Chapter Three

Loading on Bridges

3.1. Primary design of the bridge:

Considerations to be taken into account for the design of bridges:

1- Shape of the bridge:

1. 1-Planning the horizontal and vertical bridge.
2. Sections of construction allowed for bridge.
3. Study of traffic during the implementation of the bridge.
4. 4-Main Services [3].

2-The estimated cost of the bridge:

1. The nature of the soil at the site of the bridge.
2. The total time required to create a bridge.
3. Implementation details.
4. Requirements for future extension.
5. Maintenance requirements.

3- Coefficients safety:

1- Safety factors during the Implementation.

2- Safety factors after the implementation of the bridge (Safety factor required for traffic after the implementation and work of the bridge).

4-External shape of the bridge:

1- Must be external shape of the bridge is consistent and appropriate for the buildings Neighboring to the bridge.

2- Must be external shape of the bridge homogeneous with nature on the bridge.

5-Requirements for bridges crossing the waterways:

1- Net height of the bridge as well as along the span.
2- The distance between the vertical Stents of the bridge and the impact on the amount of water pedestrians between those Stents.
3- Safety for traffic of navigation within the waterway during the implementation of the bridge.
4- Requirements of navigation traffic within the waterway after the implementation of the bridge.

Requirements for bridges with precast concrete and prestressconcrete:

1- Along the span bridge.
2- The comparison between the cost of the Factory and the cost of concrete cast-in situ.
3- The weight of the various structural elements of the bridge.
4- Transportation costs and the structural elements installed location of the bridge.
5- Costs of materials used and the production rates.
6- Obstacles Planning:

There are many bridges that need to be redesigned because of improvements that occur on the road planning (such as increasing the number of lanes traffic and change the height of the water level in the waterway, Increase the capacity of general services of water and sanitation telephones and related structure construction of the bridge) therefore you must finish the finadesign of these elements in an early stage to avoid increasing costs of constructing the bridge , As well as increasing the time required to re-design .

7-Along the span bridge:

The length of the bridge span depends on several factors, including:

1- Extensions services sections that cross bottom of the structure of the bridge.
2- Sites and the type of vertical stents of the bridge.

Type the structure of the bridge:

The choice of the type of the structure of the bridge depend on the relationship between the depth of the span bridge [3] .

Types of reinforced concrete bridges and the requirements of ACI COMMITTEE **OF BRIDGES:**

A-ReinforcedConcrete Slab Bridge:

- Structural requirements :

1- Proportion to the depth of the bridges along the span simple support (1/15)
2- Proportion to the depth of the bridges along the span continuous support (1/20)and (1/ 24).
3- This type of bridges for spans ranging between (5- 14 m).
4- In the case of the use of hollow core slab or voidedslab and uses of the span, which ranges between (12 -20m), as shown in Fig. (3.1).

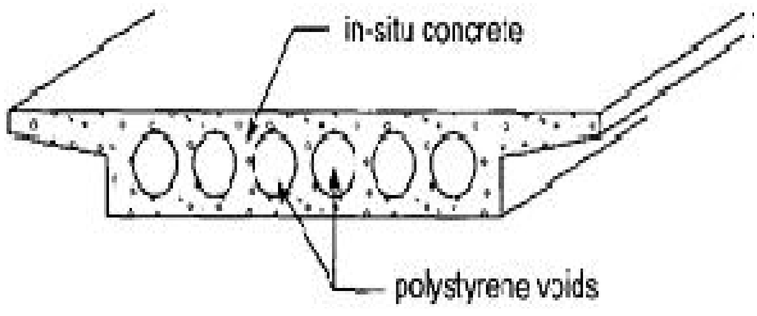


Fig. (3.1): Voided Slab Bridge.

-Implementation requirements:

1- This is kind is classify of simplest types of bridges.
2- The implementation of this bridge is requires less time than any other type.

- Maintenance Requirements :

1- This kind of bridges is requires less maintenance.
2- If you want to increase the width of the bridge in the future, this it may be difficult compared to any other type.

B- Bridges reinforced concrete (TSection), as shown inFig. (3.2):

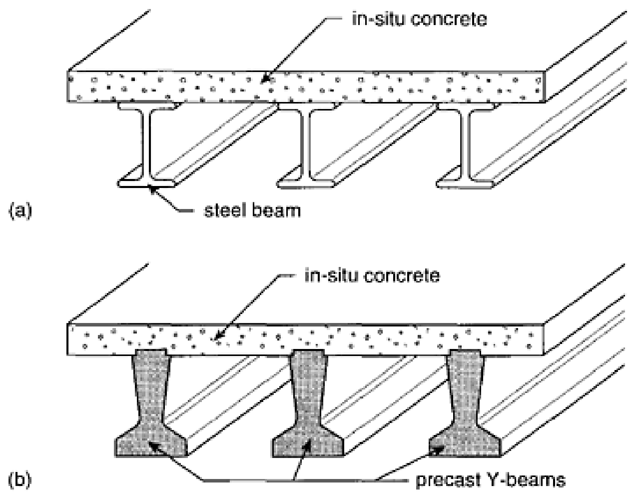


Fig.(3.2) :T- Section Bridges.

- Structural requirements:

1- Proportion of the depth to the length of the span for simple support bridges (1/ 15).
2- Proportion of the depth the length of the span for continuous support bridges (1/16).
3- This type of bridges are used for span which ranging between(9-25 m).

-Implementation requirements:

1- Needs this kind of bridges to the severity of the complex.

2- Needs this kind of bridges to good a final finishing forall exterior surfaces of the bridge.
3- Needs this kind of bridges to greater time for implementation compared to the previous type.

-Maintenance Requirements:

Require less maintenance with the exception of maintenance struts.

- Reinforced concrete bridges with box girder,as shown in Fig. (3.3).

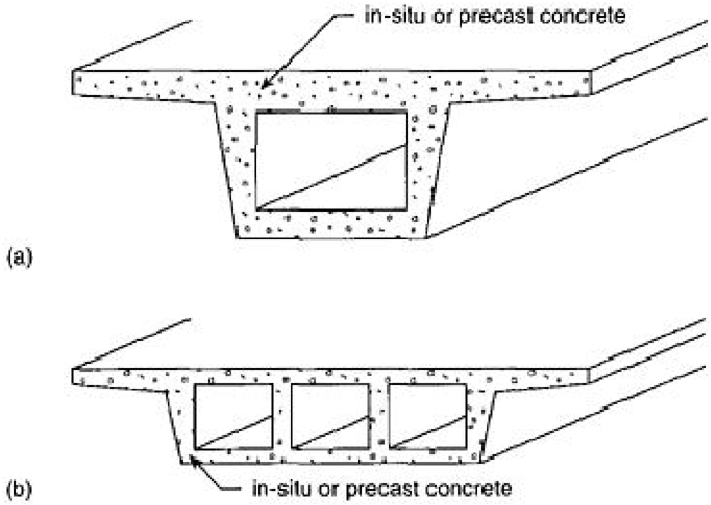


Fig. (3.3): boxGirder Bridge.

1- Proportion of depth to the length of the span for simple support bridges (1/18).
2- Proportion of the depth the length of the span for continuous support bridges (1/19).
3- You can use this type of bridges for curved planning (Curved bridges).
4- This type of bridge uses for pans ranging between (25 – 60 m).

- Implementation requirements:

1- Needs this kind of bridges to the severity of the complex.

2- Does not need this kind of bridges to the finishing of the interior surfaces [3].

3.2. The final design of the bridge:

- Calculate the loads acting on the bridge:

1- Dead Loads:

1- The weight of the elements of the structure of the bridge (such as Beams, slabs side aisles …etc).

2- Weight finishes on the bridge such as:
Finishing floors, Layers isolate the humidity, Signals, Cables, pipes (etc...).

Loads of implementation and installation:

This loads are arise from various stages of implementation.

3- Deformation Effects:

1- Displacement of Supports:

You must enter the effect of forces which resulting from the expected settlement of the supports.

2- Creep and shrinkage:

Is a calculated stress arising from the shrinkage of reinforced concrete on the basis the value of strain equals ٠.٠٠٠٢.

3- Formations axial forces :

Is calculated the effect of the difference in the settlement at the top surface of supporting which is product from the internal axial forces and reflection that on the various members of the bridge design.

4- The effect of changes in temperature:

Design of the members of the structure of the bridge under the influence of conformations resulting from high or low temperatures.

5- The effect of the pre-stress:

Stresses are emerging powers already account for stress before casting and the effect of the remaining forces after the loss account.

Is calculated the Stresses are arise from pre-stress forces before casting and the effect of the remaining forces after calculated the loss.

6- Friction forces:

Is the horizontal forces account for the friction arising from the rooftops as the focal case fulcrum bridge.

Is calculated the horizontal forces for the friction which are arising from the supporting surface based on case of bridge support [3].

4 - Live loads:

Live loads are calculated on the bridge so that they are the largest of the following:

1- Truck with three axes carry load about (600KN) to one lane of the bridge lanes.
2- In case if net width of the bridge between 6.00: 7.30 meters is used in a number 2 lane design.
3- Width is determined by dividing the display design warm Net Bridge on the number of lanes design.

1- When calculating the design live load on lane design It must not be less than the distance between the axis of each truck and the truck neighbors 3.00 meters at the direction of lane width[3] .

3.3. Method of analysis:

3.3.1. Line Beam:

Line beam (using simple beam theory) is satisfactory linear method of analysis where a quick order of magnitude check on a statically determinate bridge deck is required . In this circumstances, the bridge deck is thought of as a beam,as shown inFig.(3.4),Whereas:

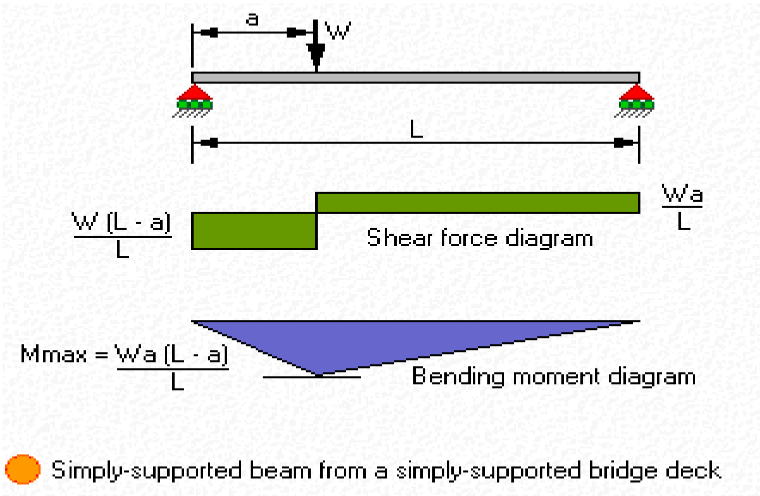


Fig.(3.4): Line beam .

Multiple –span bridges are sometimes built with individual spans simply supported beam(which makes the analysis simpler due to them being statically determinate), in general, multi- span bridges benefit from having a continuous deck [4].

3.3.2. Yield Line:

Yield Lineis a satisfactory method for analysis slab, but the main difficulty is choosing the worst failure mechanism .The Yield Line theory is an ultimate-load theory for slab design and is based on collapse mechanism and plastic properties of under-reinforced slabs. The assumed collapse mechanism is defined by pattern of yield Lines along which thereinforcement has yielded and the location of which depends on the loading and boundary conditions. This is an upper bound method;where as elastic analysis is a lower boundmethod. This requires a separate elasticanalysis for serviceability limit state [4], asshown inFig .(3.5).

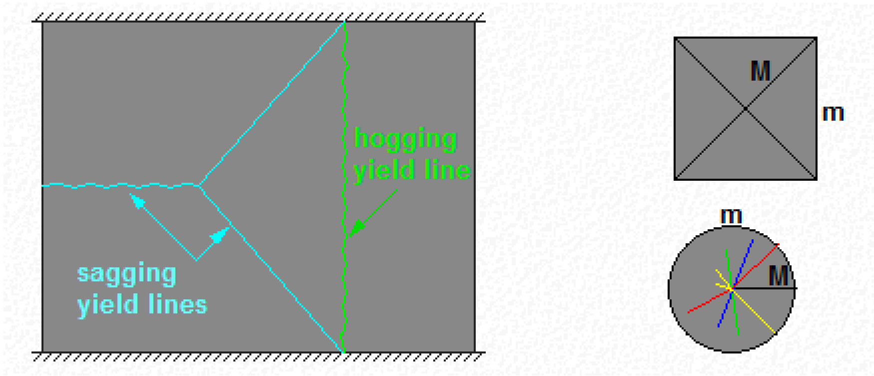


Fig. (3.5) :Yield Line.

3.3.3. DistributionCoefficients:

Chart of ‘distribution Coefficients’ were therefore produced the most widely used for load distribution in orthotropic slabs being produced by morice and little in 1965.These charts which were based on harmonic or fourier analysis , provided a convenient of load distribution which was used in design offices for about fifteen years.

More up-to-date charts are still available today and many enable accurate assessments to be made for longitudinal bending moments in simply supported bridge decks. However , in general ,they are not able to present sufficiently well the wide variety of cross-section configurations and , in particular , the prediction of transverse moment are unsatisfactory for all but the most simple slab decks[4] .

3.3.4. Grillage Analysis:

Grillage analysis is a popular method for analyzing bridge decks because it is easy to use and understand, and can be applied to a wide range of bridge configurations including:

- Beam and slab decks.
- Voided or solid slab.
- Simple and continuous bridges.
- Elastic supports and settlement.
- Right, skew or curved decks [4].

Although computational power has increased many-fold since then, the method is still widely used for bridge deck analysis. Some of the benefits that have been quoted are that grillage analysis is inexpensive and easy to use and comprehend. These benefits traditionally favored the method over finite element analysis which was typically only used for the most complex problems. In today's environment of inexpensive, high-powered computers coupled with elaborate analysis programs and user-friendly graphical interfaces, the finite-element method has begun to replace the grillage method in many instances, even for more straight forward bridge decks.

That said, the grillage method has proved to be a versatile tool for the analysis of many bridges and benefits from numerous favorable comparisons with experiments such as thoseof West (1973).

The plane grillage method involves the modeling of a bridge slab as a skeletal structuremade up of a mesh of beams lying in one plane. Fig. 3.6(a) shows a simple slab bridge deck supported on a number of discrete bearings at each end and Fig. 3.6(b) shows an equivalent grillage mesh. Each grillage member represents a portion of the slab, with the longitudinal grillage mesh. Each grillage member represents a portion of the slab, with the longitudinal beams representing the longitudinal.

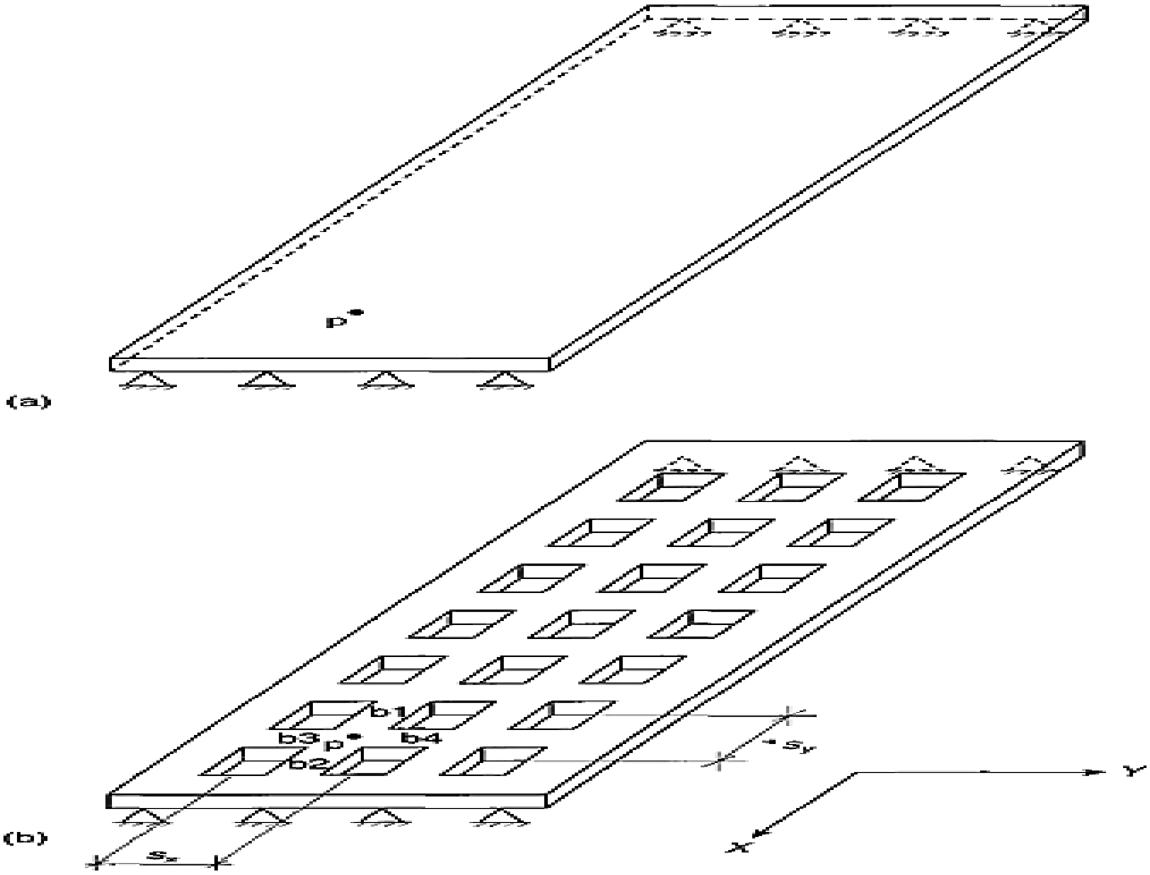


Fig.(3.6): Grillage idealization of a slab: (a) original slab; (b) corresponding grillage mesh.

Stiffness of that part of the slab and the transverse grillage members representing thetransverse stiffness. In this way, the total stiffness of any portion of the slab is represented by two grillage members. The grillage mesh and individual beam properties are chosen with reference to the part of the slab which they represent. The aim is that deflections, moments and shears be identical in both the slab and the grillage model. As the grillage is only an approximation, this will never be achieved exactly. Clearly different levels of accuracy are acceptable for different applications. For example, a crude representation might be sufficient at the preliminary design stages.

Similitude between grillage and bridge slab:

It is necessary to achieve correspondence or similitude between the grillage model and thecorresponding bridge slab. A point p is illustrated in Fig. 3.6 corresponding to the junction of longitudinal beams b1 and b2 and transverse beams b3 andb4. The forces and moments have not been shown on beams b2 and b4 for clarity.

The moments at the ends of beams b1 and b2 adjacent top in the grillage give a measure b4 give a measure of the moment my. The moments in the grillage members are totalmoments while those which are required in the slab are moments per unit breadth. Therefore, it is necessary to divide the grillage member moments by the breadth of slab represented by each.

This breadth is indicated in Fig. (3.6)as sx and sy for the longitudinal and transverse beamsrespectively. Unfortunately, in the grillage, the moments at the ends of beams b1 and b2adjacent to p are generally not equal, nor are those in beams b3 and b4. For a fine grillagemesh, the difference is generally small, and it is sufficiently accurate to take the averagemoment at the ends of the beams meeting at the junction.

The magnitude of this difference isoften used as a check on the accuracy of the grillage, but it should be borne in mind that asmall inequality does not necessarily mean an accurate grillage, as other factors may beinvolved.

The moments per unit breadth in the slab at point p are therefore obtained from the grillageusing the following equations, with reference to Figs. (3.6):

$$m_x = \frac{M_{b1} + M_{b2}}{2s_x}$$

Similarly:

$$m_y = \frac{M_{b3} + M_{b4}}{2s_y}$$

The moments at any other point in the slab can be found in a similar way. If the point is not atthe intersection of longitudinal and transverse grillage members, it is necessary to interpolatebetween adjacent beams. Care should be taken while doing this, especially if a coarse grillagemesh is used. Some computer programs carry out this interpolation automatically, in which case it is necessary to confirm that the program has interpolated the results in a sensible manner. It is often more convenient to start by considering the locations at which moments will be required and to formulate the grillage mesh in such a way as to avoid the need for interpolation between beams.

3.3.5. FiniteElement:

Finite element analysis provides versatile method of analyzing complex structures by sub-dividing the prototype into a large number of small 'elements' which are connected at discrete joints termed' nodes'. For each element, stiffness equations are derived which relate displacements of the nodes to the node forces between elements. This results in a very large number of simultaneous equations which can only be solved sensibly by computer.

Since the principle of sub-dividing a structure into a number of small elements can be applied to any structure however simple or complex ,it is tempting to use FEA for all bridge problems .However, unless the complexity of the structure demands the use FEA, it is probably easier to use grillage analysis which is less demanding computationally , although with the increasing power of modern computers this is not the problem that it used to be .The choice of element type is critical and , if incorrect , the results me be far more inaccurate than those produced by less complex forms of analysis.

Modern finite element programs are usually supplied with pre- and post-processors which allow fast and easy mesh definitions and interpretation of the results[4] .

3.3.6.Non-linear finite element:

NLFEA uses an iterative method,whereby the full load history of structure is modeled by applying incrementsof load upfailure. Failure is deemed to have occurred when the deflection continues to increase at a constant applied load.

At each increment load the structure has to be' solved' for the applied loads checked for cracking. And the output used to modify the structural section and properties for the next increment. Superposition is not possible with NLFEA and as such, each load combination has to be analyzed separately.

Concrete structures fail at a higher load than is apparent from conventional forms of analysis. It is possible to predict this higher failure load more 'accurately' with NLFEA.

Although expensive in engineer and computer time, NLFEA isconsiderably cheaper than physical experiments [4] .

3.3.7Finite Strip:

It is a particular version of the finite element analysis technique. it is a useful but very restrictive method since it can deal only with bridge s which have a constant cross-section throughout the length of the bridge .

The structure is modeled as a number of finite strips which extend from one end of the deck to the other .the Strips are connected by nodes , which also run from end to end . As for finite element, the analytical procedure involves obtaining stiffness equations which must be solved for each harmonic of the load, the results being summed to give the total stress distribution.

Today the finite strip method has virtually disappeared from us due to its limitations. Advances in engineering lead to more complex bridge design and this method was ill-equipped to deal with the required analysis [4].

3.4 LOADING:

3.4.1Definitions:

1. Loads:

External forces applied to the structure and imposed deformations such as those caused by restraint of movement due to changes in temperature

2. Loadeffects:

The stress resultants in the structure arising from its response to loads.

3. Dead load:

The weight of the materials and parts of the structure that are structural elements, but excluding superimposed materials such as road surfacing, rail track ballast, parapets, mains, ducts, miscellaneous ,furniture, etc.

4.Superimposed dead load:

The weight of all materials forming loads on the structure that is not structural elements.

5.Live loads: Loads due to vehicle or pedestrian traffic [5].

- Primary live loads :

Vertical live loads, considered as static loads, due directly to the mass of traffic.

- Secondary live loads :

Live loads due to changes in speed or direction of the vehicle traffic, e.g. lurching, nosing, centrifugal, longitudinal, skidding and collision loads.

6.Traffic lanes:

The lanes that are marked on the running surface of the bridge and are normally used by traffic.

7.Notional lanes:

The notional parts of the carriageway used solely for the purpose of applying the specified live loads [5] .

8.Carriageway:

That part of the running surface which includes all traffic lanes, hard shoulders, hard strips and markerstrips. The carriageway width is the width between raised kerbs. In the absence of raised kerbs it is the width between safety fences, less the amount of set-back required for these fences, being not less than 0.6 m or more than 1.0 m from the traffic face of each fence.

- Carriageway widths of 4.6 m or more :

Notional lanes shall be taken to be not less than 2.3 m or more than 3.8 m wide. The carriageway shall bedivided into the least possible integral number of notional lanes having equal widths,as shown in Table (3.1).

Table (3.1): Number of notional lanes.

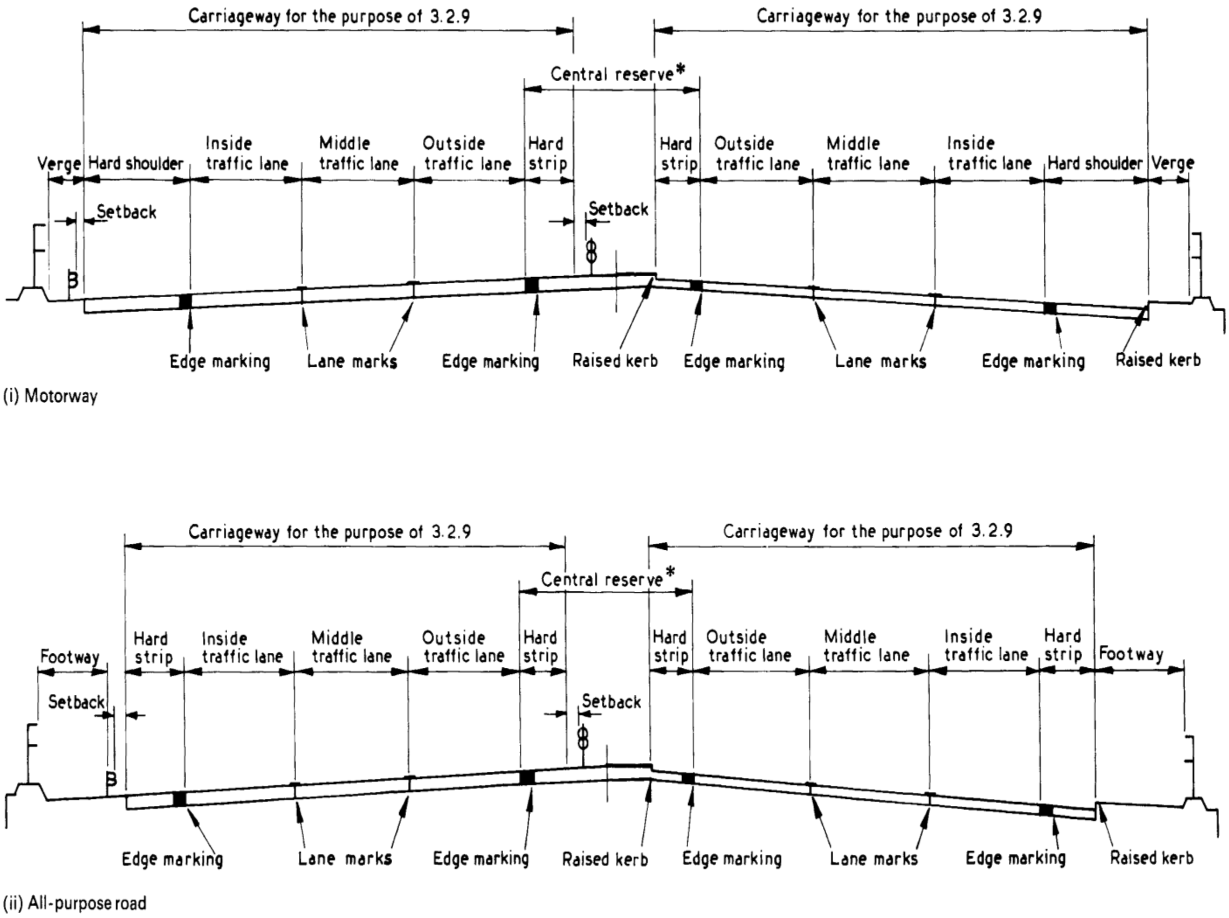
carriageway width(m)	number of notional lanes
4.6 up to and including 7.6	2
above 7.6 up to and including 11.43	3
above 11.4 up to and including 15.24	4
above 15.2 up to and including 19.05	5
above 19.0 up to and including 22.86	6

-Carriageway widths of less than 4.6 m:

The carriageway shall be taken to have a number of notional lanes =

$$\frac{width\ \text{of carriageway (in meters)}}{3} \longrightarrow \qquad (3.1)^{[5]}$$

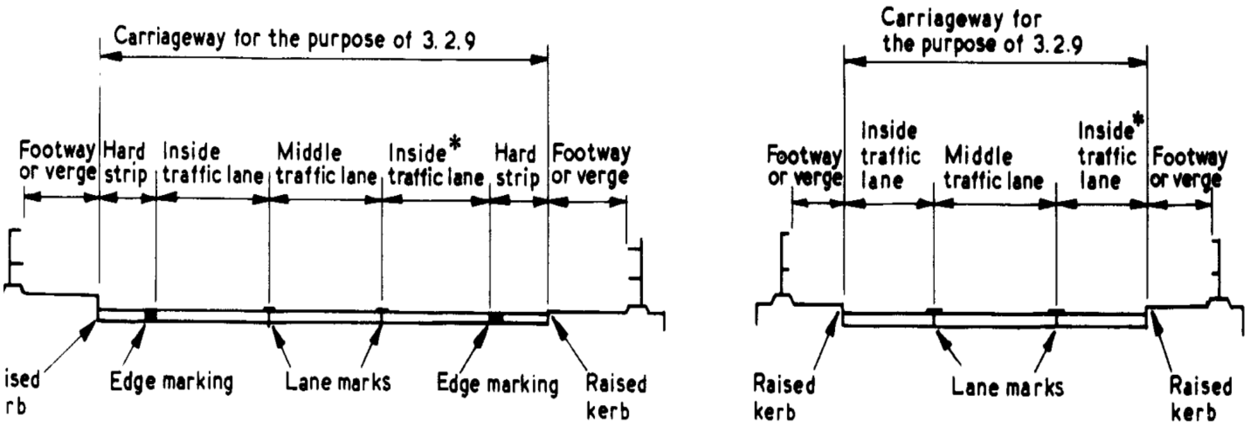
Where the number of lanes is not an integer, the loading on the fractional part of a lane shall be taken prorata the loading for one lane.



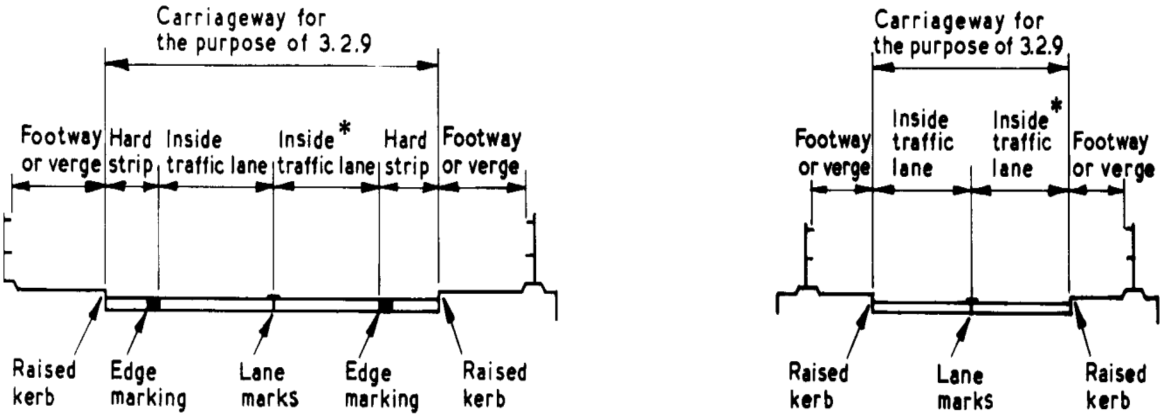
(a) Superstructures : dual carriageway

*Central reserve will be split on separate superstructures

Fig.(3.7): Highway carriageway and traffic lanes.



ı Single 3-lane carriageway



ı Single 2-lane carriageway

) Bridge carrying a single carriageway

Fig. (3.8) :Highway carriageway and traffic lanes (concluded).

3.5. Classification ofloads:

(1) Nominal loads:

Where adequate statistical distributions are available, nominal loads are thoseappropriate to a return period of 120 years. In the absence of such statistical data, nominal load values that are considered to approximate to a 120-year return period are given.

(2) Design loads*:*

Nominal loads shall be multiplied by the appropriate value of ¾fL to derive the designload to be used in the calculation of moments, shears, total loads and other effects for each of the limit statesunder consideration. Values of γ_{fL} are given in each relevant clause and also in (Table 1) in the code.

(3)Additional factor:

Moments, shears, total loads and other effects of the design loads are also to be multiplied by load factor in certain circumstances.

(4) Fatigue loads:

Fatigue loads to be considered for highway and railway bridges, together with the appropriate values are given in (BS 5400).

The loads applied to a structure are regarded as either permanent or transient.

(5)Permanent loads:

For the purposes of this standard, dead loads, superimposed dead loads and loads due to filling material shall be regarded as permanent loads.

(6)Transient loads:

For the purposes of this standard all loads other than permanent ones shall be considered transient. The maximum effects of certain transient loads do not coexist with the maximum effects of certain others. The reduced effects that can coexist are specified in the relevant clauses.

(7)Combinations of loads and γ_{fl} factors:

Three principal and two secondary combinations of loads are specified; values of load factor for each load for each combination in which it is considered are given in the relevant clauses and also summarized in (Table 1) in the (BS 5400) .

- **Combinations:**

(1) Combination 1:

For highway and foot/cycle track bridges, the loads to be considered are the permanent loads, together with the appropriate primary live loads, and, for railway bridges, the permanent loads, together with the appropriate primary and secondary live loads.

(2) Combination 2:

For all bridges, the loads to be considered are the loads in combination 1 together with those due to wind, and, where erection is being considered temporary erection loads.

(3) Combination 3:

For all bridges, the loads to be considered are the loads in combination 1 together with those arising from restraint due to the effects of temperature range and difference, and, where erection is being considered, temporary erection loads.

(4) Combination 4:

Combination 4 does not apply to railway bridges except for vehicle collision loading on bridge supports. For highway bridges, the loads to be considered are the permanent loads and the secondary live loads, together with the appropriate primary live loads associated with them. Secondary live loads shall be considered separately and are not required to be combined. Each shall be taken with its appropriate associated primary live load. For foot/cycle track bridges, the only secondary live load to be considered is the vehicle collision load with bridge supports.

(5) Combination 5:

For all bridges, the loads to be considered are the permanent loads, together with the loads due to friction at bearings [5] .

- Load factors :

A reduced version of table (3.2) is given below in which the γ_{fl}factors for the most commonly occurring loads and combinations asfollow :

Table (3.2)**:** Load factors

load	**limit state**	**$\gamma_{fl}$to be considered in combination** 1	2
Dead load : steel elements	ULS*	1.05	1.05
	SLS	1.00	1.00
Dead load : concrete elements	ULS*	١.١٥	1.15
	SLS	١.٠٠	1.00
Superimposed dead load : deck surfacing	ULS**	١.٧٥	1.75
	SLS	١.٢٠	
Superimposed dead load : other loads	ULS**	١.٢٠	1.20
	SLS	١.٠٠	1.00

Wind :during erection	ULS SLS		1.10 1.00
Wind : with dead load plus Superimposed dead load only and for members primarily resisting wind load	ULS SLS		1.40 1.00
Wind : with dead load plus Superimposed dead plus other appropriate combination 2 loads	ULS SLS		1.10 1.00
Wind : where effects are relieving	ULS SLS		1.00 1.00
Erection : temporary loads	ULS SLS		1.15 1.00
Highway bridge live loads : HA alone	ULS SLS	١.٥٠ ١.٢٠	1.25 1.00
Highway bridge live loads : HA alone with HB alone	ULS SLS	١.٣٠ ١.١٠	1.10 1.00
Highway bridge live loads : footway and cycle track loading	ULS SLS	١.٥٠ ١.٠٠	1.25 1.00
Highway bridge live loads : type RU, RL and secondary	ULS SLS	١.٤٠ ١.١٠	1.20 1.00

3.6. Loads applicable to all bridges:

3.6 .1.Dead load:

(1) Nominal dead load:

The nominal dead load initially assumed shall be accurately checked with the actual weights to be used in construction and, where necessary, adjustments shall be made to reconcile any discrepancies.

(2) Design load:

The load factor, to be applied to all parts of the dead load, irrespective of whether these parts have an adverse or relieving effect, shall be taken for all five toad combinations as follows:

	For the ultimate Limit state	for the serviceability limit state
Steel	1.05	1.0
Concrete	1.15	1.0`

3.6 .2.Superimposed dead load:

(1) Nominal superimposed dead load:

The nominal superimposed dead load initially assumed shall in all cases be accurately checked with the actual weights to be used in construction and, where necessary, adjustments shall be made to reconcile any discrepancies. Where the superimposed dead load comprises filling, e.g. on spandrel filled arches, consideration shall be given to the fill becoming saturated.

(2) Design load:

The load factor, to be applied to all parts of the superimposed dead load irrespective of whether these parts have an adverse or relieving effect, shall be taken for all five load combinations as follows:

For the ultimate limit state limit state	For the serviceability limit state
1.75	1.20

(3)Wind loads:

The wind pressure on a bridge depends on the geographical location, the local topography, the height of the bridge above ground, and the horizontal dimensions and cross section of the bridge or element under consideration. The maximum pressures are due to gusts that cause local and transient fluctuations about the mean wind pressure.

(4) Liveloads:

Standard highway loading consists of HA and HB loading.

HA loading is a formula loading representing normal traffic in Great Britain. HB loading is an abnormal vehicle unit loading. Both loadings include impact.

- HA loading:

Type HA loading consists of a uniformly distributed load and a knife edge load combined, or of a single wheel load.

(1) Nominal uniformly distributed load (UDL:

The UDL shall be taken as 30 KN per linear meter of notional lane for loaded lengths up to 30 m, and for loaded lengths in excess of 30 m it shall be derived of notional lane for loaded lengths up to 30 m, and for loaded lengths in excess of 30 m it shall be derived from the equation [5] .

(2) Nominal knife edge load (KEL) :

The KEL per notional lane shall be taken as 120 KN [5] .

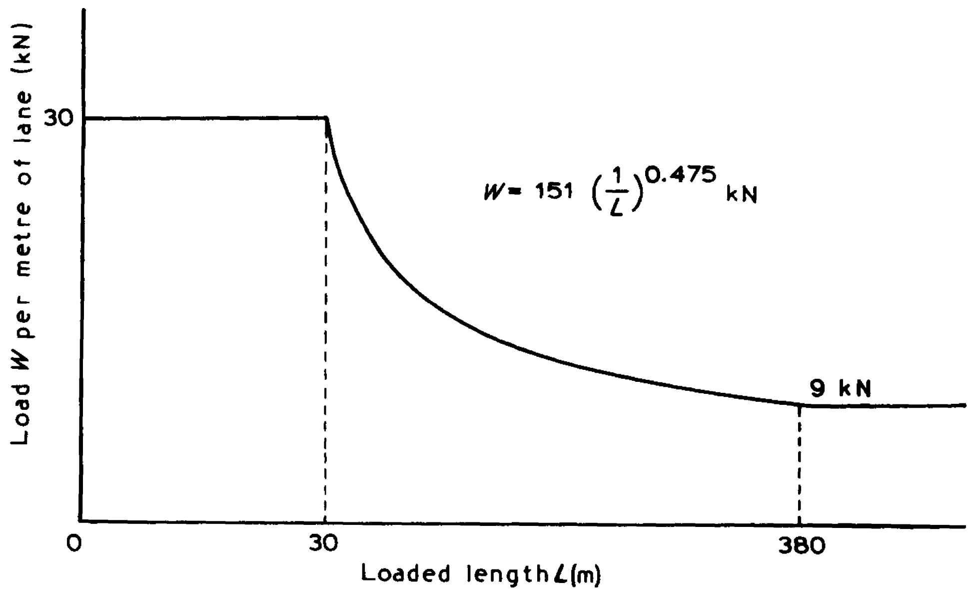


Fig.(3.9) : Loading curve for HA, uniformly distributed load(UDL)

(3) Single wheel load:

This is an alternative form of HA loading to the UDL+KEL loading, a value of 100 KN is always taken for single wheel load. Single wheel load can be placed anywhere on the carriageway, and occupies either a circular area of 340 mm diameter or a square area of 300mm side [4] .

This is contact pressure area which is dispersed through the road thickness as shown in the diagram below.

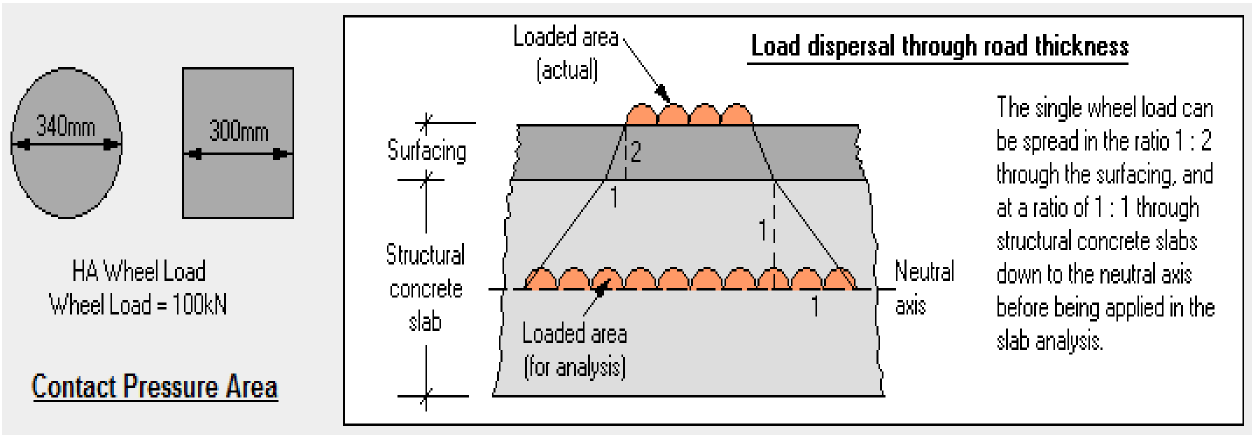


Fig. (3.10): dispersion area of wheel.

- HB loading:

Type HB loading For all public highway bridges in Great Britain, the minimum number of units of type HB loading that shall normally be considered is 25, but this number may be increased up to 45 if so directed by the appropriate authority[4] , as shown in Fig .(3.11).

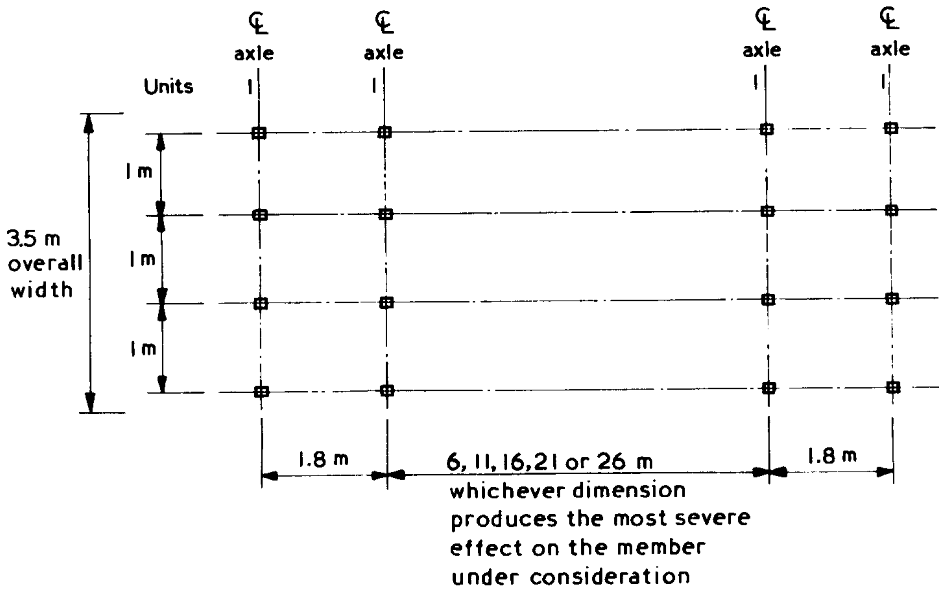


Fig. (3.11): Dimensions of HB vehicle.

3.6.3. Application of HA and HB loading:

3.6.3.1. General:

HA loading is used to represent normal traffic in GreatBritain.

HB loading is used to represent avehicle carrying abnormally heavy loads.

Both HA and HB loading include impact.

The structure and its elements must be designed to resist the more severe effects of either:

-Designed HA loading alone or

-Designed HA loading combined with designed HB loading.

Positioning of load combinations to give the most severe effects can be difficult. These positions can be found by trial and error but it may be better to use a computer package which is capable of analyzing a number of options and reporting the worst effects.

The loaded length is calculated separately for each notional lane.

This means that the loaded length, and hence the UDL intensity, can be different for different lanes. A simple case where this arises is on a curved bridge where the inner lanes have shorter loaded length than the outer lanes [4] .

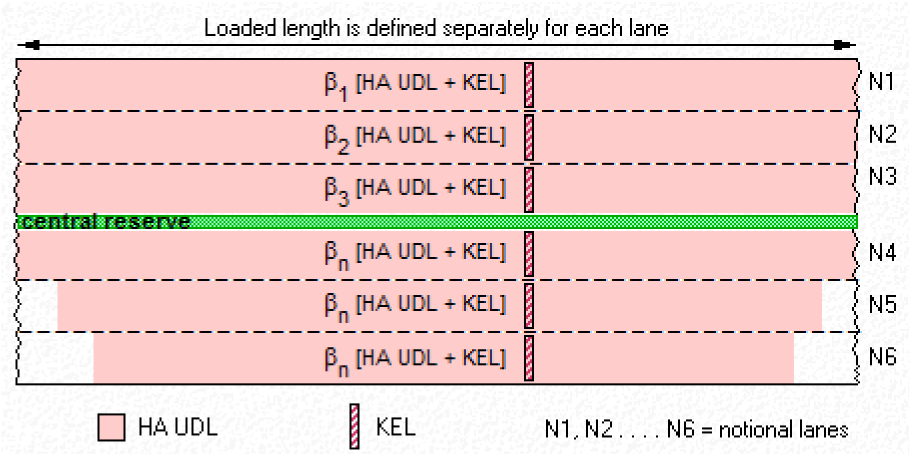


Fig. (3.12): loaded length for each lane.

Lane factors β are applied to lanes as indicated in the table opposite. Lane factors basically allow for the low probability of all lanes being fully loaded at the same time , but can also allow for lateral bunching on short loaded lengths[4], as shown in Table (3.3).

Table (3.3): HA lane factors

Loaded Length (m)	First Lane Factor β_1	Second Lane Factor β_2	Third Lane Factor β_3	Fourth & subsequent Lane Factor βn
$0 < L \leq 20$	α_1	α_1	0.6	0.6 α_1
$20 < L \leq 40$	α_2	α_2	0.6	0.6 α_2
$40 < L \leq 50$	1.0	1.0	0.6	0.6
$50 < L \leq 112$ $N < 6$	1.0	$7.1/\sqrt{L}$	0.6	0.6
$50 < L \leq 112$ $N \geq 6$	1.0	1.0	0.6	0.6

$L < 112$ $N < 6$	1.0	0.67	0.6	0.6
$L > 112$ $N \geq 6$	1.0	1.0	0.6	0.6

NOTE: $\alpha_1 = 0.274\ b_L$ but not more than 1.0

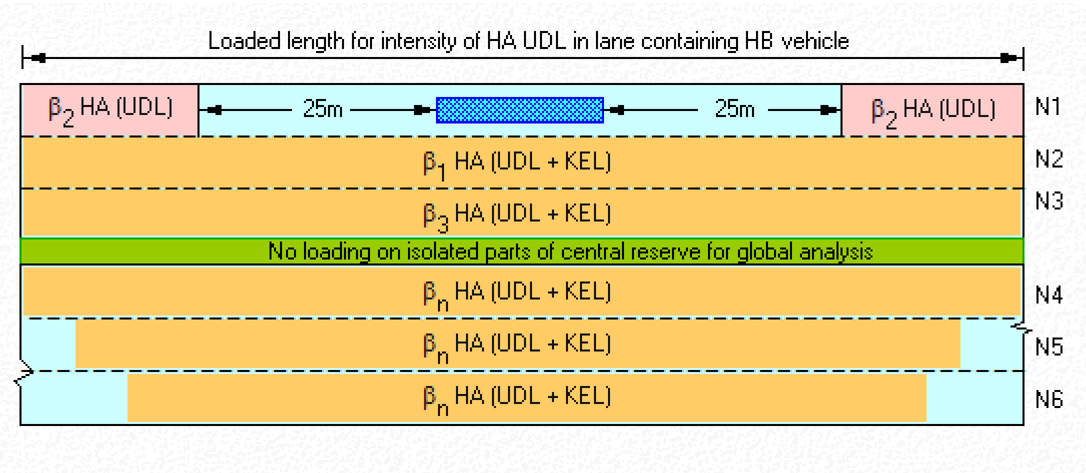
$\alpha_2 = 0.0137\ [b_L\ (40\text{-}L) + 3.65(L\text{-}20)]$.

Where b_L is the notional lane width in meters.

3.6.3.2. Types HA and HB loading combined:

(1)Case 1:

Types HA and HB loading combined case 1- HB vehicle wholly with in one lane,as shown in Fig.(3.13).



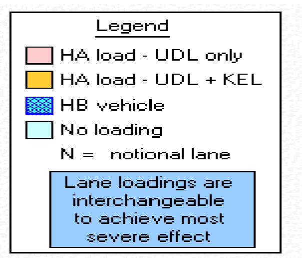


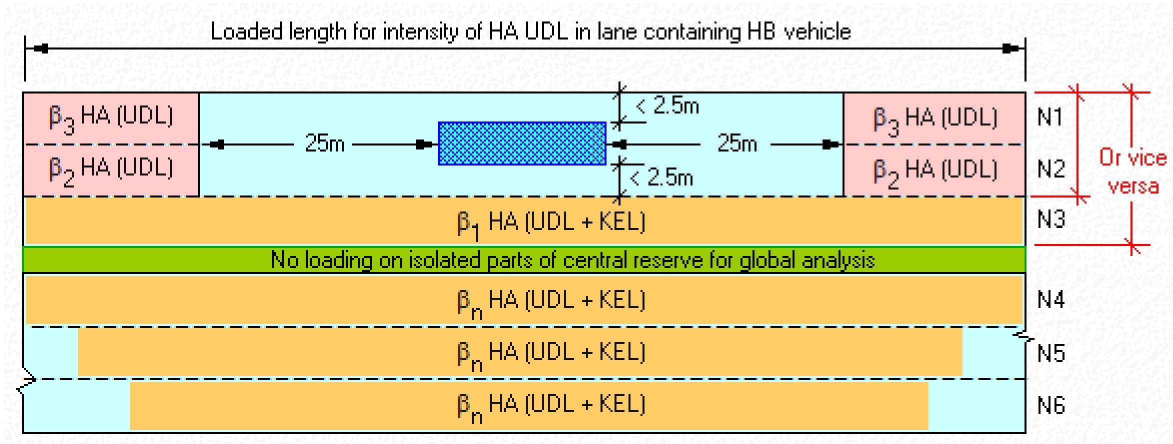
Fig.(3.13) :Types (HA + HB)loading case 1.

The loaded length is calculated for each notional lane.

The HB vehicle replaces one lane of HA for a distance extending form 25 m in front of the vehicle to 25 m behind. If there is any lane length left to be loaded with HA the KEL is not applied. The remaining lanes are loaded with HA (i.e including the KEL) in the normal way [4] ,See above Fig.(3.13).

(1) Case 2a:

Types HA and HB loading combined case 2a – HB vehicle straddling two notional lanes ,asshown in Fig.(3.14).



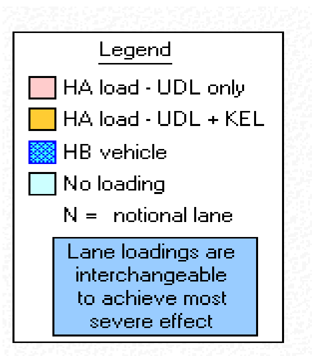


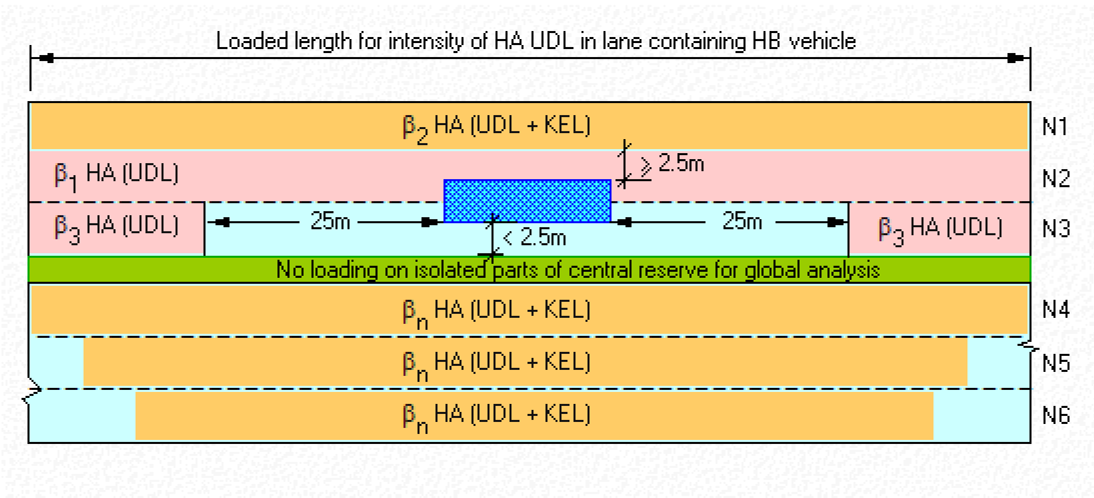
Fig. (3.14): Types (HA + HB) loading case 2a.

Where the HB vehicle lies partially with in a notional lane and the remaining width of that lane (measured from the side of the HB vehicle to the edge of that lane) is less than 2.5 m, the HB vehicle replaces the HA loadings in the straddled lanes for a distance extending from 25 m in front of the vehicle to 25 m behind.

If there is any lane length left to be loaded with HA, KEL is not applied. The remaining lanes are loaded with HA (i.e including the KEL) in the normal way [4].

(2) Case 2b:

Types HA and HB loading combined case 2b – HB vehicle straddling two notional lanes, as shown in Fig .(3.15).



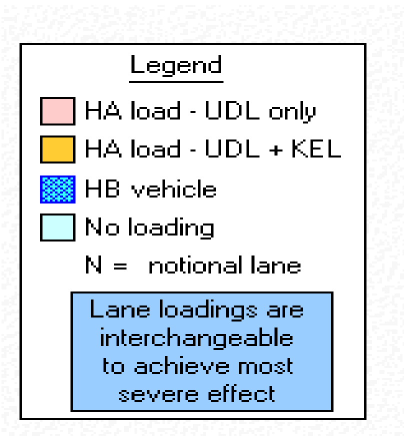


Fig. (3.15) :Types (HA + HB)loading case 2b.

Where the HB vehicle lies partially with in a notional lane and the remaining width of that lane (measured from the side of the HB vehicle to the far edge of that lane) is greater than or equal to 2.5m, the HA UDL loading in that lane remains, but is multiplied by an appropriate lane factor for anotional lane of width 2.5 m irrespective of the actual lane width.

The HA KEL for that lane is omitted, the remaining lanes are loaded with HA (i.e including the KEL) in the normal way [4] .

3.6.4. Wind loading:

Wind is rarely critical in slab concrete bridges with:

- Span less than 20m.
- Width of 10m or more.
- Normal heights above the ground.

A suitable check for wind effects in normal circumstances is to consider a wind pressure of 6 KN/m^2 applied to the vertical projected area of the bridge neglecting those areas where wind effects the load would be beneficial .

For bridges where wind effects are crucial, the nominal longitudinal transverse and vertical wind loads must be considered in combination with the other loads in combination 2.

The Helgeland Bridge on the west coast of Norway illustrated in Fig. (3.16) was a case where wind effects were crucial. The location of the bridge meant that it would be exposed to severe wind climates and, to aid the design of the bridge a two year investigation was carried out to study wind conditions.

This study carried out over two years prior to construction, was an essential element in selecting the correct bridge design and form of construction [4] .

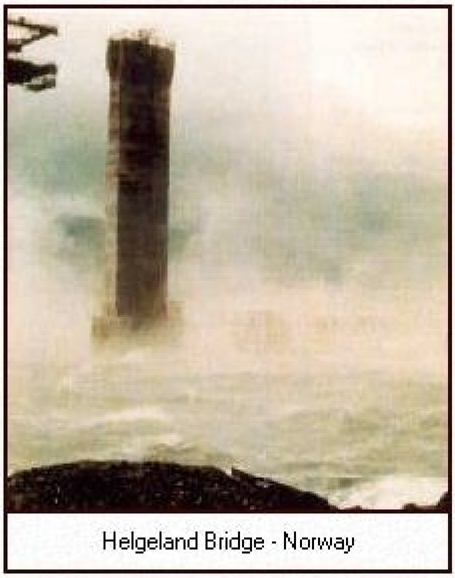


Fig. (3.16): The Helgeland Bridge.

3.6.5. Temperature effects:

When determining temperature loading on bridge, two aspects should be considered:

The restraint to the overall bridge movement due to the temperature range.

The effects of temperature differences through the depth of the bridge deck.

Effective bridge temperatures are used for:

Designing the bearings and movement joints.

Determining stress resultants in the bridge if movement is restrained.

-To calculate temperature range:

Find the maximum and minimum shade air temperatures for the location of the bridge.

The shade air temperatures are for a return period of 120 years, 50 years for a foot or cycle track bridge.

The maximum and minimum effective bridge temperatures are derived from the relevant design code tables which relate shade air temperature to effective bridge temperature.

The effective bridge temperatures are dependent upon the depth of surfacing where this depth differs from that given in the relevant design code tables, a correction to depth must be made [4] .