Chapter Four

Superstructure Analysis and Design

4.1. Introduction:

Applicationloads onthe structural are produced forcesanddistortion, the appointment ofall these forcesanddeformationscalledstructuralanalysis.

While the structural designincludes organizationand the relativedistribution of the structure of thevariousparts so as to ensurethe mainstayloadssubjected to it [4] .

- **The objective of the structural design:**

  -To getthe structural consistency ofthesafety andlow cost.

  -The achievement of structural requirements, taking into account the aestheticfactor.

  - The sustainabilityof structural of anyextensionin the future.

  - Selection ofstructural elementsable to bear the effortsresultingfromthose loads [4] .

4.2. Description of the Bridge:-

The Bridgeis locatedin Sudan at Northern Kordofan State, Elsimeyh road project. The bridge is consist of one simply supported span , the total length of 16.6m , a total width of 8.5m, carriage way of 7 m width and internal sidewalks of 0.4 m for both sides, crash barriers wide of 0.35m, loading capacity of(HA)+ 30units (HB),and concrete deck.

- **Bridge data:**

Deck span	1٦.6m
Total Deck width	8.5 m
Width of carriageway	7m
Thickness of deck cast In-situ slab	220 mm
Surfacing and overlay thickness	50 mm
Number of girders	4
Girder width	500mm
Girder depth	1200mm
spacing of girder c/c	٢.٢m
Width of walkway	0.75m
side walk width	400mm
Crash barrier width	350mm

- **Materials:**

Unit weight of concrete	24 kN/m^3
Unit weight of Asphalt	22kN/m^3
Strength of concrete (f_{cu})	30N/mm^2
Reinforcing bars strength Steel, (f_y)	410N/mm^2

- **Design Considerations**

(1) The design of this bridge is based on the Bridge Standards BS 5400 (Part 2(1978) modified by BD 37/01).

(2) The bridge is designed for 30 units of HB abnormal vehicle and the associated HA loading. Two notional lanes are adopted as required by the codes for the carriageway width of 7 m.

4.3. Manual Analysis:

4.3.1. Loadingcalculation:

1- Permanent load :

Slab load = 0.22*24 = 5.28 KN/m2

Girder = $24(0.5*0.15*2+ 0.075^2+0.15^2+ 0.28*0.9) = 10.32$ KN/m

2- Super imposed dead load :

Surfacing = 0.05*22 = 1.1 KN/m^2

Railings = 1.0 KN/m

3- Live load :

HA loading includes HA (UDL) and nominal HA (KEL).

HB Loading: 30 units HB load = 30* 10 = 300KN per axle load.

= 300/4 = 75 KN per wheel

Vehicular HA & HB live loads

Number of notional lanes from table (3.1) = 2

Width of notional lane 7/2 = 3.5 m

HA per notional lane (for loaded length = 16.6m)

(BD 37/01) Clause 6.2.1[9]

$W = 336(1/L)^{0.67} = 336(1/16.6)^{0.67} = 51.15$ KN/m

51.15/3.5 =14.61 KN/m per notional lane

HA (KEL) per notional lane = 120 KN

(BD 37/01) Clause 6.2.2[9]

= 120 /3.5 = 34.29 KN per notional lane

HA& HB LoadingApplications:

Lane 1 factor, $\beta_1 = 0.959$

Lane 2 factor, $\beta_2 = 0.959$

(BD 37/01) Table (3.3) [9]

From table (3.3) HA lane factor

For L < 20 $\beta_1 = \beta_2 = \alpha_1$

$\alpha_1 = 0.274 b_L = 0.274 * 3.5 = 0.959 < 1$

load on lane (i) = β_i (UDL + KEL)

For(lane1& lane 2):

HA (UDL) * β_1= 14.61 * 0.959 = 14.01KN/m^2

HA (UDL) * β_2= 14.61 * 0.959 = 14.01KN/m^2

HA (KEL) *β_1 = 34.29 * 0.959 = 32.88 KN /m

HA (KEL) *β_2 = 34.29 * 0.959 = 32.88 KN /m

4.3.2.Analysisof slab:

(1)Loading:

- **Dead load :**

Self-weight of slab = .22×24 = 5.28 KN/m^2

Super imposed dead load = .05× 22 = 1.1 KN/m^2

U_D = 1.15×DL + 1.75× SIDL = 1.15×5.28 + 1.75×1.1 = 7.9 KN/m^2

$$\pm M_{UD} = \frac{U_D L^2}{10} \qquad (4.1)$$ [11]

$$= \frac{7.9 \times 2.2^2}{10} = 3.8 \text{KN.m/m}$$

- **Live load :**

The most severe effect of live load on the deck is a single wheel of (100 KN) which equivalent of HA load. The contact area of this load at pavement surface is (300×300) mm square area. With dispersion of 1:2 with in the pavement, the contact area at the concrete slab surface is (350×350) mm using westergaard theory for plate bending the maximum moment is:

$$\pm M_L = \frac{0.8\,(S+0.6\,)P}{9.8}$$

$$= \frac{0.8\,(2.2+0.6\,)100}{9.8} = 22.8 \text{ KN.m/m}$$

$\pm M_{UL}$ = 1.2 ×1.5 ×22.8 =41KN.m/m

Total moment is:

M_U =± (1.2×3.8+41) = 45.5 KN.m/m

4.3.3.Design of slab:

For 220mm thickness of slab;

$d = h - c - \varnothing/2$

Assume $\varnothing = 16mm$ ⟶ $d = 220 - 25 - 16/2 = 187mm$

Assuming the depth of compression zone is (x) then ;

$0.4\ f_{cu}\ b\ x\ (d - x/2) = MU$ ⟶ (4.2)[10]

$0.4 * 30 *1000\ x(187- x/2) = 45.5 *10^6$

$X = 21.5$ mm

C = T   (compression = tension)

$0.87 * f_{yv} *A_S = 0.4 * f_{cu} *b* x$

$$A_S = \frac{0.4\times30\times1000\times21.5}{0.87\times410} = 723.3mm^2$$

Using $\varnothing = 16mm$ ⟶ $A_S = 201\ mm^2$

Use 4T16 ($A_S = 804\ mm^2$)Top & Bottom

- **Distribution reinforcement :**

$$A_s\ min = \frac{0.13\ bh}{100}$$ ⟶ (4.3) [11]

$$= \frac{0.13\times\ 1000\times\ 220}{100} = 286\ mm^2$$

Use T12mm ⟶ $A_s = 113\ mm^2$

Use 3T12 (A_s = 339 mm^2)

- **The ultimate shear force due to equivalent wheel load is :**

$V_u = \gamma_{f3} \times \gamma_{fL} \times P$ ⟶ (4.3) [10]

$= \gamma_{f3} \times \gamma_{fL} \times 100$

= 1.2 × 1.5 × 100 = 180 KN

$v_U = \frac{V_u}{b \times d} = \frac{180 \times 10^3}{1000 \times 187} = 0.96\ N/mm^2$

- **Critical Shear stress ;v_c:**

$v_c = \frac{0.27}{\gamma_m} \left(\frac{100\ As_p}{b_w\ d}\right)^{1/3} (f_{cu})^{1/3}$ ⟶ (4.4)[10]

γ_m = 1.25 A_{SP} = 804mm^2

b_w= 1000mm

v_c= 2.8N/mm^2

$v_c \xi_s$ ⟶

$\xi_s = \left(\frac{500}{d}\right)^{1/4}$ = 1.3 >0.75 (take ξ_s = 0.75)

$v_c \xi_s$ = 0.75×2.8 = 2.1

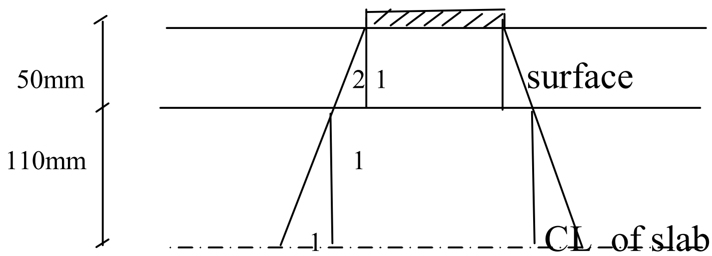
Thus; $v_u < v_c \xi_s$

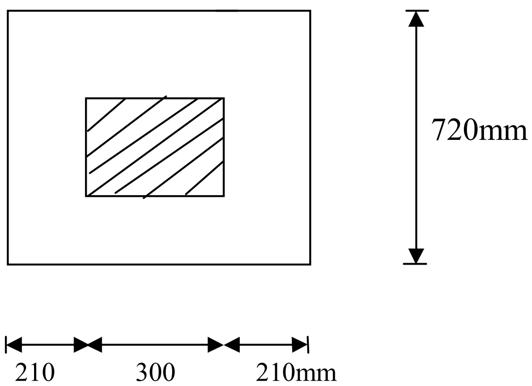
$\nu_c + 0.4 = 3.2\ N/mm^2$

Shear ok

•Check punching shear :

Ultimate shear force ; $V_u = 180$ KN





The loaded length area $= (210 \times 2 + 300)^2 = 518.4 \times 10^3 mm^2$

The perimeter $= 2(210 \times 2 + 300) = 1440mm$

$$\nu = \frac{V}{b_0 d} = \frac{180 \times 1000}{1440 \times 187} = 0.67 N/mm^2$$

$$\nu < 0.75\sqrt{f_{cu}} = 4.44\ N/\ mm^2$$

Therefore ; no further check is required.

4.4. Analysis by sap 2000:

4.4.1. Methodology:

All the components of bridge were modeled according to submitted details of concrete sections and all dimensions were checked for consistency and were entered in the model.

Material property was determined and defined to the model elements and member's .Sap2000 software and manual calculation was used.

In this study two positions HB vehicle have been experienced to obtain the worst case for the bending moment and shear forces [seeAppendix (A)].

4.7.2. Load pattern values :

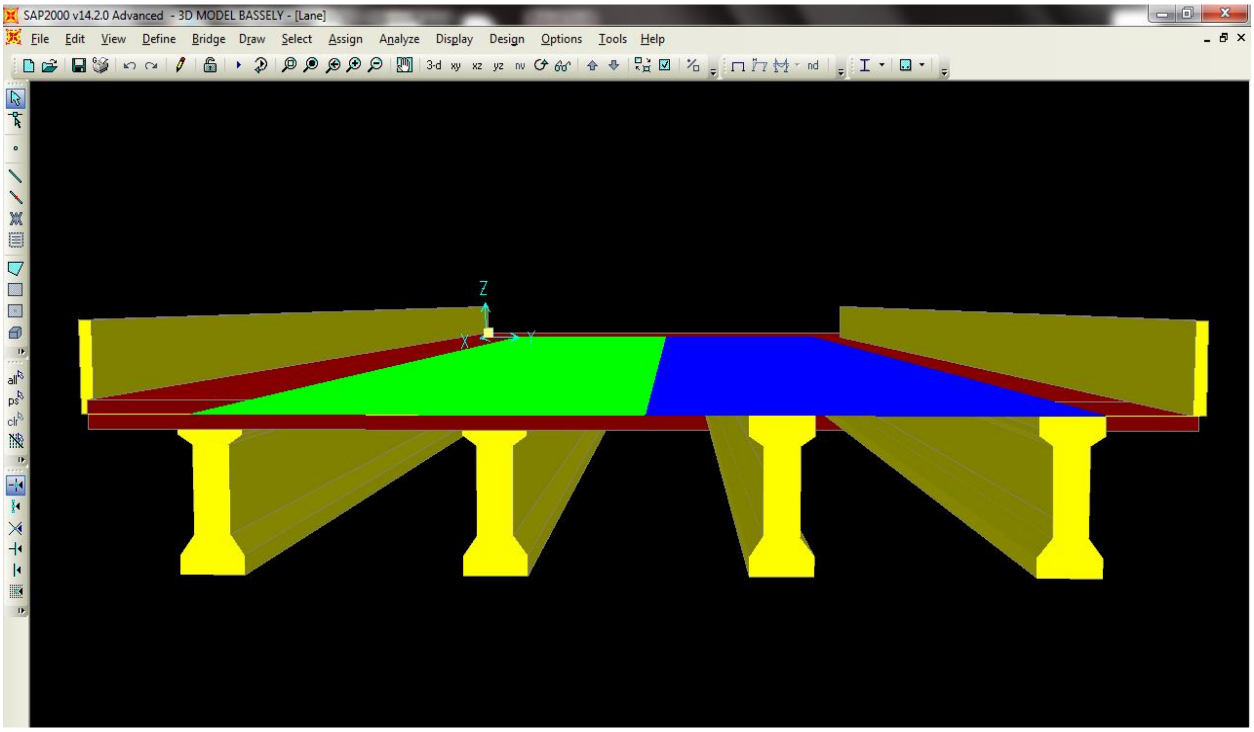


Fig. (4.1) : Bridge superstructure 3D.

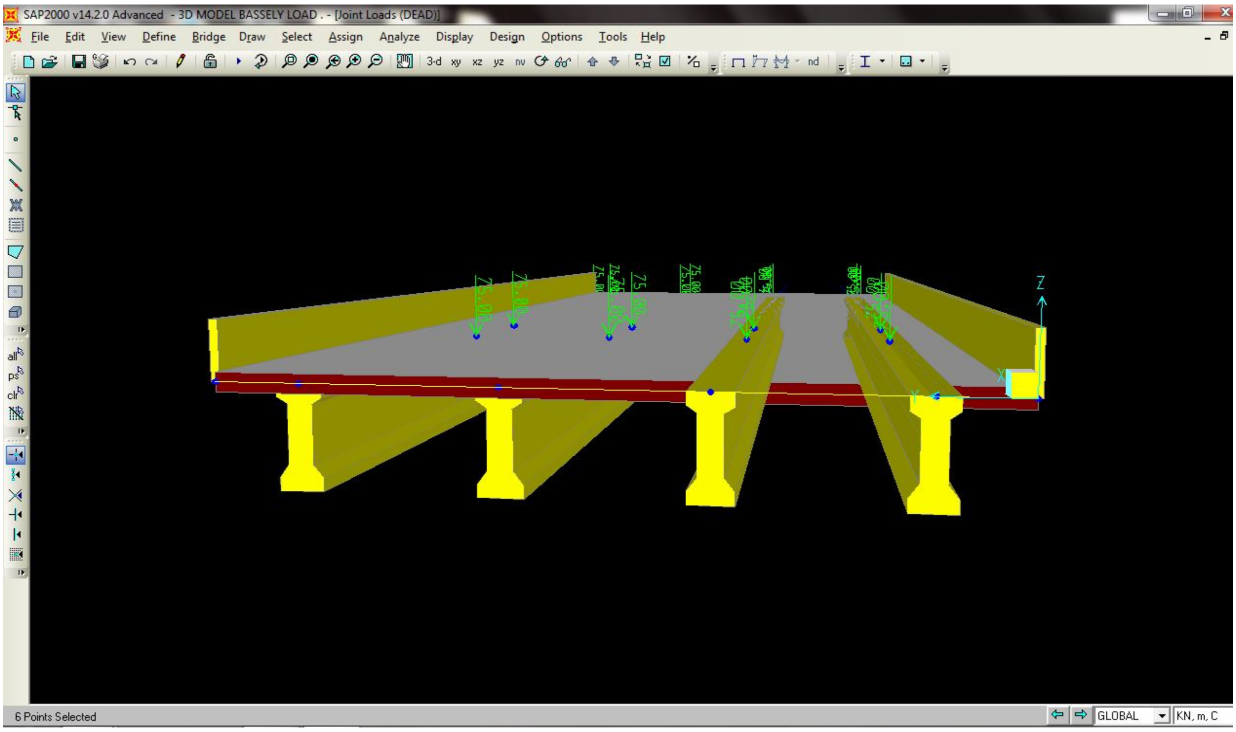


Fig. (4.2) :HB vehicle on superstructure of Bridge.

1- Walk way loading :

i-Crash barrier load = 3 KN/m

ii - Side walk load = 2.4 KN/m.

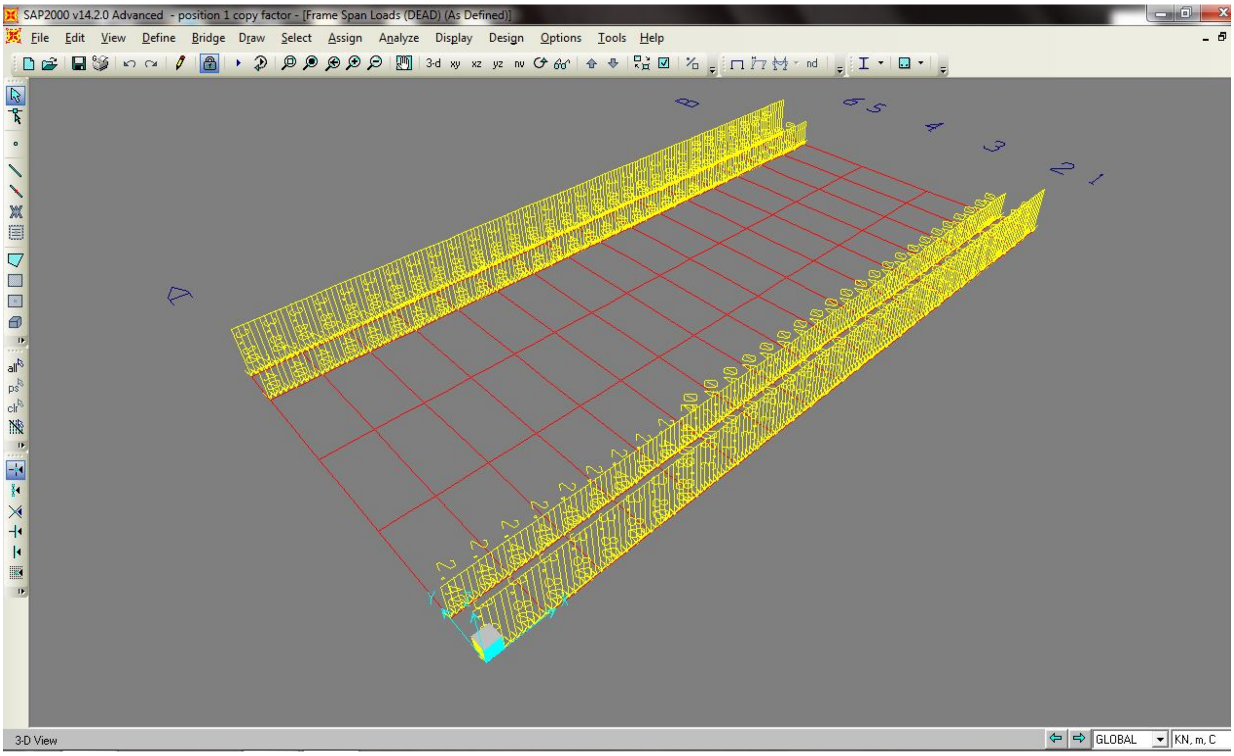


Fig. (4.3) : Walk way load.

2- Surfacing load :

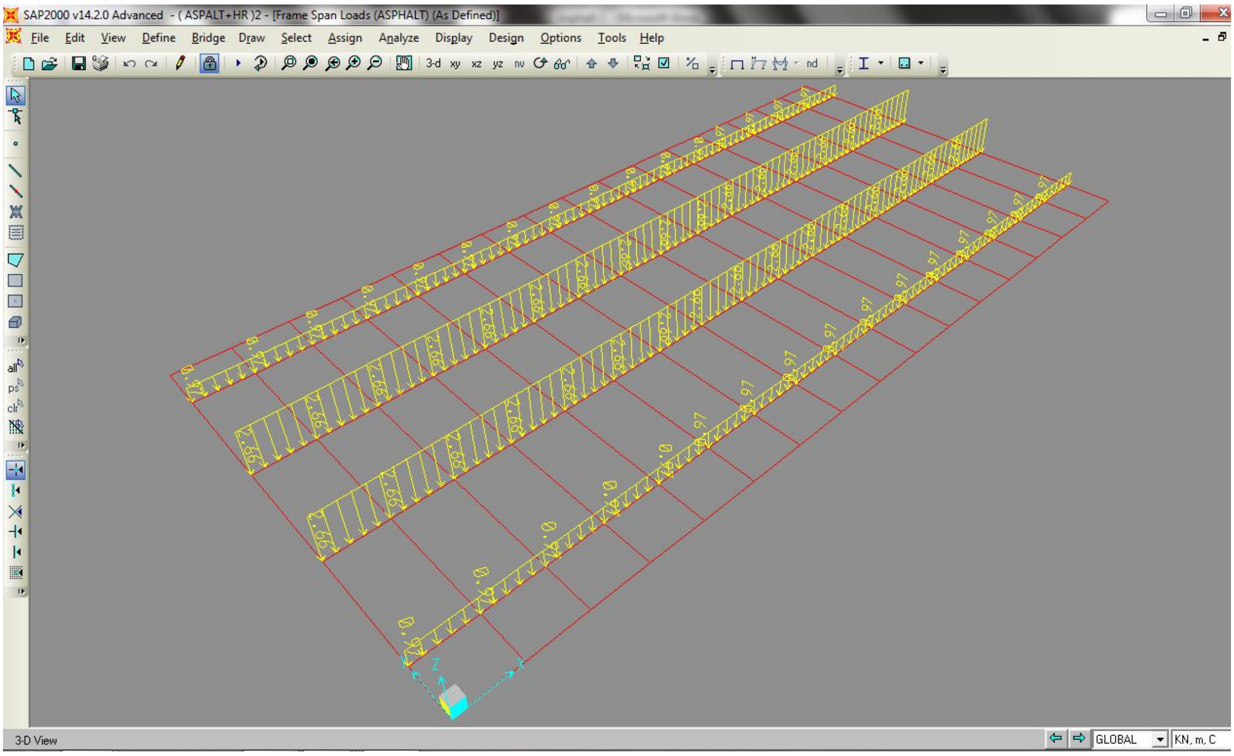


Fig. (4.4) : Surfacing load.

3- Railing loading :

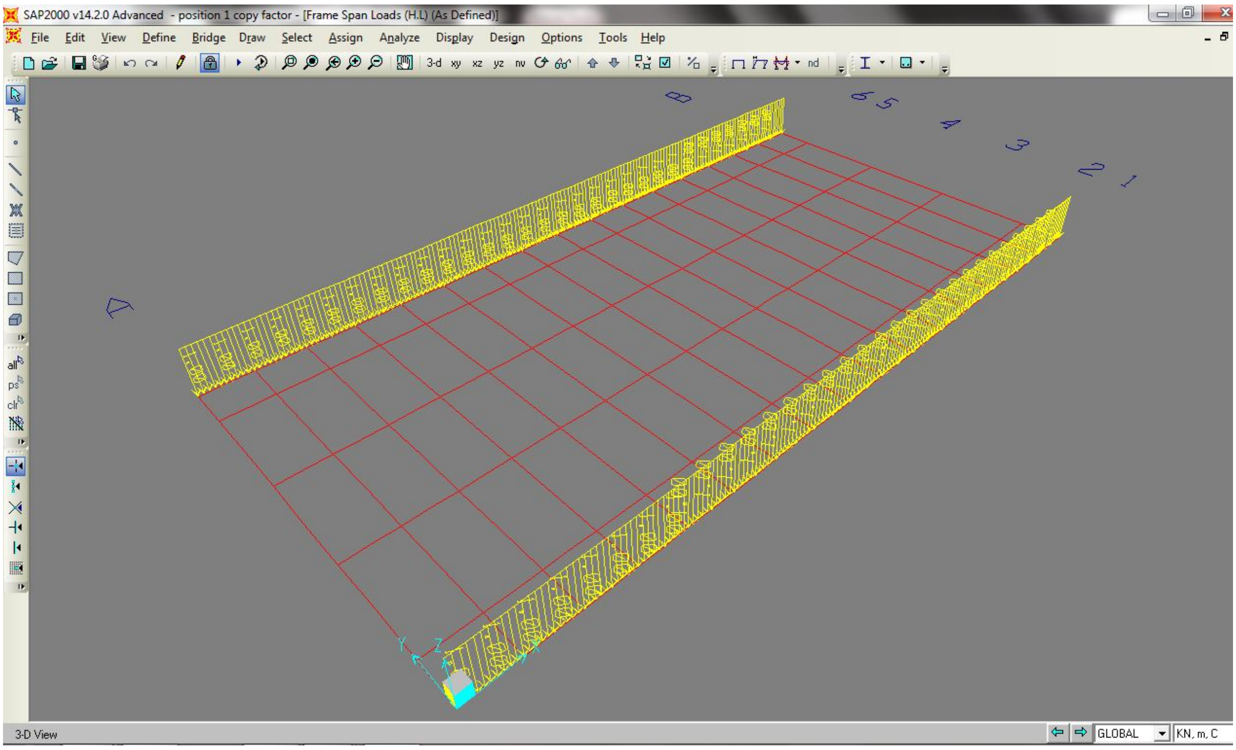


Fig. (4.5) : Railing loading.

4- HA Alone(UDL+KEL)moment :

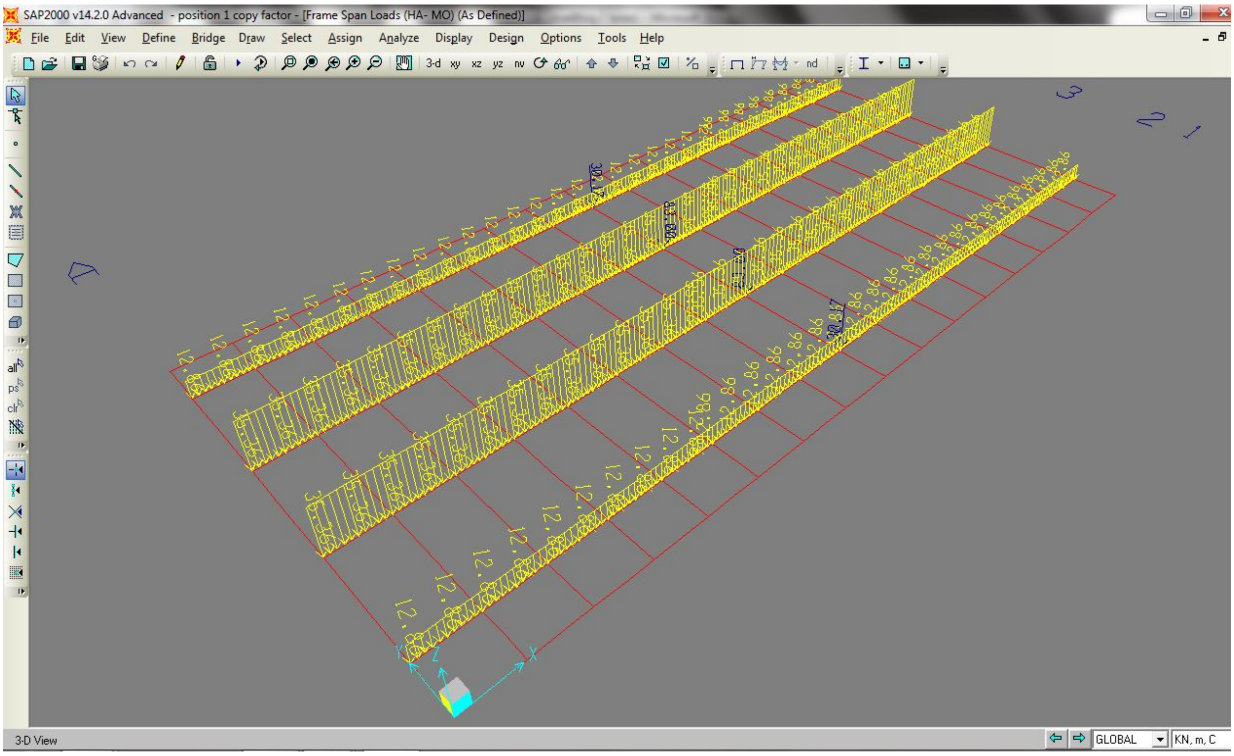


Fig.(4.6) : HA Alone(UDL+KEL) moment .

5- HA Alone(UDL+KEL) shear :

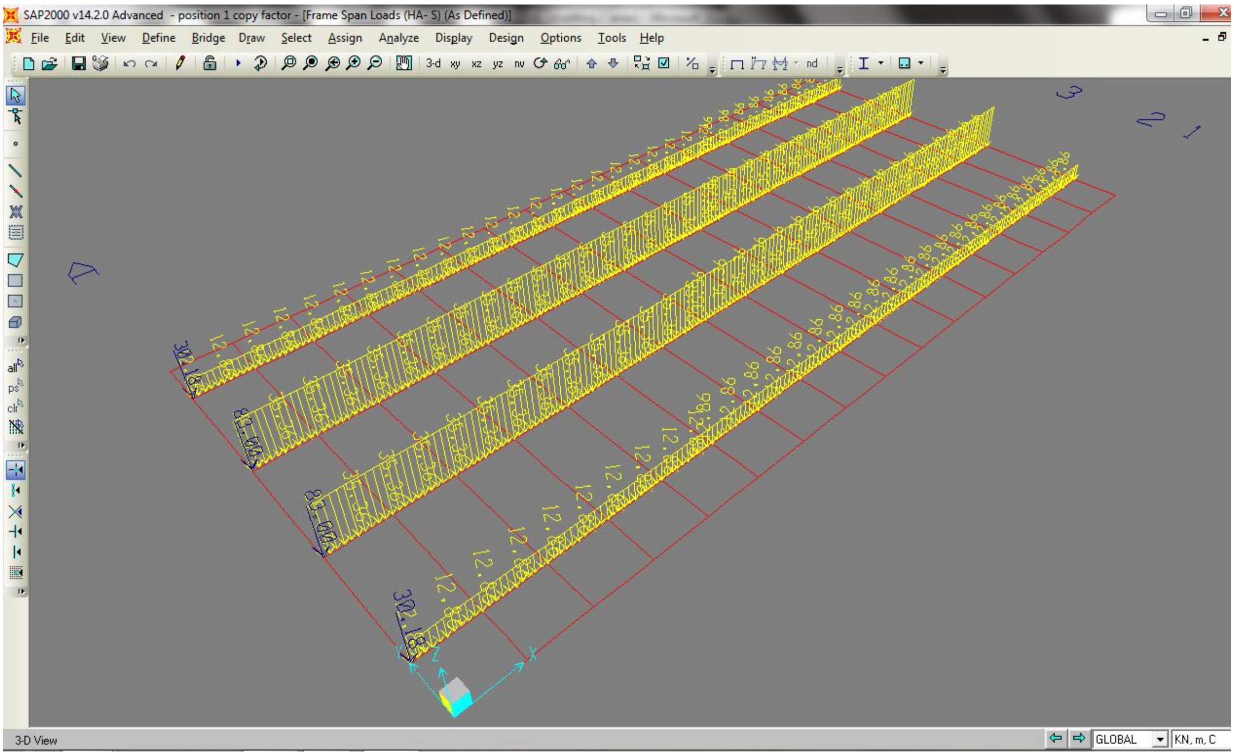


Fig. (4.7):HA Alone(UDL+KEL) shear.

(HB+HA) Loading- moment :

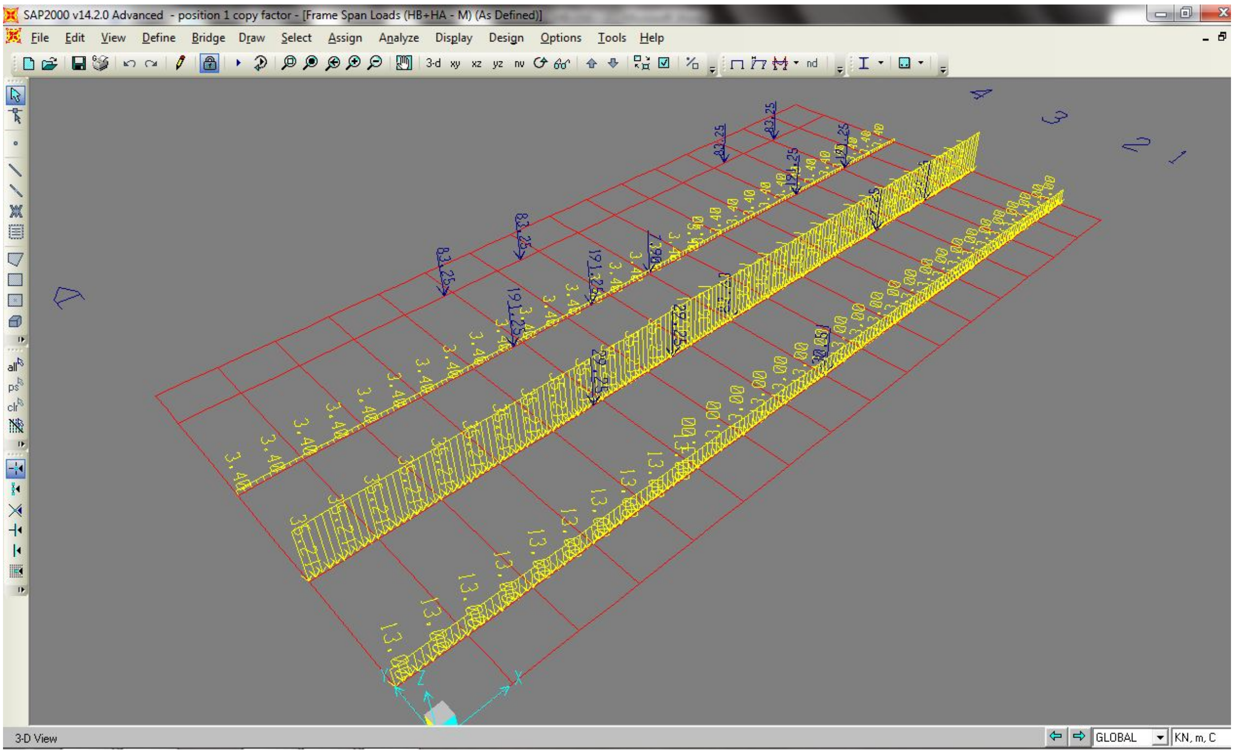


Fig. (4.8): (HB+HA) Loading- moment.

(HB+HA) Loading- shear:

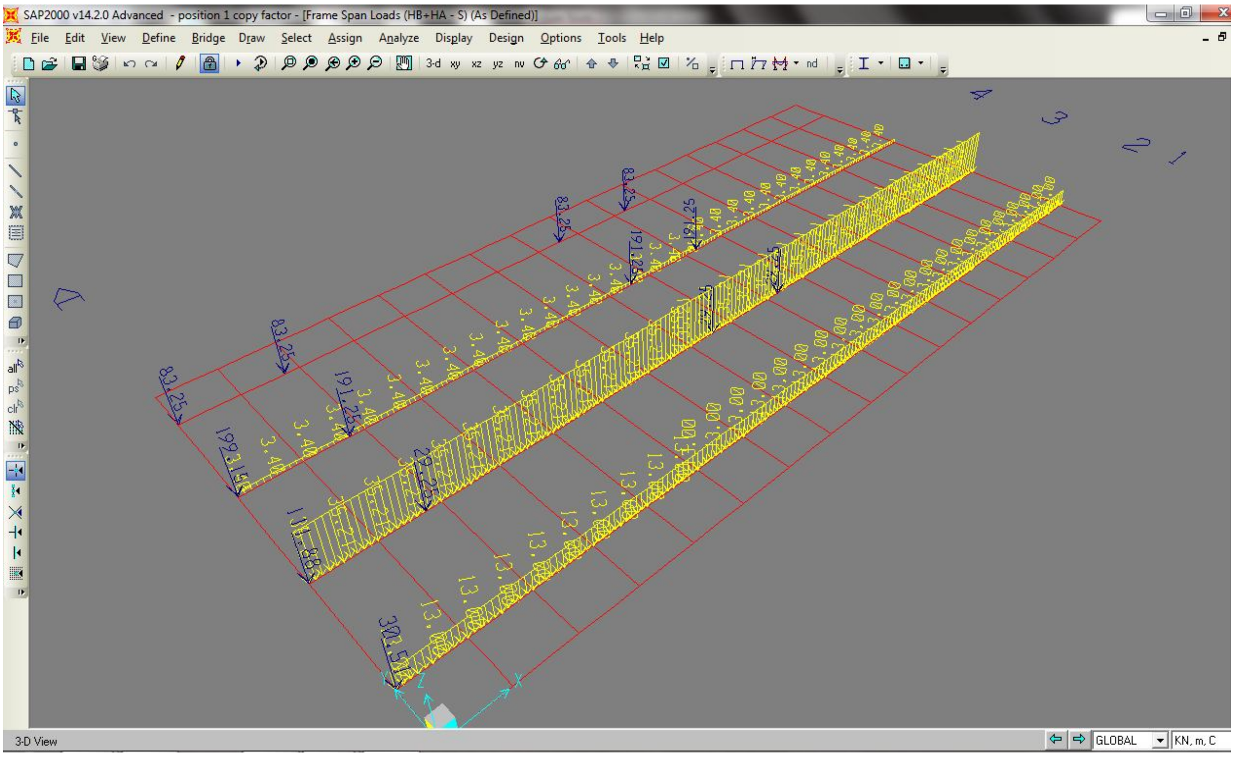


Fig. (4.9): (HB+HA) Loading- shear.

4.4.2. Load cases and load combination:

- The load cases have been used in accordance with BS 5400 as follow:

Table (4.1) : Load cases

	Load case	Group
1	Self-weight	Dead load
2	Surfacing	Superimposed dead load
3	Railing	Superimposed dead load
4	HA alone	Live load
5	HB + HA	Live load

- Load combinations using strength design factor in the BS5400 as critical combinations were considered as follow:

Table (4.2) : Load combinations

combination	Self-weight	Surfacing	Railing	HA alone	HB + HA
ULS C 1	1.15	1.75	1.2	1.5	-
ULS C2	1.15	1.75	1.2	-	1.3
SLS C1	1	1	1	1	-
SLS C2	1	1	1	-	1

- **Result of HA alone :**

Table (4.3): analysis results due to HA

Load combination	Beam	
	M max (KN.m)	Shear max (KN)
ULC1 (edge girder)	2593.1	578.76
ULC1 (internal girder)	2921.5	745.71
SLS C1 (edge girder)	1960	445.44
SLS C1(internal girder)	2125.35	534.69

- **Result of HA+HB:**

1-Position (1):

HB Loading case (1): straddling one lane.

Center of gravity of the HB vehicle divides the distance between center line of the bridge and near axle equally.

Table (4.4): analysis results due to HB position (1)

Load combination	**Beam**	
	M max (KN.m)	Shear max (KN)
ULS C2 (edge girder)	2712.28	684.27
ULS C2 (internal girder)	3000.91	921.47
SLS C2 (edge girder)	2185.53	552.87
SLS C2 (internal girder)	2374.51	722.29

2-Position (2):

HB Loading case (1): straddling one lane.

The Center of gravity of the HB vehicle located in center line of the bridge.

Table (4.5): analysis results due to HB position (2)

Load combination	**Beam**	
	M max (KN.m)	Shear max (KN)
ULC2 (edge girder)	2663.39	684.27
ULC2 (internal girder)	2998.08	921.47
SLS C2 (edge girder)	2148.7	552.87
SLS C2 (internal girder)	2375.61	722.29

3-Position (3):

HB Loading case (1): straddling one lane.

One of inner axes of the HB vehicle is placed in center line of the bridge.

Table (4.6): analysis results due to HB position (3)

Load combination	**Beam**	
	M max (KN.m)	Shear max (KN)
ULC2 (edge girder)	2712.13	684.27
ULC2 (internal girder)	2956.77	921.47
SLS C2 (edge girder)	2179.26	552.87
SLS C2 (internal girder)	2336.49	722.29

4.5.Design ofGirder:

- **Output data :**

From previous analysis the worst cases of loading as follow:

The girder:

The worst cases of loading due to position (1) of HB vehicle where the center of gravity of HB vehicle located in the mid of the bridge span.

Therefore ;

The maximum moment = + 3000.91 KN.m

The maximum shear = 921.47KN

f_{cu} = 35 N/mm^2	fy = 410 N/mm^2
h = 1420 mm	b = 500mm
cover = 30 mm	AS min = 0.13%

γ_{f3} = 1.2

The maximum moment = 3000.91 KN.m

The maximum shear = 921.47 KN

- **Flexural reinforcement :**

The design moment = 3000.91 × γ_{f3} = 3601 KN.m

Use ∅ = 25mm

d = h – c - ∅/2 = 1420 – 30 – 12 = 1378 mm

$k = \frac{M}{bd^2 f_{cu}}$ ⟶ (4.5)[11]

$= \frac{3601 \times 10^6}{500 \times 1378^2 \times 35} = 0.11 < 0.156$

Compression reinforcement is not required

The lever arm (Z) = 0.95d

= 0.95 × 1378 = 1309 mm

$As = \frac{M}{0.95\, f_y\, z}$ ⟶ (4.6)[10]

$= \frac{3601 \times 10^6}{0.95 \times 410 \times 1309} = 7062.8\ mm^2$

Use T 25 ⟶ Area of bar = 491 mm^2

As provide = 15 × 491= 7365 mm^2

Use 15 T 25 (As = 7365 mm^2)

- **Distribution steel (As min) :**

As min = $\frac{0.13\, bh_f}{100} = \frac{0.13 \times 1000 \times 500}{100} = 650\ mm^2$

Use T16 mm ⟶ Area of bar = 201mm^2

Use 4T16 (As = 804 mm^2)

- Check maximum allowable shear **stress :**
- From analysis :

Shear force ; V = 921.47 × γ_{f3} =1105.8KN

Shear stress ; $\nu = \frac{V}{bd} = \frac{1105.8 \times 10^3}{280 \times 1378}$ = 1.6 N/mm^2< 0.75 $\sqrt{f_{cu}}$

Max shear stress = 0.75 $\sqrt{f_{cu}}$ = 4.44 N/ mm^2

Shear OK

- **Design of shear reinforcement :**

Shear force ; V = 1105.8 KN

Shear stress ; ν = 1.6 N/ mm^2

Critical Shear stress ;ν_c:

$$\nu_c = \frac{0.27}{\gamma_m}\left(\frac{100\, As_p}{b_w\, d}\right)^{1/3}(f_{cu})^{1/3} \longrightarrow (4.7)$$ [10]

γ_m = 1.25 As_p = 7365 mm^2 b_w = 280mm

ν_c = 0.88N/ mm^2

$\nu_c\xi_s \longrightarrow$

$\xi_s = \left(\frac{500}{d}\right)^{1/4}$ = 0.776 >0.75 (take ξ_s = 0.75)

$\nu_c\xi_s$ = 0.75 × 0.88 = 0.66< 1.6

$\therefore \nu > \nu_c\xi_s$

Shear reinforcement required.

$\therefore \frac{A_{sv}}{S_v} \geq \frac{b\ [\ \nu + 0.4 - \nu c\ \xi s\ ]}{0.87\ f_{sv}}$ ⟶ (4.8)[10]

Use ∅ 12mm ⟶ 2 legs ; A_{sv} =226mm^2

$\frac{A_{sv}}{S_v} \geq \frac{280\ [\ 1.6 + 0.4 - 0.66\ ]}{0.87 \times 275}$

$S_v \leq \frac{226}{1.7}$

$\therefore S_v \leq 132$ mm

$S_v = 130$ mm c/c

Use ∅ 12mm @130 mm c/c on side face of beam

- **Check deflection :**

For simply supported beam

$l_e = l_o + d = 16600+1378 = 17978$mm

$\frac{l_e}{d} = \frac{17978}{1378} = 13.04$

Basic span / (depth)ratio = 20 from BS8110 Table (3.9) [11]

Service stress ; $f_s = f_y(\frac{5}{8\beta_b})\ (\frac{A_{s\,req}}{A_{sp}})$

$\beta_b = 1$ $(\frac{A_{s\,req}}{A_{sp}}) = 1$

$f_s = 410\ (\frac{5}{8})1 = 256.25\ N/mm^2$

From analysis:

Maximum service moment obtained from position (2) for combination.

SLS C2: M = 2375.61KN.m

$$\frac{M}{bd^2} = \frac{2375.61\times 10^6}{500\times 1378^2} = 2.5$$

Modification factor for tension reinforcement = 0.85 BS 8110 table (3.10)[11]

Modified $\frac{span}{depth}$ ratio= 20 × 0.85 = 17 > 13.04

$\frac{l_e}{d}$ < Modified span / (depth) ratio

Deflection is ok.

- **Check torsion :**

No torsion moment obtained from analysis.