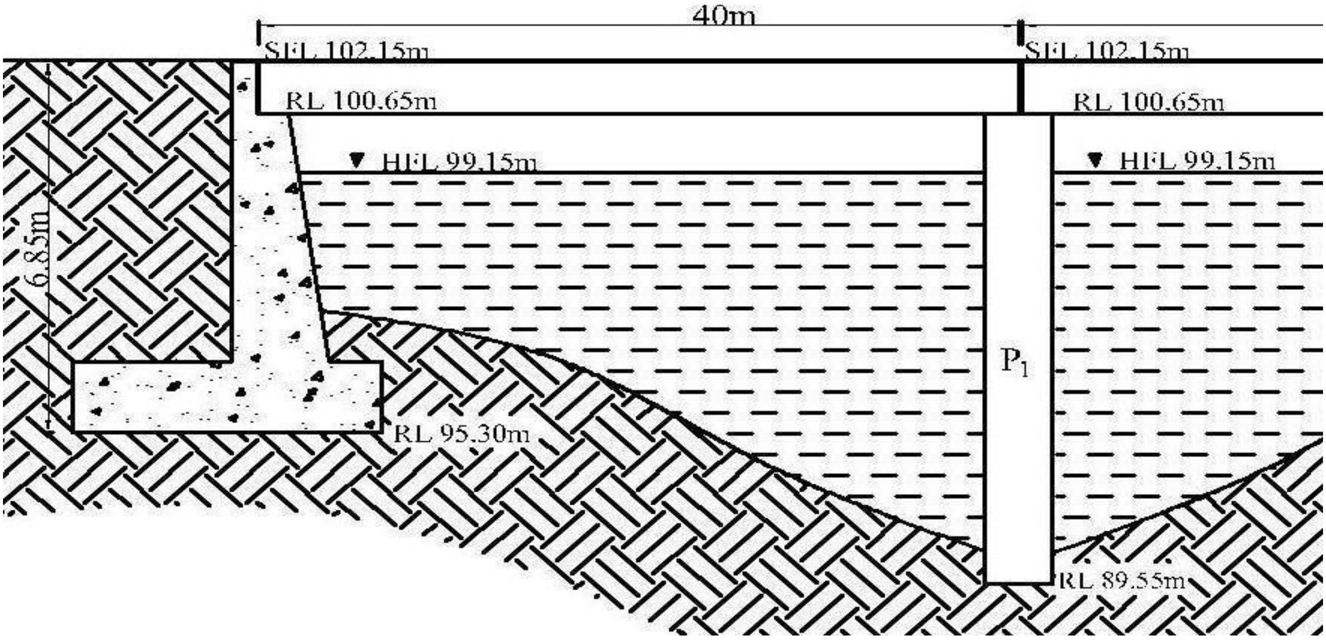

Fig 3.20 Pier position on Bridge Span of 40m

ISSN No:-2456-2165

➢ *Load Calculation for Pierof Span 40m*

- *Vertical Load on Pier from Superstructure of Span 40m*
  From shear foce calculation of beam of bridge deck analysis:

Self weight of the superstructure = 1000 kN/ Bearing Pad
Self weight of the diaphragm structure =108 kN/ Bearing Pad
Total dead load on the abutment = 1108 kN/Bearing pad
Uniform dead load on abutment = 554 kN/m
Load from the pedestrain (Imposed) = 149.34 kN/Bearing pad
Load from the traffic (Imposed) = 475.8 kN/Bearing pad
Load from the TS (Imposed) = 340 kN/Bearing pad
Load imposed load on the abutment = 965.14 kN/Bearing pad
Uniform imposed load on abutment = 482.57 kN/m.

F. Creation of a Computer Program for the Design of Bridge Substructures

SUBBridge is a computer program written in Java Programming language based on the design of Bridge substructures. It is written to reduce the time used in the analysis and design of bridge foundation and has rich a graphical interface to aid the user visualize the result of the analysis. Design of bridge substructures using SUBBridge, is organized into several classes. Using a Unified Modeling Language (UML) diagram, the various packages classes are presented in the next section:

➢ *Package SUBBridge*

This package shall contain the main classes which include SUBMain, SUBDetails, SUBTables and SUBGraphs.

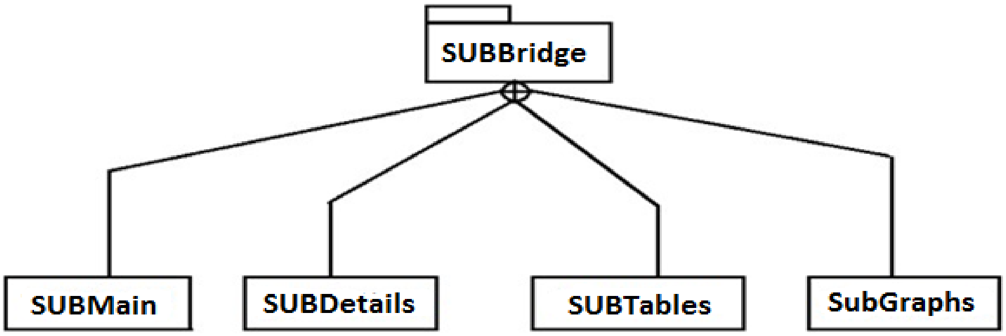

Fig 3.21 Package SUBBridge showing its Member Classes

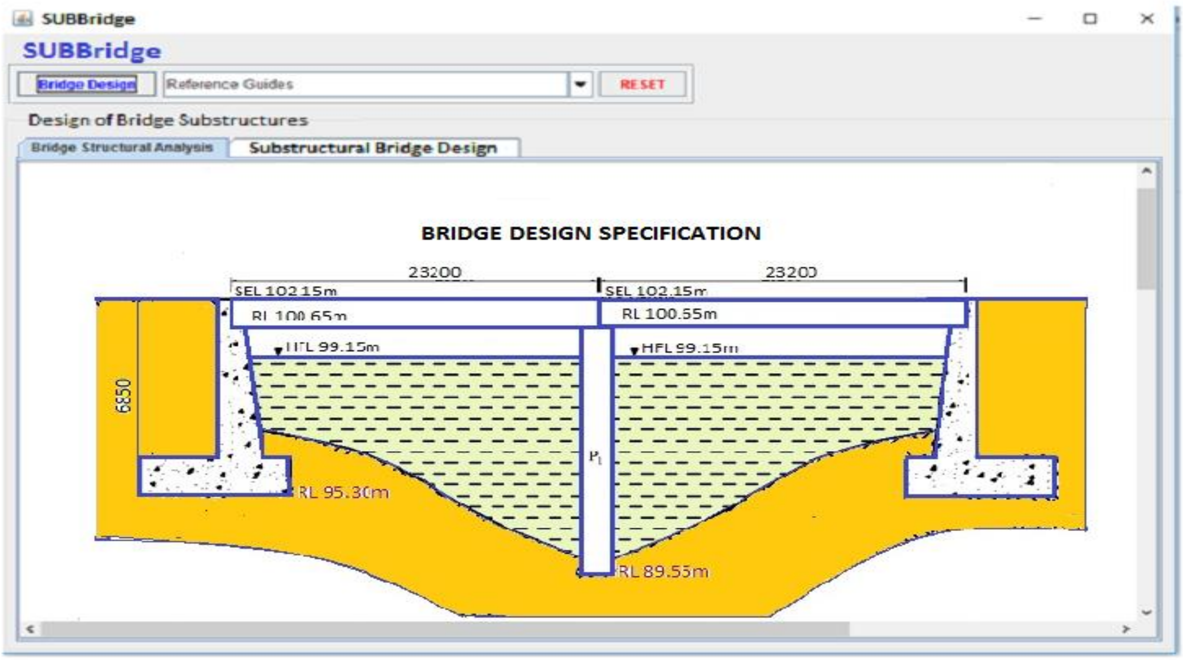

Fig 3.22 The Main Application Window for the Developed Computer Program

CHAPTER FOUR
DESIGN, RESULTS AND ANALYSIS

A. Bridge Analysis

Fig 4.1 shows the actions on the selected beam, the self-weight of the structures are going to be uniformly distributed to the beam which is proportion to the effective depth.

{See Fig 4.1 Load Distribution by Bridge Components, page 9 in Tables for Research work}.

UDL on the dead load $= 6.25 + 0.72 + 7.98 + 9.05 = 24\text{kN/m}$
Hand dail doad $= 1\ \text{kN/m}$
Total load on the deck $= 24 + 1 = 25\ \text{kN/m}$
Point load on girder 'A' = 18 kN at diaphram connection
Total dead doad = 25 kN/m
Diaphragm load = 18 kN at 4.25 m c/c
Total pedastrian load = 3.73 kN/m
Total traffic load = 11.9 kN/m
Total tandem system (TS) load = 340 kN at midlle of the girder

➢ *Shear Force Calculations of Span 23.2m*

Shear force from diaphragm $= 18 \times 3 = 54$ kN
Shear force from pedestrain $= (3.7335) \times 23.2 \times 0.5 = 43.31$kN
Shear force from (TS) $= 340 \times 0.5 = 170$ kN
Shear force from dead load $= 25 \times 23.2 \times 0.5 = 290$ kN
Shear force from traffic load $= (11.895) \times 23.2 \times 0.5 = 137.982$ kN
Total shear force of end of beam at serviceability limit state (SLS) = 695.292 kN
Fig 4.1 shows the shear force diagram for the bridge span of 23.2m.

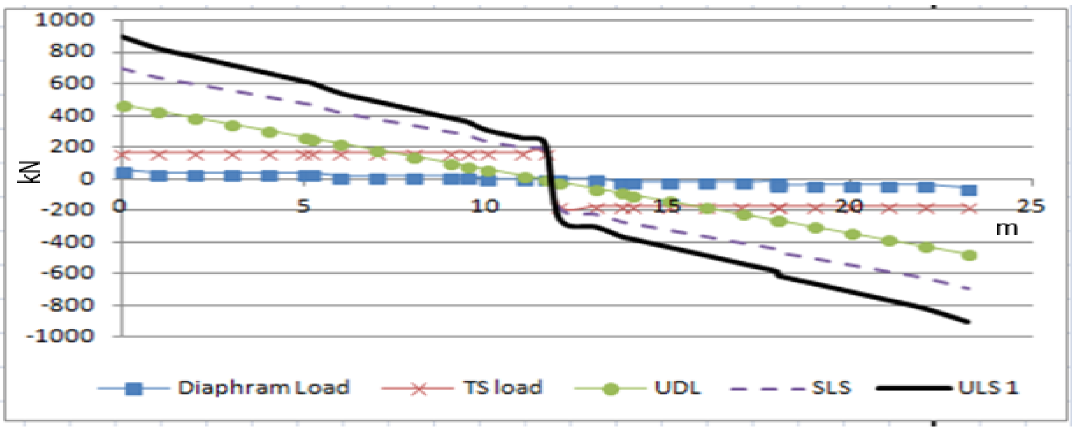

Fig 4.1 Shear Force Diagram of Span 23.2m

➢ *Bending Moment Calculations of Span 23.2m*

$$\text{Middle moment (uniformly distributed load)UDL dead} = 25 \times \frac{23.2^2}{8} = 1682\ \text{kNm}$$

$$\text{Middle moment UDL traffic} = 11.895 \times \frac{23.2^2}{8} = 800.3\ \text{kNm}$$

$$\text{Middle moment pedestrian} = 3.74 \times \frac{23.2^2}{8} = 251.19\ \text{kNm}$$

$$\text{Mddle moment from TS} = 340 \times \frac{23.2}{4} = 1972\ \text{kNm}$$

$$\text{Middle moad from diaphragm} = 264.6\ \text{kNm}$$

Thus, SLS resultant middle moment $= 264.6 + 1972 + 1682 + 800.3 + 251.19 = 4970.09\text{kNm}$.

Figure 4.2 shows the moment envelope diagram of bridge deck span of

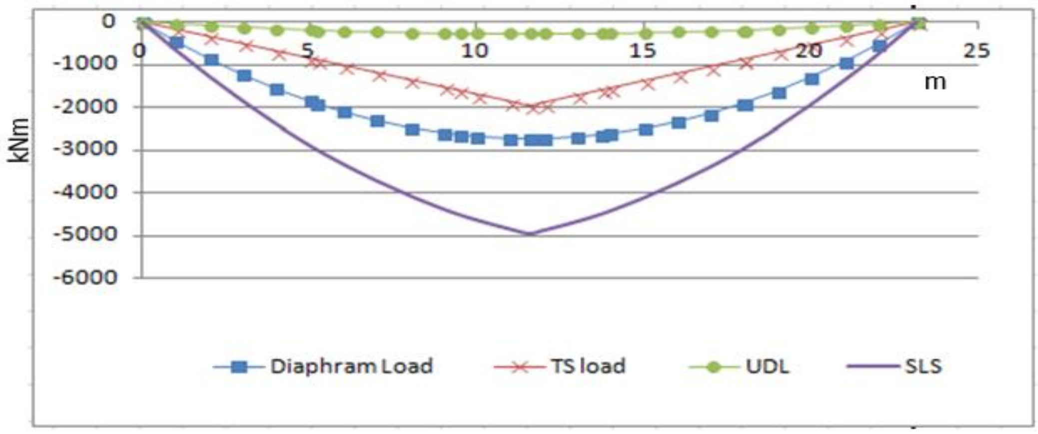

Fig 4.2 Moment Diagram of Span 23.2m

➢ *Ultimate Limit State (ULS) of Span 23.2m*
For the ULS, consider the following load combinations (BS EN 1990:2002)

$$\text{ULS1: } 1.35\text{G}_{\text{kj,sup}} + 1.0\ \text{G}_{\text{kj,inf}} + 1.35\ (\text{TS} + \text{UDL} + 0.4 \times \text{q}_{\text{fk}}) \quad (4.1)$$

ULS2: $1.35\text{G}_{\text{kj,sup}} + 1.0\ \text{G}_{\text{kj,inf}} + 1.35\ \text{gr1b}$
ULS3: $1.35\text{G}_{\text{kj,sup}} + 1.0\ \text{G}_{\text{kj,inf}} + 1.35\ \text{gr2}$
ULS4: $1.35\text{G}_{\text{kj,sup}} + 1.0\ \text{G}_{\text{kj,inf}} + 1.35\ \text{gr5}$

So,maximum moment at middle for first combination of ultimate limit state (ULS)
$= 1.35(1682 + 264.6) + 1.35(800.3 + 1972 + 0.4 \times 251.19) = 6506.16\text{kNm}$ (from Equation 4.1)

So, maximum shear force at beam end for first combination of ULS
$= 1.35(290 + 54) + 1.35(170 + 138 + 0.4 \times 43.31) = 903.563\ \text{kN}$ (from Equation 4.1)

The ULS diagram is shown in Fig 4.3

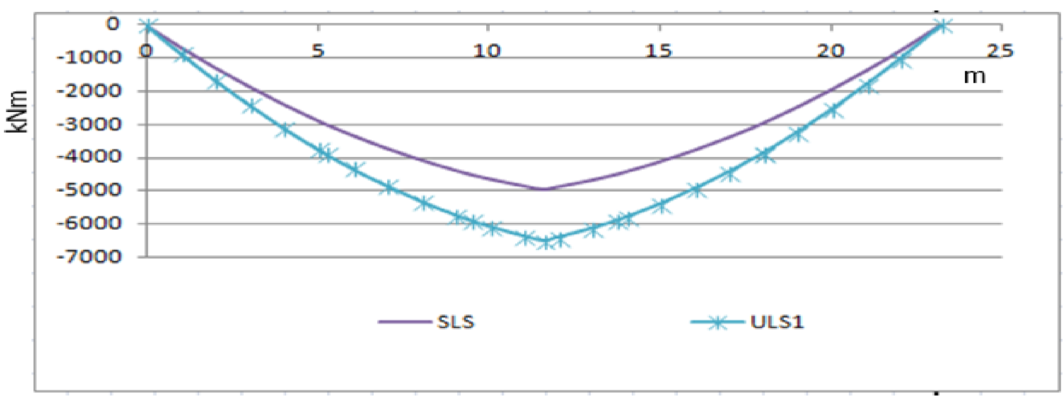

Fig 4.3 Moment at Middle of the Beam for First Combination Diagram of Span 23.2m

➢ *Check for the bearing failure*

Lever arm were considered from the center of the base. Fig 4.2, shows the stability check for the abutment. {See Fig 4.2 Stability Check of Span 23.2m, page 9 in Tables for Research work}.

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➢ *Summary of the Base Reactions of Span 23.2m*

Vertical load N = 922.698 kN (Fig 4.3).

Moment around the base center = 764.1537 kNm (Fig 4.3 and 4.4).

$$M = \frac{N}{BD} \pm \frac{6M}{BD^2} \tag{4.2}$$

Where: N = axialforce; B = width of the footing; M = Moment; D = length of the footing

$$M_{max} = \frac{922.698}{(5.1 \times 1)} + 6 \times \frac{764.1537}{(5.1^2)} = 357.1966 < \text{Ground bearing capacity of } 670 \text{ kN/m}^2$$

$$M_{min} = \frac{922.698}{(5.1 \times 1)} - 6 \times \frac{764.1537}{(5.1^2)} = 4.6458 < \text{Ground bearing capacity of } 670 \text{ kN/m}^2$$

So, there is no negative pressure that is which tensioning the base.

{See Fig 4.3 Calculations of Moment for the Determination of Bearing Failure of Span 23.2m, page 9 in Figures for Research work}.

{See Fig 4.4 Calculations of Moments for the Determination of Bearing Failure of Span 23.2m, page 10 in Figures for Research work}.

The reinforcement design for the abutment, as per the stability calculations following, the EC0 and EC4 load combination, are given by:

$Q_k + 1.5G_k + 1.5G_{ki}\psi_i$

Where;

Q_k = Characteristic value of a variable actio;
G_{ki} = Characteristic value of a permanent action;
ψ_i = Factor for frequent value of a variable action.

{See Fig 4.5 Summary of Stability Check of Span 23.2m, page 10 in Fig for Research work}.

Fig 4.6 shows, thesummary of the wall design moment,axial force and shear force at base top level (Fig 3.6 and 3.7).
{See Fig 4.6 The Summary of all the Forces acting on the Abutment of Span 23.2m, page 10 in Tables for Research work}.

➢ *Design of Abutment of Span 23.2m*

Assume abutment concrete grade is 25 N/mm^2

Concrete compression capacity of the wall = $0.567 f_{ck} bd$ (4.3)

Where; f_{ck}= concrete grade , b = width, d = effective depth.

Concrete compression capacity $= 0.567 \times 25 \times 1000 \times 1175 = 16655.65$ kN

So, axial force capacity is higher than the existing axial force. So, it shall be considered as a cantilever wall.

$$k = \frac{M_{ed}}{bd^2 f_{ck}} \tag{4.4}$$

Where; M_{ed} = design moment (Fig 4.6).

$$k = \frac{935.90 \times 10^6}{1000 \times 1175^2 \times 25} = 0.02712 \quad , \quad \text{(from Equation 4.4)}$$

0.02712 < 0.156. So, section is singly reinforced.
$l_a = 0.95$
$z = 0.95 \times 1175 = 1116.25$ mm

Where; la = lever arm factor; z = lever arm.

$$A_s = \frac{M_{ed}}{0.87 f_y z} \tag{4.5}$$

Where; A_s = area of reinforcment; M_{ed} = design moment; f_y = steel grade

$$A_s = \frac{935.90 \times 10^6}{0.87 \times 500 \times 1116.25} = 1927.43\text{mm}^2 \text{ (from Equation 4.5)}$$

$$A_{smin} = 0.13 \times \frac{bd}{100} \tag{4.6}$$

Where; A_{smin} = area of minimum reinforcement.

$$A_{smin} = 0.13 \times 1000 \times \frac{1175}{100} = 1527.5\text{mm}^2 \quad \text{(from Equation 4.6)}$$

$1527.5 \text{ mm}^2 < 1927.43 \text{ mm}^2$

Therefore, $A_{sreq} = 1927.43\text{mm}^2$

Provide H20 @ 100mm c/cvertical and H20-200 horizontal

Therefore, $A_{sprov} = 3130 \text{ mm}^2$

Therefore, $A_{sproH} = 1565 \text{ mm}^2$

- *Check For Shear*

$$\text{Design Shear stress on wall} = \frac{366693}{1000 \times 1175} = 0.31\text{N/mm}^2$$

$$\text{Shear capacity of concrete, } V_{Rdc} = \left[0.12k\left(100\rho f_{ck}\right)^{\frac{1}{3}}\right]bd \tag{4.7}$$

$$V_{Rdc} = 372.38 \text{ kN} > 366.69\text{kN} \text{ (from Equation 4.7)}$$

Therefore, shear capacity is adequate as per the design calculation on bearing pressure calculation.

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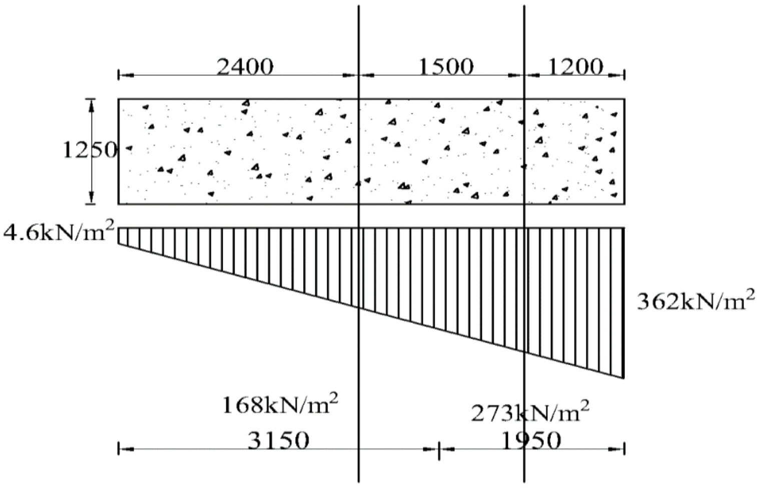

Fig 4.4 Abutment Base Moment Diagram

$$\text{Base moment at bottom RHS} = (362.4 + 273) \times 0.5 \times 1.2^2 \times 0.5 = 228.744 \text{ kNm/m}$$

$$\text{Base moment at bottom LHS} = (4.6 + 168) \times 0.5 \times 2.4^2 \times 0.5 = 248.544 \text{ kNm/m}$$

$$\text{So, maximum bottom moment} = 248.544 \text{ kNm}$$

$$k = \frac{248.544 \times 10^6}{1000 \times 1175^2 \times 25} = 0.0072 \,, \qquad \text{(from Equation 4.4)}$$

0.02712 < 0.156. So, section is singly reinforced.

$l_a = 0.93$

$z = 0.93 \times 1175 = 1093 \text{ mm}$

$$A_s = \frac{248.544 \times 10^6}{0.87 \times 500 \times 1093} = 522.869 \text{ mm}^2 \qquad \text{(from Equation 4.5)}$$

$$A_{smin} = 0.13 \times 1000 \times \frac{1175}{100} = 1527.5 \text{mm}^2 \quad \text{(from Equation 4.6)}$$

1527.5 mm^2 > 522.869 mm^2

Thus, use H20 @200 top and bottom reinforcement.

- *Bridge of Span of 25m Analysis and Design*

- *Shear Force Calculations of Span 25m*

Shear force from diaphragm $= 18 \times 3 = 54$ kN
Shear force from Pedestrain $= (3.7335) \times 25 \times 0.5 = 46.669$kN
Shear force from TS $= 340 \times 0.5 = 170$ kN
Shear force from dead load $= 25 \times 25 \times 0.5 = 312.5$ kN
Shear force from traffic load $= (11.895) \times 25 \times 0.5 = 148.688$ kN
Total shear force of end of beam at SLS $= 54 + 46.669 + 170 + 312.5 + 148.68 = 731.857$ kN

Fig 4.5, shows the shear force diagram for the bridge span of 25 m.

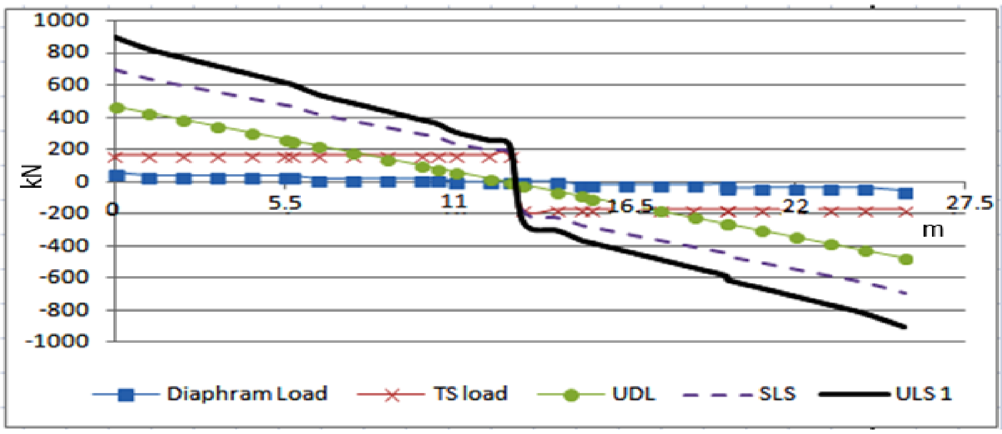

Fig 4.5 Shear force diagram of Span 25m

- *Bending Moment Calculations of Span 25m*

$$\text{Middle moment UDL dead} = 25 \times \frac{25^2}{8} = 1953.125 \text{ kNm}$$

$$\text{Middle moment UDL traffic} = 11.895 \times \frac{25^2}{8} = 929.297 \text{ kNm}$$

$$\text{Middle moment pedestrian} = 3.74 \times \frac{25^2}{8} = 292.188 \text{ kNm}$$

$$\text{Mddle moment from TS} = 340 \times \frac{25}{4} = 2125 \text{ kNm}$$

$\text{Middle load from diaphragm} = 264.6 \text{ kNm}$

$\text{Thus, SLS resultant middle moment} = 264.6 + 2125 + 292.188 + 929.297 + 1953.125 = 5564.21\text{kN}$

The Fig4.6 shows the momoent envelope diagram of bridge deck span of 25 m

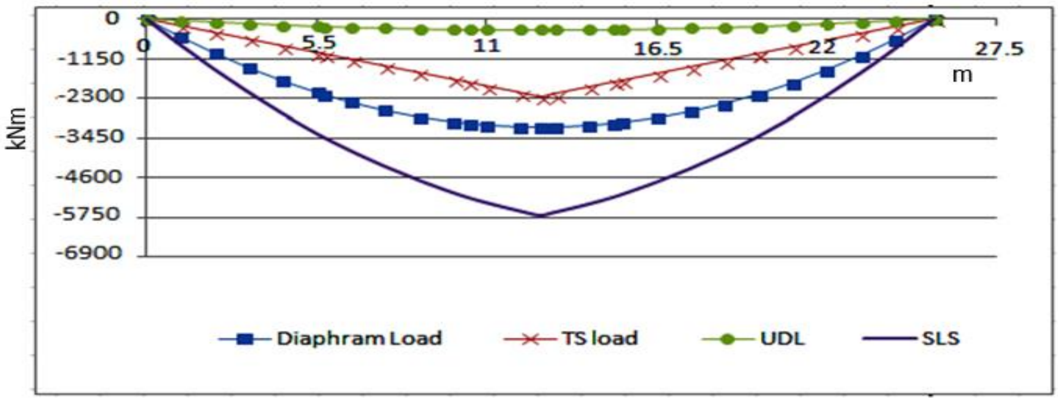

Fig 4.6 Moment Diagram of Span 25m

- *Ultimate Limit State (ULS) Calculations of Span 25m*
  For the ULS shall consider following load combinations (BS EN 1990:2002)

So,maximum moment at middle for first combination of ULS
$= 1.35\,(1953.125 + 264.6) + 1.35\,(929.297 + 2125 + 0.4 \times 292.188) = 7274.842 \text{ kNm} \quad (\text{from Equation } 4.1)$

So,maximum shear force at beam end for first combination of ULS

$= 1.35(312.5 + 54) + 1.35(170 + 148.688 + 0.4 \times 46.669) = 950.205\text{kN}$ (from Equation 4.1)

The ULS diagram for bridge span of 25m was shown in Fig 4.7.

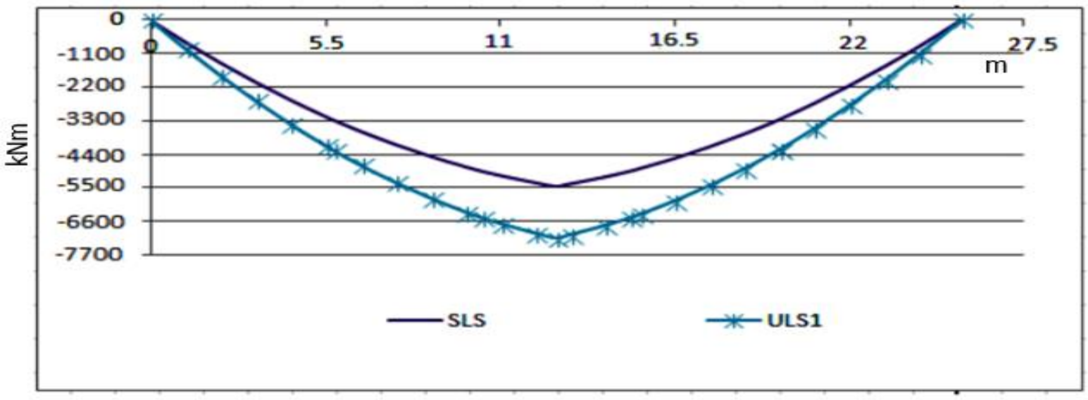

Fig 4.7 Moment diagram for first combination of Span 25m

- *Summary of the Base Reactions of Span 25m*

Vertical Load N = 940.981kN(Fig 4.7).
Moment around the base center = 738.9232 kNm (Fig 4.7 and 4.8).

$$M_{max} = \frac{940.981}{(5.1 \times 1)} + 6 \times \frac{738.9232}{(5.1^2)} = 354.9613 < \text{Ground bearing capacity of } 670\ \text{kN/m}^2$$
(from Equation 4.2)

$$M_{min} = \frac{940.981}{(5.1 \times 1)} - 6 \times \frac{738.9232}{(5.1^2)} = 14.0509 < \text{Ground bearing capacity of } 670\ \text{kN/m}^2$$ (Fig 4.5)
(from Equation 4.2)

So, there is no negative pressure tensioning the base.

Therefore, selected abutment parameters are adequate in terms of structural stability.

- *Check for the Bearing Failure.*
  Lever arm were considered from the center of the base.

{See Fig 4.7 Calculations of Moments for the Determination of Bearing Failure of Span 25m, page 10 in Figures for Research work}.

{See Fig 4.8 Calculation of Moment for the Determination of Bearing Failure of Span 25m, page 11 in Figures for Research work}.

{See Fig 4.9 Summary of Stability Check of Span 25m, page 11 in Figures for Research work}.

Fig 4.10 shows, thesummary of the wall design moment, axial force and shear force at base top level (Fig 3.6 and 3.7 in Chapter Three). {See Fig 4.10 The Summary of all the Forces Acting on the Abutment of Span 25m, page 11 in Figures for Research work}.

➢ *Design of Abutment of Span 25m*

Assume abutment concrete grade is 25 N/mm^2
M_{ed} = 939.76 kNm (Fig 4.10).

$$k = \frac{939.76 \times 10^6}{1000 \times 1175^2 \times 25} = 0.0272\ ,$$ (from Equation 4.4)

0.0272< 0.156. So, section is singly reinforced.
$l_a = 0.95, \quad z = 0.95 \times 1175 = 1116.25$ mm

$$A_s = \frac{939.76 \times 10^6}{0.87 \times 500 \times 1116.25} = 1935.38\text{mm}^2 \quad \text{(from Equation 4.5)}$$

$$A_{smin} = 0.13 \times 1000 \times \frac{1175}{100} = 1527.5\text{mm}^2 \quad \text{(from Equation 4.6)}$$

$1527.5 \text{ mm}^2 < 1935.38 \text{ mm}^2$

Therefore, $A_{sreq} = 1935.38 \text{ mm}^2$

Provide H20 @ 100 mm c/c vertical and H20-175 horizontal

Therefore , $A_{sproV} = 3130 \text{ mm}^2$

Therefore, $A_{sproH} = 1800 \text{ mm}^2$

- *Check For Shear*

$$\text{Design shear stress on wall} = \frac{366693}{1000 \times 1175} = 0.31\text{N/mm}^2$$

$$V_{Rdc} = 346.41 \text{ kN} < 356.78 \text{ kN} \text{(from Equation 4.7)}$$

Therefore, shear capacity is adequate.

➢ *Bridge of Span of 30m Analysis and Design*

- *Shear Force Calculations of Span 30m*

Shear force from diaphragm $= 18 \times 3 = 54$ kN
Shear force from pedestrain $= (3.7335) \times 30 \times 0.5 = 56.003$kN
Shear force from TS $= 340 \times 0.5 = 170$ kN
Shear force from dead load $= 25 \times 30 \times 0.5 = 375$ kN
Shear force from traffic load $= (11.895) \times 30 \times 0.5 = 178.425$ kN
Total shear force of end of beam at SLS $= 54 + 56.003 + 170 + 375 + 178.425 \quad = 833.428$ kN

Fig 4.8, shows the shear force diagram for the bridge span of 30 m.

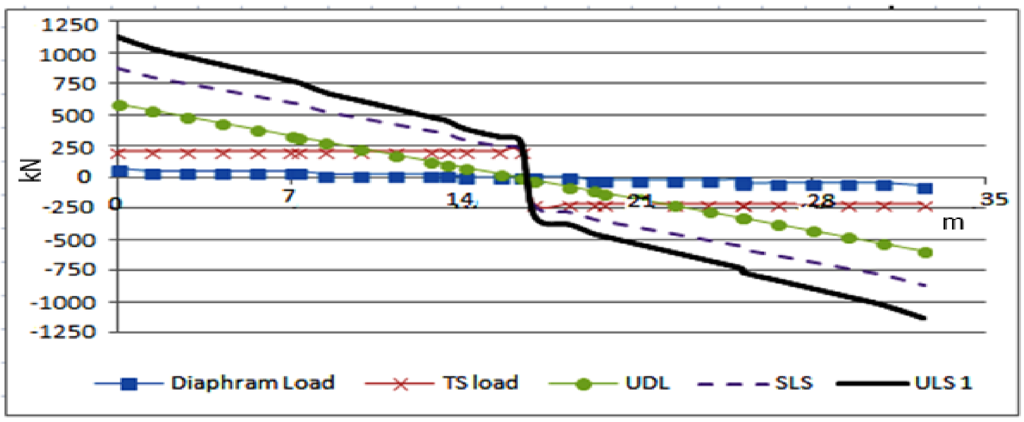

Fig 4.8 Shear Force Diagram of Span 30m

- *Bending Moment Calculations of Span 30m*
-

$$\text{Middle moment UDL dead} = 25 \times \frac{30^2}{8} = 2812.5 \text{ kNm}$$

$$\text{Middle moment UDL traffic} = 11.895 \times \frac{30^2}{8} = 1338.188 \text{ kNm}$$

$$\text{Middle moment pedestrian} = 3.74 \times \frac{30^2}{8} = 420.75 \text{ kNm}$$

$$\text{Mddle moment from TS} = 340 \times \frac{30}{4} = 2550 \text{ kNm}$$

$$\text{Middle load from diaphragm} = 264.6 \text{ kNm}$$

$$\text{Thus, SLS resultant middle moment} = 264.6 + 2550 + 420.75 + 1338.188 + 2812.5 = 7386.038 \text{ kNm}$$

The Fig 4.9 shows the momoent envelop diagram of bridge deck span of 30 m.

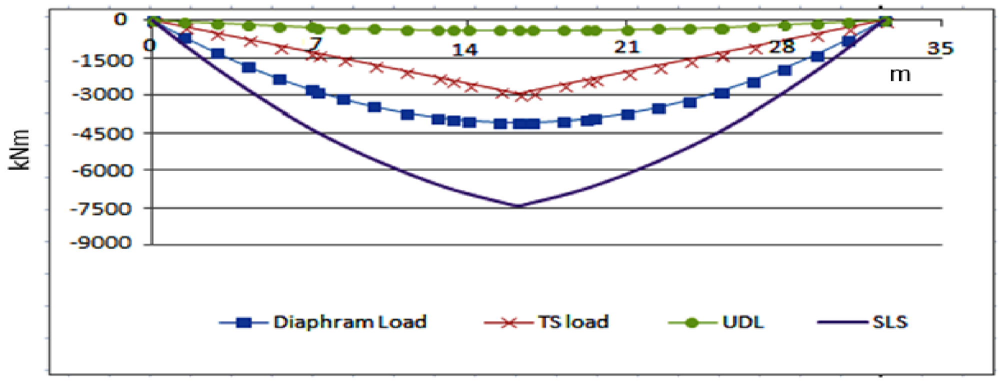

Fig 4.9 Moment Diagram of Span 30m

- *Ultimate Limit State (ULS) Calculations of Span 30m*

For the ULS shall consider following load combinations (BS EN 1990:2002)

So,maximum moment at middle for first combination of ULS
$= 1.35(2812.5 + 264.6) + 1.35(1338.188 + 2550 + 0.4 \times 420.75) = 9630.3438\text{kNm}$ (from Equation 4.1)

So,maximum shear force at beam end for first combination of ULS
$= 1.35(375 + 54) + 1.35(170 + 178.425 + 0.4 \times 56.003) = 1079.7954\text{kN}$ (from Equation 4.1)

The ULS diagram for bridge span of 30m was shown in Fig 4.10.

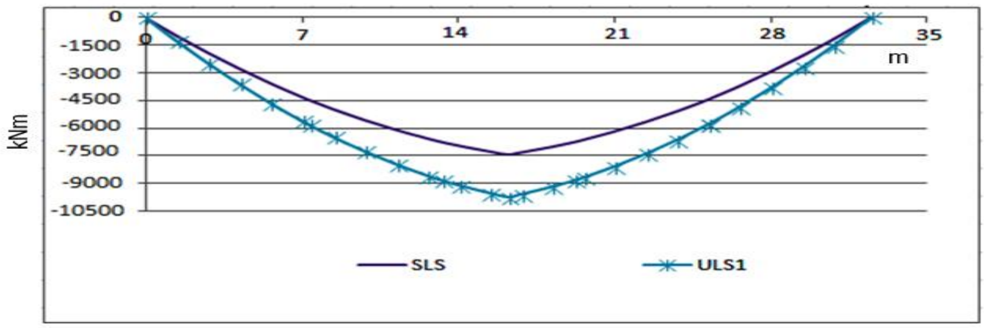

Fig 4.10 Moment Diagram for first Combination of Span 30m

- *Summary of the Base Reactions of Span 30m*

Vertical Load N = 991.766 kN, (Fig 4.11).

Moment around the base center = 691.5412 kNm, (Fig 4.11 and 4.12).

$$M_{max} = \frac{991.766}{(5.1\times1)} + 6\times\frac{691.5412}{(5.1^2)} = 368.3937 < \text{Ground bearing capacity of } 670\ \text{kN/m}^2 \quad \text{(from Equation 4.2)}$$

$$M_{min} = \frac{991.766}{(5.1\times1)} - 6\times\frac{691.5412}{(5.1^2)} = 189.4639 < \text{Ground bearing capacity of } 670\ \text{kN/m}^2 \quad \text{(from Equation 4.2)}$$

So, there have no negative pressure tensioning the base.

Therefore, selected abutment parameters are adequate in terms of structural stability (Fig 4.19)

- *Check for the Bearing Failure.*

Lever arm were considered from the center of the base. {See Fig 4.11 Calculation of Moment for the Determination of Bearing Failure of Span 30m, page 11 in Figes for Research work}.

{See Fig 4.12 Calculation of Moment for the Determination of Bearing Failure of Span 30m, page 12 in Figures for Research work}.

{See Fig 4.13 Summary of Stability Check of Span 30m, page 12 in Figures for Research work}.

Fig 4.14 shows, thesummary of the wall design moment, axial force and shear force at base top level (Figures 3.6 and 3.7in Chapter Three).

{See Fig 4.14 The Summary of all the Forces Acting on the Abutment of Span 30m, page 12 in Figures for Research work}.

➢ *Design of Abutment of Span 30m*

Assume abutment concrete grade = 25N/mm^2
$M_{ed} = 950.46$ kNm (Fig 4.14).

$$k = \frac{950.46\times10^6}{1000\times1175^2\times25} = 0.0275, \quad \text{(from Equation 4.4)}$$

0.0275 < 0.156. So, section is singly reinforced.
$l_a = 0.95$,
$z = 0.95\times1175 = 1116.25$ mm

$$A_s = \frac{950.46\times10^6}{0.87\times500\times1116.25} = 1957.46\text{mm}^2 \quad \text{(from Equation 4.5)}$$

$$A_{smin} = 0.13\times1000\times\frac{1175}{100} = 1527.5\text{mm}^2 \quad \text{(from Equation 4.6)}$$

$1527.5\ \text{mm}^2 < 1935.38\ \text{mm}^2$

Therefore, $A_{sreq} = 1957.46\ \text{mm}^2$

Provide H25 @ 150mm c/cvertical and H20-200horizontal

Therefore, $A_{sproV} = 3270\ \text{mm}^2$

Therefore, $A_{sproH} = 1565\ \text{mm}^2$

- *Check for Shear*

$$\text{Design shear stress on wall} = \frac{366693}{1000 \times 1175} = 0.31\text{N/mm}^2$$

$$\text{ShearCapacityofconcrete}, V_{Rdc} = 372.19 \text{ kN} < 3335.44 \text{ kN} \quad \text{(from Equation 4.7)}$$

Therefore, shear capacity is adequate.

➢ *Bridge Span of 35m Analysis and Design*

- *Shear Force Calculations of Span 35m*

$\text{Shear force from diaphragm} = 18 \times 3 = 54 \text{ kN}$
$\text{Shear force from pedestrain} = (3.7335) \times 35 \times 0.5 = 65.336 \text{ kN}$
$\text{Shear force from TS} = 340 \times 0.5 = 170 \text{ kN}$
$\text{Shear force from dead load} = 25 \times 35 \times 0.5 = 437.5 \text{ kN}$
$\text{Shear force from traffic load} = (11.895) \times 35 \times 0.5 = 208.163 \text{ kN}$
$\text{Total shear force of end of beam at SLS} = 54 + 65.336 + 170 + 437.5 + 208.163 = 934.999 \text{ kN}$

Fig 4.11, shows the shear force diagram for the bridge span of 35 m.

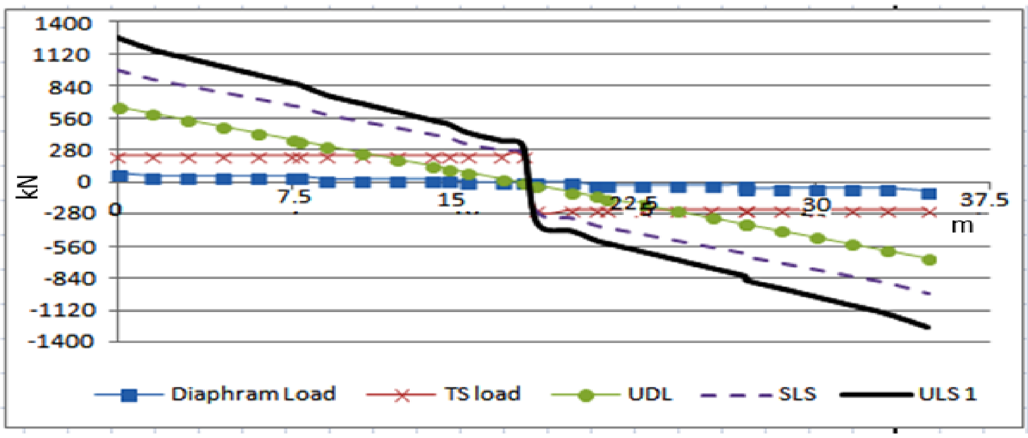

Fig 4.11 Shear Force Diagram of Span 35m

- *Bending Moment Calculations of Span 35m*

$$\text{Middle moment UDL dead} = 25 \times \frac{35^2}{8} = 3828.125 \text{ kNm}$$

$$\text{Middle moment UDL traffic} = 11.895 \times \frac{35^2}{8} = 1621.142 \text{ kNm}$$

$$\text{Middle moment pedestrian} = 3.74 \times \frac{35^2}{8} = 572.688 \text{ kNm}$$

$$\text{Mddle moment from TS} = 340 \times \frac{35}{4} = 2975 \text{ kNm}$$

$$\text{Middle load from diaphram} = 264.6 \text{ kNm}$$

$$\text{Thus, SLS resultant middle moment} = 264.6 + 2975 + 572.688 + 1621.142 + 3828.125 = 9261.555 \text{ kNm}$$

The Fig 4.12 shows the momoent envelop diagram of bridge deck span of 35 m

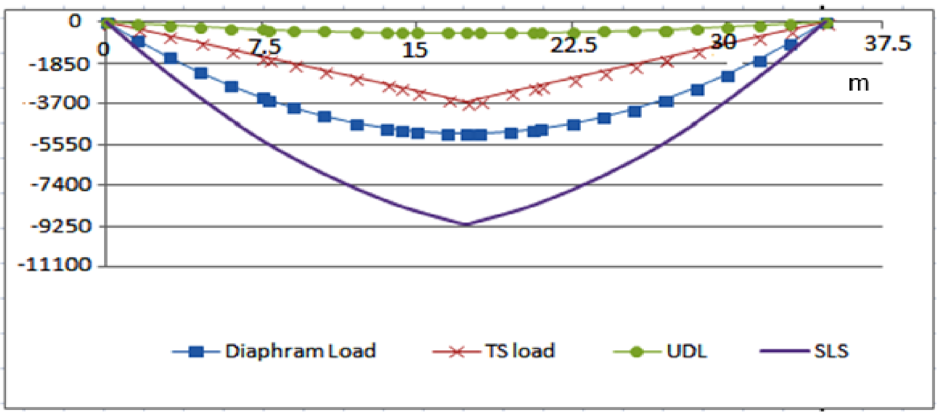

Fig 4.12 Moment diagram of Span 35m

- *Ultimate Limit State (ULS) Calculations of Span 35m*
  For the ULS shall consider following load combinations (BS EN 1990:2002)

So,maximum moment at middle for first combination of ULS
$=1.35(3828.125+264.6)+1.35(2975+1621.142+0.4\times 572.688)=12039.223\text{kNm}$ (from Equation 4.1)

So,maximum shear force at beam end for first combination of ULS
$=1.35(437.5+54)+1.35(170+208.163+0.4\times 65.336)=1209.326\text{kN}$ (from Equation 4.1)

The ULS diagram for bridge span of 35m was shown in Figu 4.13.

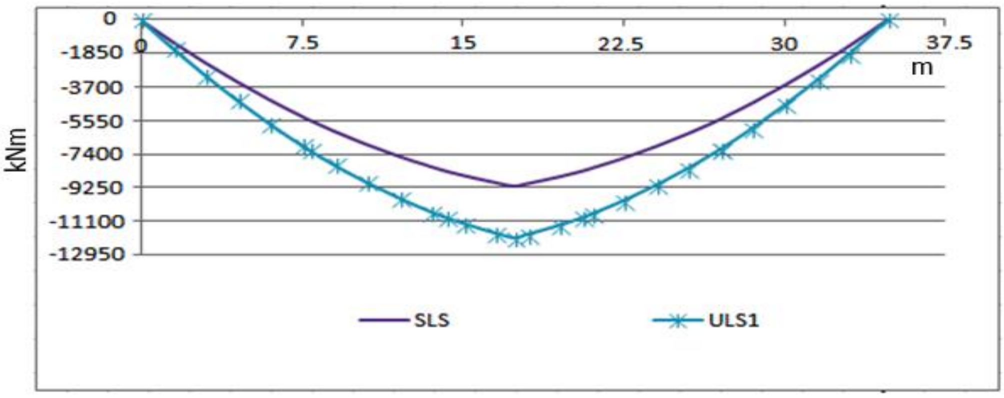

Fig 4.13 Moment Diagram for First Combination of Span 35m

- *Summary of the Base Reactions of Span 35m*

Vertical Load N = 1042.552 kN,(Fig 4.15).

Moment around the base center = 666.4155 kNm,(Fig 4.15 and 4.16).

$$M_{max} = \frac{1042.552}{(5.1\times 1)} + 6\times\frac{666.4155}{(5.1^2)} = 358.1511 < \text{Ground bearing capacity of } 670\ \text{kN/m}^2$$
(from Equation 4.2)

$$M_{min} = \frac{1042.552}{(5.1\times 1)} - 6\times\frac{666.4155}{(5.1^2)} = 50.6929 < \text{Ground bearing capacity of } 670\ \text{kN/m}^2$$
(from Equation 4.2)

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So, there have no negative pressure tensioning the base.

Therefore, selected abutment parameters are adequate in terms of structural stability(Fig 4.17)

- *Check for the Bearing Failure.*

Lever arm were considered from the center of the base.

{See Fig 4.15 Calculation of Moment for the Determination of Bearing Failure of Span 35m, page 12 in Figures for Research work}.

{See Fig 4.16 Calculation of Moment for the Determination of Bearing Failure of Span 35m, page 13 in Figures for Research work}.

{See Fig 4.17 Summary of Stability Check of Span 35m, page 13 in Figures for Research work}.

Fig 4.18 shows, thesummary of the wall design moment, axial force and shear force at base top level (Figures 3.6 and 3.7in Chapter Three).

{See Fig 4.18 The Summary of All the Forces Acting on the Abutment of Span 35m, page 13 in Figures for Research work}.

➢ *Design of Abutment of Span 35m*

Assume Abutment concrete grade is $25 N/mm^2$
$M_{ed} = 961.21$ kNm (Fig 4.18).

$$k = \frac{961.21 \times 10^6}{1000 \times 1175^2 \times 25} = 0.0278, \quad \text{(from Equation 4.4)}$$

0.0278< 0.156. So, section is singly reinforced.

$l_a = 0.95, \qquad z = 0.95 \times 1175 = 1116.25$ mm

$$A_s = \frac{961.21 \times 10^6}{0.87 \times 500 \times 1116.25} = 1979.55 mm^2 \quad \text{(from Equation 4.5)}$$

$$A_{smin} = 0.13 \times 1000 \times \frac{1175}{100} = 1527.5 mm^2 \quad \text{(from Equation 4.6)}$$

$1527.5\ mm^2 < 1979.55\ mm^2$

Therefore, $A_{sreq} = 1979.55\ mm^2$

Provide H25 @ 150 mm c/cvertical and H20-200horizontal

Therefore, $A_{sproV} = 3270\ mm^2$

Therefore, $A_{sproH} = 1565\ mm^2$

- *Check for Shear*

$$\text{Design shear stress on wall} = \frac{366693}{1000 \times 1175} = 0.31 N/mm^2$$

$$\text{ShearCapacityofconcrete}, V_{Rdc} = 372.19\ kN < 320.21\ kN \quad \text{(from Equation 4.7)}$$

Therefore, shear capacity is adequate.

➢ *Bridge Span of 40m Analysis and Design*

- *Shear Force Calculations of Span 40m*

$$\text{Shear force from diaphragm} = 18 \times 3 = 54 \text{ kN}$$
$$\text{Shear force from pedestrain} = (3.7335) \times 40 \times 0.5 = 74.67 \text{ kN}$$
$$\text{Shear force from TS} = 340 \times 0.5 = 170 \text{ kN}$$
$$\text{Shear force from dead load} = 25 \times 40 \times 0.5 = 500 \text{ kN}$$
$$\text{Shearforcefromtrafficload} = (11.895) \times 40 \times 0.5 = 237.9 \text{ kN}$$
$$\text{Totalshearforceofendofbeamat SLS} = 54 + 74.67 + 170 + 500 + 237.9 = 1036.57 \text{ kN}$$

Fig 4.14, shows the shear force diagram for the bridge span of 40 m.

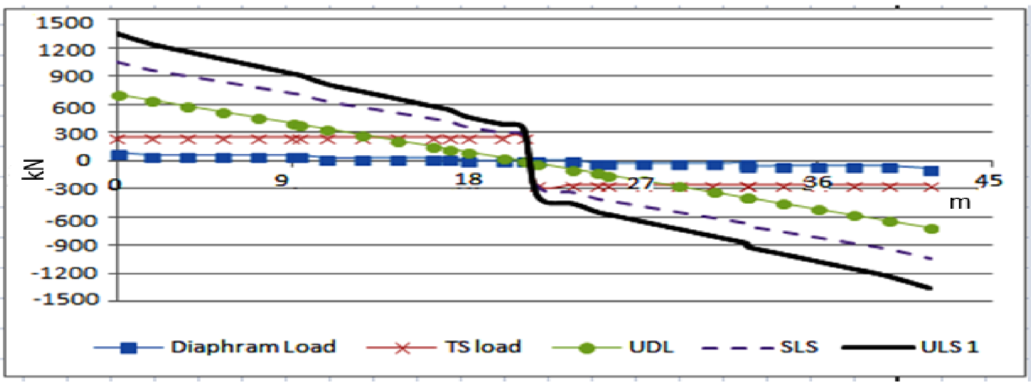

Fig 4.14 Shear Force Diagram of Span 40m

- *Bending Moment Calculations of Span 40m*

$$\text{Middle moment UDL dead} = 25 \times \frac{40^2}{8} = 5000 \text{ kNm}$$

$$\text{Middle moment UDL traffic} = 11.895 \times \frac{40^2}{8} = 2379 \text{ kNm}$$

$$\text{Middle moment pedestrian} = 3.74 \times \frac{40^2}{8} = 748 \text{ kNm}$$

$$\text{Mddle moment from TS} = 340 \times \frac{40}{4} = 3400 \text{ kNm}$$

$$\text{Middle load from diaphragm} = 264.6 \text{ kNm}$$

$$\text{Thus, SLS resultant middle moment} = 264.6 + 3400 + 748 + 2379 + 5000 = 11791.6 \text{ kNm}$$

The Fig 4.15 shows the momoent envelop diagram of bridge deck span of 40 m

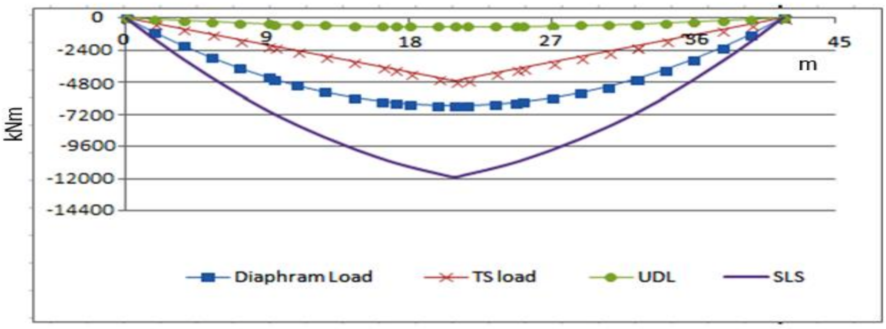

Fig 4.15 Moment Diagram of Span 40m

- *Ultimate Limit State (ULS) Calculations of Span 40m*
  For the ULS shall consider following load combinations (BS EN 1990:2002)

So,maximum moment at middle for first combination of ULS
$=1.35(5000+264.6)+1.35(3400+2379+0.4\times 748)=15312.78\text{kNm}$ (from Equation 4.1)

So,maximum shear force at beam end for first combination of ULS
$=1.35(500+54)+1.35(170+237.9+0.4\times 74.67)=1338.8868\text{kN}$ (from Equation 4.1)

The ULS diagram for bridge span of 40m was shown in Fig 4.16.

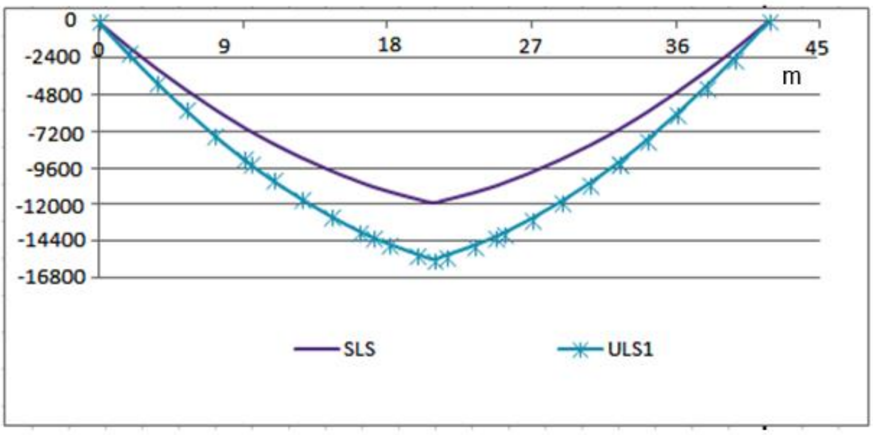

Fig 4.16 Moment Diagram for First Combination of Span 40m

- *Summary of the Base Reactions of Span 40m*
  Vertical Load N = 1093.337kN,(Fig 4.20).

Moment around the base center = 655.1684 kNm,(Fig 4.19 and 4.20).

$$M_{max} = \frac{1093.337}{(5.1\times1)} + 6 \times \frac{655.1684}{(5.1^2)} = 365.5144 < \text{Ground bearing capacity of } 670\ \text{kN/m}^2$$ (from Equation 4.2)

$$M_{min} = \frac{1093.337}{(5.1\times1)} - 6 \times \frac{655.1684}{(5.1^2)} = 63.2452 < \text{Ground bearing capacity of } 670\ \text{kN/m}^2$$ (from Equation 4.2)

So, there have no negative pressure tensioning the base.

Therefore, selected abutment parameters are adequate in terms of structural stability (Fig 4.21)

- *Check for the Bearing failure*

Lever arm were considered from the center of the base.

{See Fig 4.19 Calculation of Moment for the Determination of Bearing Failure of Span 40m, page 13 in Figures for Research work}.

{See Fig 4.20 Calculation of Moment for the Determination of Bearing Failure of Span 40m, page 14 in Figures for Research work}.

{See Fig 4.21 Summary of Stability Check of Span 40m, page 14 in Figures for Research work}.

Fig 4.22 shows, thesummary of the wall design moment, axial force and shear force at base top level (Fig 3.6 and 3.7in Chapter Three). {See Fig 4.22 The Summary of all the Forces Acting on the Abutment of Span 40m, page 14 in Figures for Research work}.

➢ *Design of Abutment of Span 40m*

AssumeAbutmentconcretegradeis 25 N/mm^2
$M_{ed} = 971.93$ kNm (Fig 4.22).

$$k = \frac{971.93 \times 10^6}{1000 \times 1175^2 \times 25} = 0.02816 \ , \qquad \text{(from Equation 4.4)}$$

0.02816< 0.156. So, section is singly reinforced.
$l_a = 0.95$, $z = 0.95 \times 1175 = 1116.25$ mm

$$A_s = \frac{971.93 \times 10^6}{0.87 \times 500 \times 1116.25} = 2001.63 mm^2 \qquad \text{(from Equation 4.5)}$$

$$A_{smin} = 0.13 \times 1000 \times \frac{1175}{100} = 1527.5mm^2 \qquad \text{(from Equation 4.6)}$$

$1527.5\ mm^2 < 2001.63\ mm^2$

Therefore, $A_{sreq} = 2001.63\ mm^2$

Provide H25 @ 125 mm c/c,vertical and H20-175,horizontal

Therefore, $A_{sproV} = 3930\ mm^2$

Therefore, $A_{sproH} = 1800\ mm^2$

- *Check for Shear*

$$\text{Design shear stress on wall} = \frac{366693}{1000 \times 1175} = 0.312\ N/mm^2$$

$$\text{ShearCapacityofconcrete}, V_{Rdc} = 346.41\ kN < 308.77\ kN \qquad \text{(from Equation 4.7)}$$

Therefore, shear capacity is adequate.

➢ *Horizontal Load due to Water current on Pier*

$$P = 52kV^2 \tag{4.8}$$

Assume Square ended pier, $k = 1.5$
River velocity, $v = 1.1$ m/s

$$P = 52 \times 1.5 \times 1.1 \times 1.1 = 94.4\ kg/m^2 = 0.94\ kN/m^2 \qquad \text{(from Eqution 4.8)}$$

Force due to the debris, $P = 52kV^2 = 0.63\ N/m^2$, where; $k = 1$

Assume length of 11.6m with depth of 1.2m debris

Therefore, maximum debris force on pier $= 0.63 \times 1.2 \times 11.6 = 8.77$ kN
Force due to log impact $= 0.1WV = 0.1 \times 20 \times 1.1 = 2.2$ kN(4.9)

Water current force shall be applied over the height of pier and debris and log impact load to be applied at the, HFL level normal to the river flow direction.

- *Pier Capping Beam Design*

Maximum moment = 1202 kNm
Maximum shear = 1197 kN
Assume 1000×1000 mmcappingbeamonpier
Assumepiershaftandcappingbeamconcretegrade is $25\mathrm{N/mm^2}$

$$k = \frac{1202 \times 10^6}{1000 \times 925^2 \times 25} = 0.05619 \quad , \qquad \text{(from Equation 4.4)}$$

0.05619 < 0.156. So, section is singly reinforced.

$l_a = 0.95, \qquad z = 0.95 \times 925 = 879 \text{ mm}$

$$A_s = \frac{1202 \times 10^6}{0.87 \times 500 \times 879} = 3144.49\mathrm{mm}^2 \qquad \text{(from Equation 4.5)}$$

$$A_{smin} = 0.13 \times 1000 \times \frac{925}{100} = 1202.5\mathrm{mm}^2 \quad \text{(from Equation 4.6)}$$

$1202.5\ \mathrm{mm}^2 < 3144.49\ \mathrm{mm}^2$

Therefore, $A_{sreq} = 3144.49\mathrm{mm}^2$

Provide H25 @ 125 mm c/c,bottom/top and H20-200side bars,

Therefore, $As_{prov}V = 3984\ \mathrm{mm}^2$

Therefore, $As_{prov}H = 1565\ \mathrm{mm}^2$

- *Check for Shear*

$$\text{Design shear stress on beam} = \frac{1197000}{1000 \times 925} = 1.29\ \mathrm{N/mm^2}$$

$$\text{Shearcapacityofconcrete}, V_{Rdc} = 356.32\ \mathrm{kN} < 1197\ \mathrm{kN} \qquad \text{(from eqution 4.7)}$$

- *Shear Links*

(BS EN 1992-1-1:2004, Cl 6.2.3)

$$V_{Rmax} = 0.124\ bd\left(1 - \frac{f_{ck}}{250}\right)f_{ck} = 2580.75\ \mathrm{kN} > V_{ed} = 1197\ \mathrm{kN} \tag{4.10}$$

So, valueof$\theta < 22$

$$\frac{A_{sw}}{s} = \frac{V_{ed}}{(0.78\ df_{yk} \cot\theta)} = 1.33 \tag{4.11}$$

Assume H10 @ 100mmspace

$$\frac{A_{sw}}{s} = 2 \times \frac{78}{100} = 1.56 \qquad \text{(from eqution 4.11)}$$

Hence, adequate.

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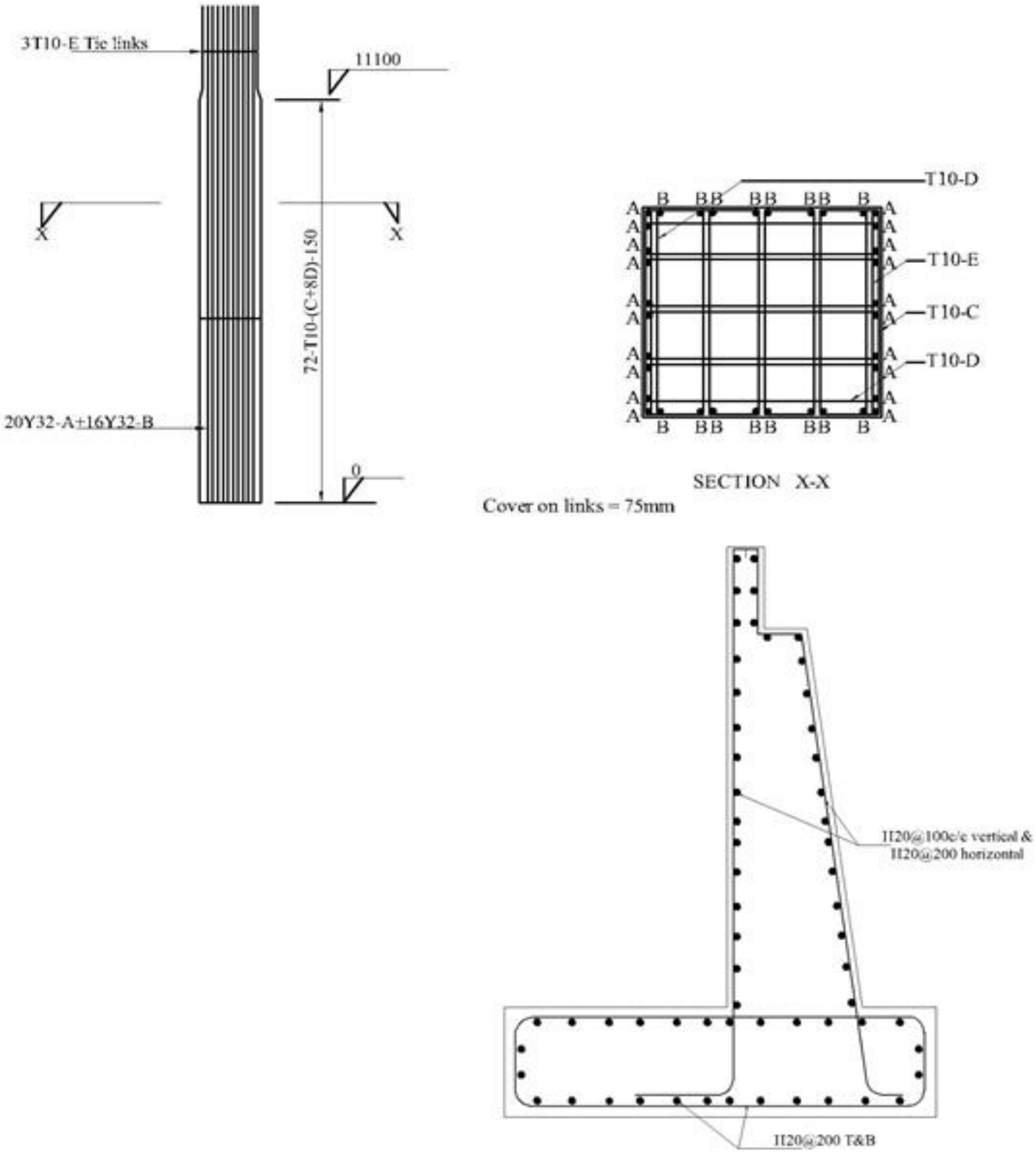

Fig 4.17 Detailing of the Abutment and Pier

B. Analysis of Results

➢ *Bridge Span of 23.2m*

The shear force diagram shown in Figure 4.18 clearly shows thattotal shear force of bridge span of 23.2m is 695.29 kN at SLS, while the maximium shear force at beam end for first combination of ULS is 903.56 kN.

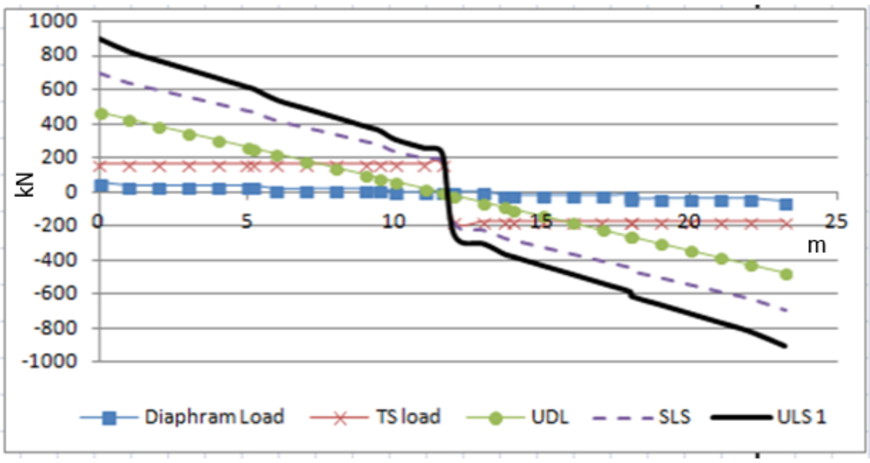

Fig 4.18 Shear Force Diagram of Span 23.2m

Figure 4.19 shows the bending of bridge span of 23.2m. The maximum moment is 4970.09 kNm. The diagram also showed the diaphragm loads, tandem system loads, uniformly distributed load and serviceability limit state at varying span distances of the bridge deck.

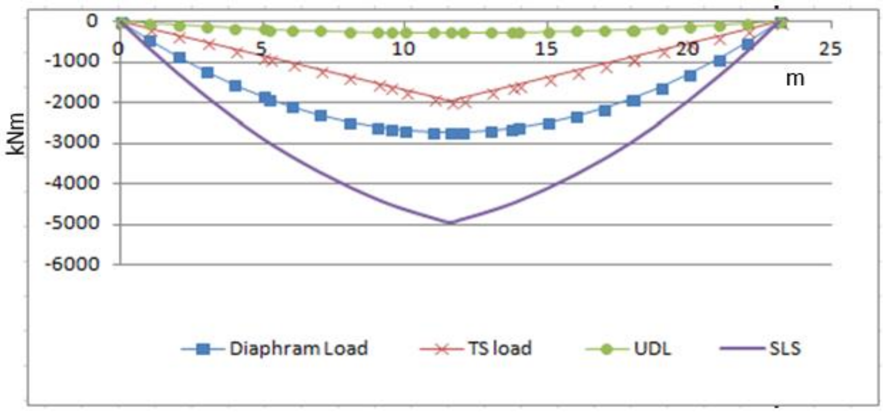

Fig 4.19 Moment Diagram of Span 23.2m

The maximum moment at middle of the beam for first combination of ULS is 6506.16 kNm as shown in Figure 4.20.

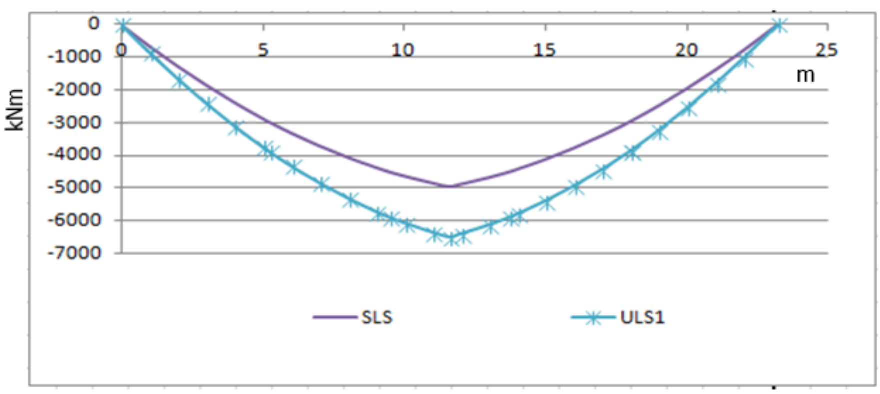

Fig 4.20 Moment at Middle of the Beam for First Combination Diagram of Span 23.2m

- *Abutment Design of Span 23.2m*

The summary of all the forces acting on the abutment for bridge span of 23.2m, which include the design moment, design shear force and axial force having 932.90 kNm, 366.69 kN and 682.48 kN respectively, as shown in Table 4.23.

{See Fig 4.23 The Summary of all the Forces Acting on the Abutment of Span 23.2m, page 14 in Figures for Research work}.

➢ *Bridge Span of 25m*

The shear force diagram shown in Figure 4.21, clearly shows thattotal shear force of bridge span of 25 m is 731.86 kN at SLS, while the maximium shear force at beam end for first combination of ULS is 950.21 kN.

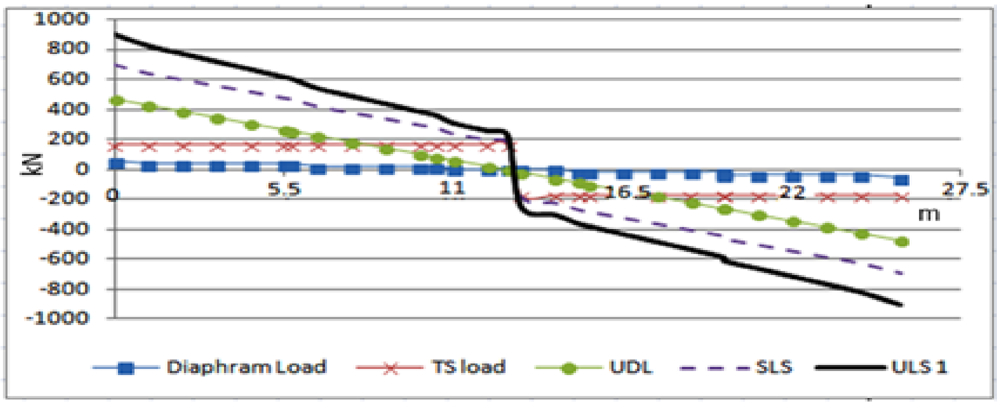

Fig 4.21 Shear Force Diagram of Span 25m

Figure 4.22 shows the bending of bridge span of 25m. The maximum moment is 5564.21kNm. The diagram also show the diaphragm loads, tandem system loads, uniformly distributed load and serviceability limit state at varying span distances of the bridge deck.

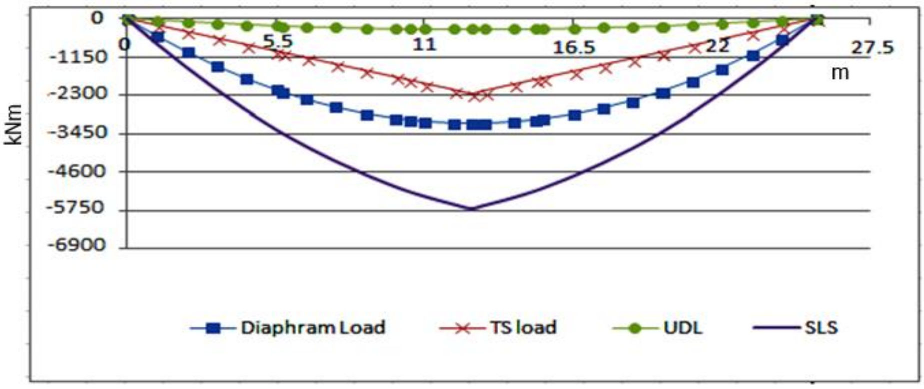

Fig 4.22 Moment Diagram of Span 25m

The maximum moment at middle of the beam for first combination of ULS is 7274.84 kNm as shown in Figure 4.23.

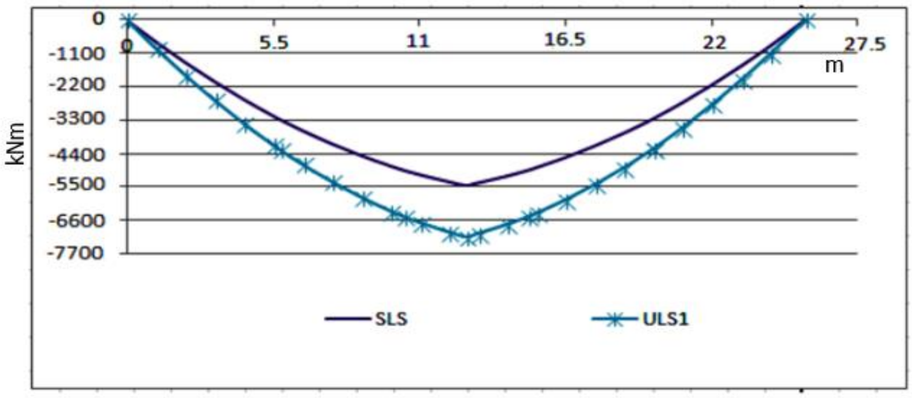

Fig 4.23 Moment at Middle of the Beam for First Combination Diagram of Span 25m

➢ *Abutment Design of Span 25m*

The summary of all the forces acting on the abutment for bridge span of 25m, which include the design moment, design shear force and axial force having 939.76 kNm, 366.69 kN and 708.21kN respectively, as shown in Table 4.24.

{See Fig 4.24 The Summary of all the Forces Acting on the Abutment of Span 25m, page 14 in Figures for Research work}.

➢ *Bridge Span of 30m*

The shear force diagram shown in Figure 4.24 shows that thetotal shear force of bridge span of 30 m is 833.43 kN at SLS, while the maximium shear force at beam end for first combination of ULS is 1079.79 kN.

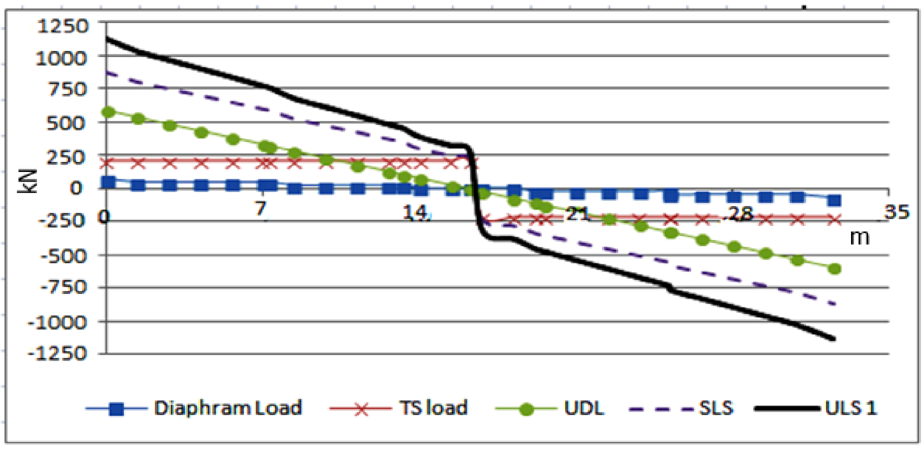

Fig 4.24 Shear Force Diagram of Span 30m

The maximum moment is 7386.04 kNm, as shown in Figure 4.25. The bridge span of 30m in consideration, also show the diaphragm loads, tandem system loads, uniformly distributed load and serviceability limit state at varying span distances of the bridge deck.

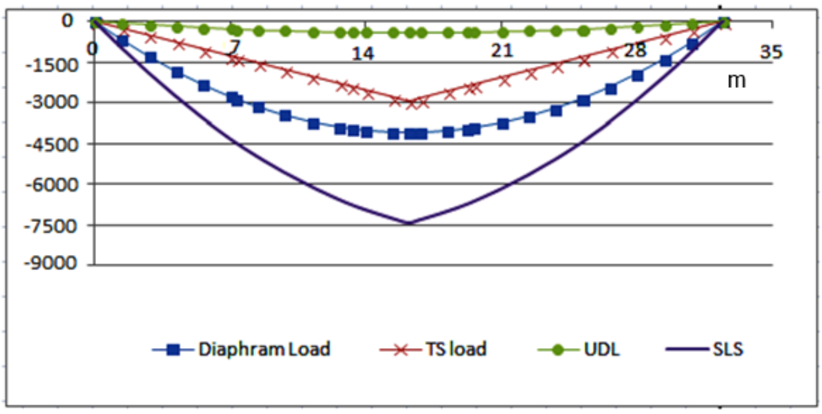

Fig 4.25 Moment Diagram of Span 30m

The maximum moment at middle of the beam for first combination of ULS is 9630.34 kNm as shown in Figure 4.26.

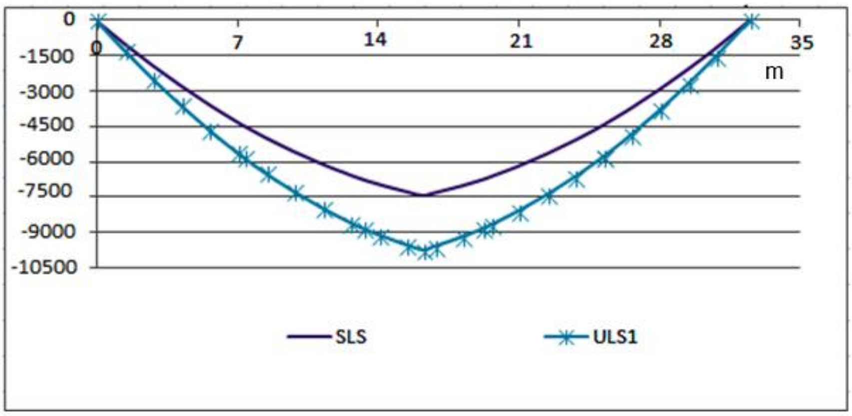

Fig 4.26 Moment at Middle of the Beam for First Combination Diagram of Span 30m

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➢ *Abutment Designof Span 30m*

The summary of all the forces acting on the abutment for bridge span of 30m, which include the design moment, design shear force and axial force having 950.48 kNm, 366.69 kN and 779.70 kN respectively, as shown in Table 4.25.

{See Fig 4.25 The Summary of all the Forces Acting on the Abutment of Span 30m, page 14 in Figures for Research work}.

➢ *Bridge Span of 35m*

In Figure 4.27, has shown that,the total shear force of the bridge span of 35m is 934.10 kN at SLS, while the maximium shear force at beam end for first combination of ULS is 1029.33 kN.

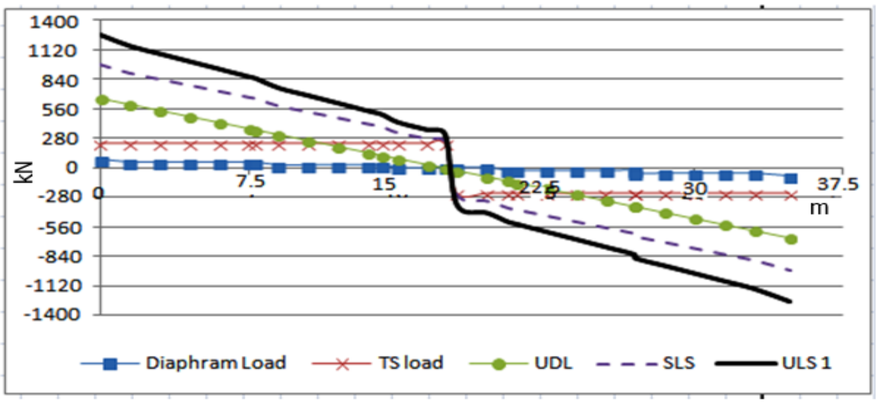

Fig 4.27 Shear Force Diagram of Span 35m

The bending moment for bridge span of 35m, has the maximum moment of9261.56 kNm. The diagram also show the diaphragm loads, tandem system loads, uniformly distributed load and serviceability limit state at varying span distances of the bridge deck.

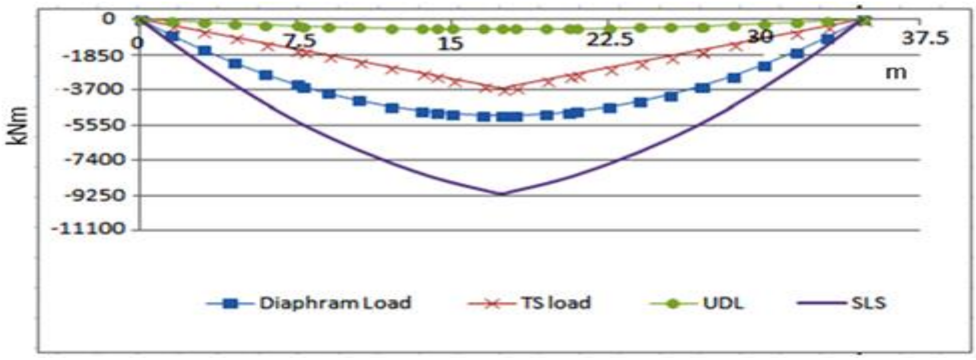

Fig 4.28 Moment Diagram of Span 35m

In Figure 4.29, the maximum moment at middle of the beam for first combination of ULS is 12039.22 kNm.

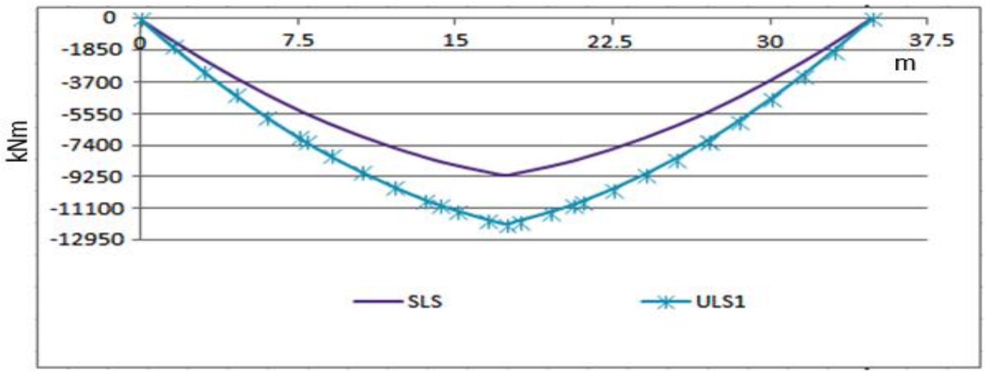

Fig 4.29 Moment at Middle of the Beam for First Combination Diagram of Span 35m

➢ *Abutment Design of Span 35m*

Fig 4.26 shows the summary of all the forces acting on the abutment for bridge span of 35m, which include the design moment, design shear force and axial force having 961.21 kNm, 366.69 kN and 851.19 kN respectively.

{See Fig 4.26 The Summary of all the Forces Acting on the Abutment of Span 35m, page 15 in Figures for Research work}.

➢ *Bridge Span of 40m*

The shear force diagram shown in Figure 4.30, shows that thetotal shear force of bridge span of 40 m is 1036.57 kN at SLS, while the maximium shear force at beam end for first combination of ULS is 1338.89 kN.

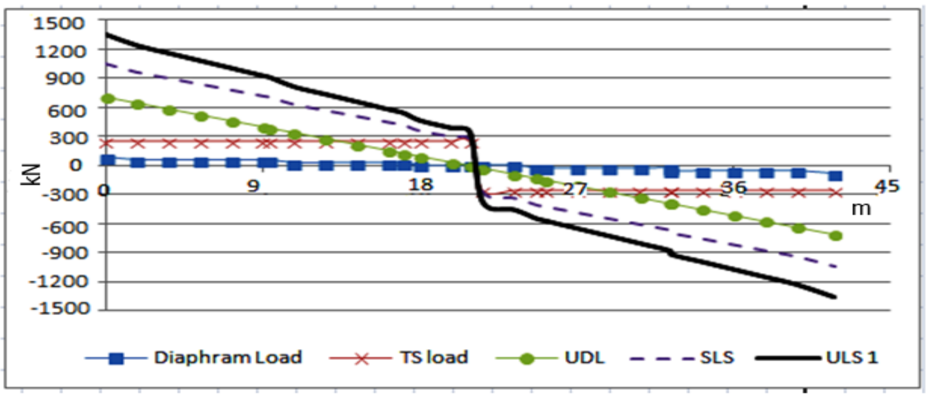

Fig 4.30 Shear Force Diagram of Span 40m

Figure 4.31 shows the bending moment of a bridge span of 40m. The maximum moment is 11791.6 kNm. The diagram also show the diaphragm loads, tandem system loads, uniformly distributed load and serviceability limit state at varying span distances of the bridge deck.

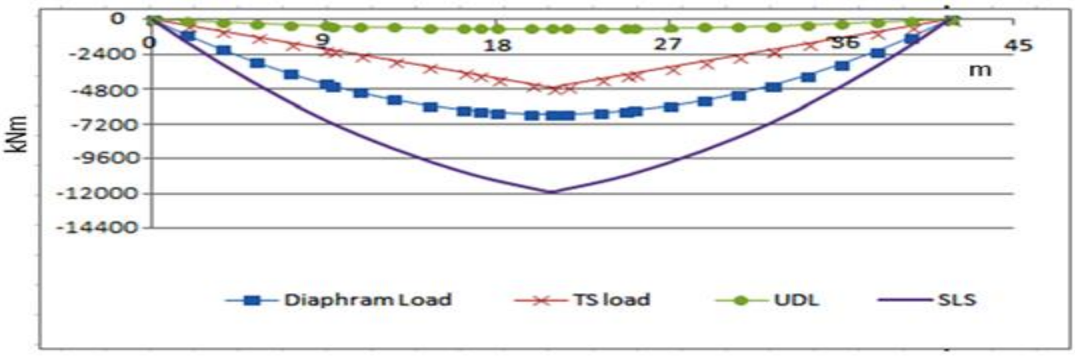

Fig 4.31 Moment Diagram of Span 40m

The Figure 4.32 has maximum moment at middle of the beam for first combination of ULS as15312.78 kNm.

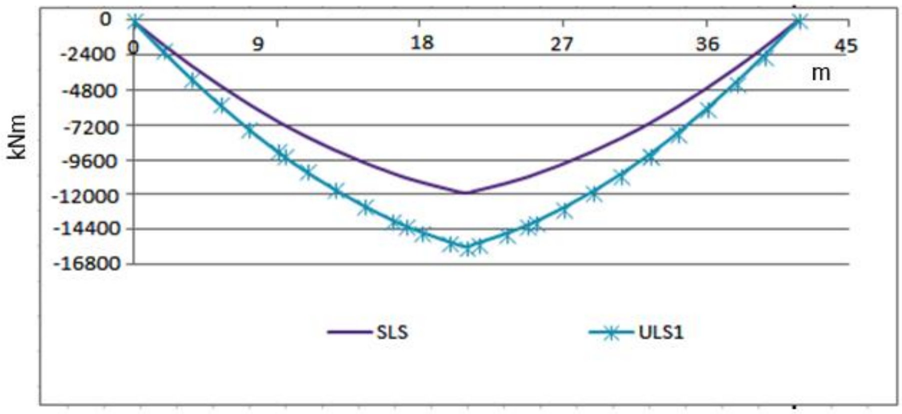

Fig 4.32 Moment at Middle of the Beam for First Combination Diagram of Span 40m

- *Abutment Design of Span 40m*

The summary of all the forces acting on the abutment for bridge span of 30m, which include the design moment, design shear force and axial force having 971.93 kNm, 366.69 kN and 855.18 kN respectively, as shown in Table 4.27.

{See Fig 4.27 The Summary of all the Forces Acting on the Abutment of Span 40m, page 15 in Figures for Research work}.

- *Computer Programs Loading*

{See Fig 4.28 Maximum Moment at Middle for First Combination of ULS, page 15 in Figures for Research work}. The maximum moment at middle for first combination of ultimatre limit state at various spans of 23.2m, 25m, 30m, 35m, and40m, from manual calculation and the developed computer program, is given in Table 4.28. The percentage differences between values obtained for the various spans are less than 1%.

{See Fig 4.29 Shear Force at Beam end for First Combination of ULS, page 15 in Figures for Research work}. The shear forces at the beam end for the first combination of ultimate limit state at various spans 23.2m, 25m, 30m, 35m, and40m,from manual calculation and the developed computer program, are given in Table 4.29. The percentage differences between values obtained for the various spans are also less than 1%.

- *Forces Acting on the Abutment*

{See Fig 4.30 Moment, M_{ed} (kNm), page 16 in Figs for Research work}.

The maximum moments on the abutment at various spans 23.2m, 25m, 30m, 35m, and40m, from manual calculation and the developed computer program, are given in Table 4.30. The percentage differences between values obtained for the various spans are less than 1%..

{See Fig 4.31 Axial Forced, N_{ed} (kN), page 16 in Figures for Research work}.

For spans 23.2m, 25m, 30m, 35m, and40m, the axial forces, $N_{ed,}$ from manual calculation and the developed computer program, are given in Table 4.31. The percentage differences between 23.2m and 25m are 0.00880% and 0.00423% respectively. These results show that, the manual calcualtions and those from the developed program are close.

{See Fig 4.32 Area of Steel Required (mm^2), page 16 in Figures for Research work}.

For spans 23.2m, 25m, 30m,35m, and 40 m, the area of steel required from manual calculation and the developed computer program, are given in Fig 4.32. The percentage differences for 23.2m and 25m spans are 0.1947% and 0.1113% respectively. These results show that the manual calcualtions and the ones from the developed program are close.

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CHAPTER FIVE
CONCLUSION AND RECOMMENDATION

A. Conclusion

The study's objectives were to create a java-based computer program for the quick and accurate design and analysis of bridge substructure components on shallow foundations and to manually analyze and design bridge substructures. This research work did analysis and design using Eurocodes 2, on bridge abutment, pier and pier cap beam. Design moments, design shear forces, axial forces, the area of the steel, and the number of steel reinforcement bars are among the outcomes that can be derived from both manual and computer methods. The results of the manual and computer program were compared for similarities, and the he found out that the percentage difference of moments, shear forces, axial forces and the area of steel were negligible. All designs were carried out in accordance with provisions and other standard literature, as well as the Eurocodes BS EN 1992-1-1, BS EN 1991-1-4, and BS EN 1991-2.

B. Contribution to Knowledge

For the purpose of analyzing and designing bridge substructures in accordance with Eurocodes and other relevant standards, this study performed both manual design and the development of a Java-based computer program. The developed program would be a valuable tool for practicing structural engineers as well as a useful interactive program for teaching structural engineering students.

C. Recommendations

The study designed and analyzed bridge substructures in Eurocode, using both a manual approach and a Java-based computer program. Reinforced concrete was used to build the bridges. On a profound establishment, future specialists can plan projections and docks physically or utilizing a PC. However, Eurocodes were used to write the majority of the computer programs developed in this study. The program can be extended to include the design of bridge substructures using other internationally recognized bridge design codes in subsequent projects. Last but not least, the developed software is able to connect to common CAD applications like AutoCAD to generate working drawings and details for the bridge substructures.

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REFERENCES

[1]. Al-Sarraf, et al., (2005); Analysis of Composite Bridge Superstructures Using Modified GRillage Method.
[2]. Anwar, N. (2015); Analysis and Modeling of Bridge Substructure; Asian Institute of Technology (AIT) Consulting
[3]. Arockiasamy, M., Narongrit, B., Sivakumar, M., "State-of-the-Art of Integral Abutment Bridges: Design and Practice," American Society of Civil Engineers (ASCE) Journal of Bridge Engineering, Vol. 9, No. 5, September/October 2004, pp. 497-506.
[4]. Arora, K. R (2003); "Soil Mechanics and Foundation Engineering (Sixth Edition)"; Standard Publishers Distributors, New Delhi.
[5]. Barker, R.M., Duncan, J.M., Rojiani, K.B., Ooi, P.S.K., Tan, C.K., and Kim, S.G., Manuals for the Design of Bridge Foundations, NCHRP Report 343, Transportation Research Board, December 1991.
[6]. Bouassida, Y., et al (2010), Bridge Design to Eurocodes, worked examples; Worked examples presented at the Workshop "Bridge Design to Euro codes", Vienna, CEN 2002. Euro code: Basis of structural design. EN 1990: 2002. European Committee for Standardization (CEN): Brussels.
[7]. Briaud, J.-L., et al, Pier and Contraction Scour in Cohesive Soils, NCHRP Report 516, Transportation Research Board, 2004.
[8]. CEN 2004. Euro code 7: Geotechnical design - Part 1: General rules. EN 1997-1:2004 (E), November 2004, European Committee for Standardization: Brussels.
[9]. CEN 2005. Euro code 8: Design of structures for earthquake resistance - Part 5: Foundations, retaining structures and geotechnical aspects. EN1998-5:2004 (E), May 2005, European Committee for Standardization: Brussels.
[10]. CEN 2007. Euro code 7: Geotechnical design - Part 2: Ground investigation and testing. EN1997-2:2007 (E), March 2007, European Committee for Standardization: Brussels.
[11]. Davaine, L. 2010a. Global analysis of a steel-concrete composite two-girder bridge according to Euro code 4, Note for Workshop "Bridge design to Euro codes" to be held in Vienna, 4-6 October 2010, June 2010, 39 pages.
[12]. Davaine, L. 2010b. Excel sheet with a synthesis of the reactions and supports, e-mail to TC 250/HGB: June 09, 2010 6:24 pm.
[13]. Davaine, L. 2010c. Supplement for support reactions, Private written communication, Workshop "Bridge design to Eurocodes", Vienna, 4-6 October 2010, June 2010, 16 August 2010, 2 pages.
[14]. De Beer, E., And Marten (1957); "Method of Computation on Upper limit for the Influences of Heterogeneity of Sand Layers in the Settlement of Bridges"; Proc. 4th Int. Conf. On SMFE, London, Vol.1.
[15]. Federal Highway Administration (FHWA), Seismic Design of Bridges, Design Examples No. 1 through 7, Publication Nos. FHWA-SA-97-006 through FHWA-SA-97-012, October 1997.
[16]. Federal Highway Administration (FHWA), Steel bridge Design Handbook, Substructure Design Publication No. FHWA-IF-12-052 - Vol. 16 2012
[17]. Gifford, D.G., Wheeler, J.R., Kraemer, S.R., and McKown, A.F. (1980), Spread Footings for Highway Bridges, Report No. FHWA/RD-86/198, 1986.
[18]. Hannigan, P.J., Goble, G.G., Thendean, G., Likins, G.E., and Raushe, F. (1997), Design and Construction of Driven Pile Foundations, Report Nos. FHWA-HI-97-013 and -014, 1997.
[19]. Kesarwani, S., et al., (2018); Analysis of T-Beam Along with Deck Slab by Courbon's Method.
[20]. Kolias, B. 2010a. Squat piers with seismic isolation. Summary of seismic design results. Note for Workshop "Bridge design to Euro codes" to be held in Vienna, 4-6 October 2010, June 2010, 25pages.
[21]. Kolias, B. 2010b. Flexible piers with limited ductile behaviour. Summary of seismic design results.
[22]. Mahadevan, A.K., (2005); To Design a Code for the Building of Skew Bridges.
[23]. Malakatas, N. 2010. Example of application for Wind actions on bridge deck and piers, Report for Workshop "Bridge design to Eurocodes", Vienna, 4-6 October 2010, January 2011, 20 pages.
[24]. Manohar, et al., (2018); Finite Element Analysis of Slabs, Cross girders and Main girders in RC T-Beam Deck Slab Bridge.
[25]. MELT-Ministère de l'Equipement, du logement et des transports 1993. Règles Techniques deConception et de Calcul des Fondations des Ouvrages de Génie Civil (in French: Technical Rulesfor the Design of Foundations of Civil Engineering Structures). Cahier des clauses techniques générales applicables aux marchés publics de travaux, FASCICULE N°62 -Titre V, TextesOfficiels N° 93-3 T.O., 182 pages.
[26]. Mitchell, D., Collins, M., Bhide, S., and Rabbat, B., "AASHTO LRFD Strut-and-Tie Model Design Examples, Portland Cement Association, 2004.
[27]. Note for Workshop "Bridge design to Euro codes", Vienna, 4-6 October 2010, July 2010, 14pages.
[28]. O'Neil, M.W., and Reece, L.C., Drilled Shafts: Construction Proceedures and Design Methods, Report FHWA-IF-99-025, August, 1999.
[29]. Patel, A.M. Analysis and design of bridge substructures using VB.NET, Department of Civil Engineering Indian Institute of Technology Roorkee, Roorkee-247 667 (India) June, 2008.
[30]. Prakash, S. (1979); "Analysis and Design of Foundations and Retaining Structures"; Sarita Prakashan, New Delhi.
[31]. Prayerful, N.K., and Hanumant, B., (2015); Comparative Analysis of T-Beam Bridge by Rotational Method and Staad Pro. Ass. Prof, Civil engineering, Jay Want College of Engineering Sangli, India, B.E. Student. Civil Engineering, Jay Want College of Engineering Sangli, India.

ISSN No:-2456-2165

[32]. Richardson, E.V., Davis, S.R., Evaluating Scour at Bridges, 4th Edition, Hydraulic Engineering Circular (HEC) 18, FHWA Publication No. FHWA NHI 01-001, May 2001.
[33]. Shama, A. A., Mander, J. B., and Chen, S. S., "Seismic Investigation of Steel Pile Bents: II. Retrofit and Vulnerability Analysis," Earthquake Spectra, Vol. 18, No. 1, Feb. 2002, pp. 143-160.
[34]. Shama, A. A., Mander, J. B., Blabac, B. A., and Chen, S. S., "Seismic Investigation of Steel Pile Bents: I. Evaluation of Performance," Earthquake Spectra, Vol. 18, No. 1, Feb. 2002, pp. 121 - 142.
[35]. Sing, V. (1981); “Wells and Caissons I Second Edition)”; Nem Chand & Bros., Roorkee
[36]. Soumya and Umadevi, 2015); Comparative Study of Courbon’s Method and Finite Element Method of RC T-Beam and Deck Slab Bridge.
[37]. Taylor, A. W. Rowell, R. B. Breen, J. E., Design and Behavior of Thin Walls in Hollow Concrete Bridge Piers and Pylons, Report CTR 1180-1F, Center for Transportation Research, University of Texas at Austin, 1990.
[38]. Terzaghi, K. and Peck, R.B. (1976); “Small Mechanics in Engineering Practice”; John Wiley and Sons Inc., New York.
[39]. Wassef, W., et al, Integral Steel Box-Beam Pier Caps, NCHRP Report 527, Transportation Research Board, 2004.