# MIEMESTERY OF COMMUNICATIONS: DIFFECTIONATE GENERAL-OF LAND TRANSPORT AND INLAND WATERWAYS

# TENDER DOCUMENTS FOR NEW RAILWAY LINE FOR CENGKARENG AIRPORT CONSTRUCTION: PROJECT

# STRUCTURAL CALGULATION SHEETS

PACKAGE L CIVIL AND ARCHITECTURAL WORK

5 of 11

ANGUST TORK

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タシスノシュ作家

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#### §1. Design criteria

1-1 Type of structure

Elevated structure of Rahmen (Rigid frame) type made of R.C.

- 1-2 Type of track

  Ballast type track
- 1-3 Type of substructure

  Foundation piles are P.C. piles driven into the ground.
  - (1). Type of footing.

    Cannected type footing.
  - (2). Bearing stratum

    Dcs stratum. Supposed N > 30
- 1-4 Design Load
  - (1). Dead Load (Unit Weight)

Track assembly weight 0.45 1/m

Ballast 1.9 1/m<sup>3</sup>

Reinforced Concrete 2.5 1/m<sup>3</sup>

Plain Concrete 2.35 1/m<sup>3</sup>

2

Steel materials

7.85 m.3

Handrail

0.2 m

Material will be used actual unit weight of relevant

#### (2). Train Load

Train Load will be equivalent to KS-16

(K-Loading)

(S-Loading)

Equivalent uniformly distributed load

		KS-16	
span (m)	W M1 (t/m)	W M2 (5/m)	W 5 (*/m)
7	5.6	4.8	6.4
8	5.4	4.6	6.0
9	5.2	4.4	5.8
10	5-1	4.2	5.6

WM1: Applied for positive span bending moment, also for negative span bending moment at the tirst support point.

WM2: Applied for negative bending moment at the intermediate support point.

Ws: Applied for shearing stress.

## (3). Impact Load

The impact of train load shall be the train load multipied by the following impact coefficient Impact coefficient (KS-Loading)

span l	(m)	0	5	10	20
impact coefficie	nt(is)	0.60	0.48	0.43	0.37

For the double track structure, impact coefficient is reduced followed the equation.

$$i = i_0 \times (1 - \frac{l}{200})$$
  
l: Span length (m)

#### (4). Centrifugal Load

The magnitude of centrifugal load shall be the train load multiplied by the following coefficient as shown below. The working height of load is 1.8<sup>m</sup> above the rail level. The acting direction of load is horizontal and right angle to the track.

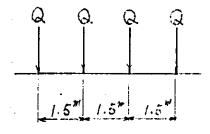
Curve Radius R (m)	Centritugal Coefficient &
R≦ 700	0.12
700 < R ≤ 1000	0.10
1000 < R ≤ 1800	0.08
1800 < R	0

#### (5). Train Lateral Load

Train lateral load under KS loading scheme shall be Q loading diagram as shown on the figure which is 15% of a driving axle load per track under K-loading scheme working horizontall on the track at rail level in direction of right angle to the track.

The line with

In the case of structure supporting two or more tracks. Train lateral load is assumed as the load of only one track.



#### (6). Brake Load and Traction

Brakeload and traction load per track shall be the Value as indicated below, working parallel to the track in the track center profile at 1.8 m above rail level.

Brakeload	15% of the train load
Traction load	25% of the weight of the driving axle

#### (7). Earth Pressure

Coulomb and Rankine's coefficient of earth pressure will be used

#### (B). Seismic Effect

Seismic effect of eartquake is assumed as dead load and surcharge load multiplied by seismic coefficient plus seismic earth pressure.

Kn = 0.10 .... in horizontal direction. Kv = 0 .... in vertical direction.

#### (9). Temperature Load

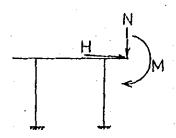
The temperature change considered in the structural analysis of statically indeterminate structure and the drying shrinkage shall be as follows:

Temperature Change ± 10°C

Drying Shrinkage - 15°C

Coefficient of temperture swelling of reinforced concrete shall be 1×10-5/10°

110). Load caused by catenary pole



Ordinary case

$$\begin{cases}
M = 5.0 & tm \\
N = 2.0 & t \\
H = 0.5 & t
\end{cases}$$

#### (11). Perestrian Load

For the structure of station and platform

Slab

500 kg/m²

beam

350 Kg/m²

earthquake case 210 kg/m²

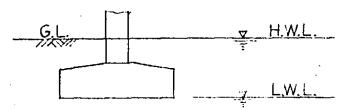
1-5. Other conditions

(1). Effect of Gelber girder

Construction of shoe used for Gelber girder is made, combined rubber shoe and steel rod stopper. Employed the said type of shoe for both support ends and accordingly eliminated distinction between movable and fixed end, shoe are supposed to share the load evenly half and half.

(2). Water level

Water level for the design is supposed as follows.



L.W.L. Stays at the bottom face level of footing.

H.W.L. .... Slays at ground level.

(3). Concrete minimum Cover

- 1-6. Material and allowable stress
  - 11). Material
    - 1). Concrete Ock = 240 kg/cm²
    - 2). Reinforcing bar SD30
    - 3). Max size of coarse aggregate 25 mm
  - (2). Allowable Stress
    - 1). Reinforced Concrete

			Design stress
Allowable	ompressive stres	5	90 1
Allowable Shearing Stress	Diagonal tension member	Bending shear Tai Punching shear Tap	
Allowable bon	ding stress	Deformed reinforcing bar	,2"

# 2). Reinforcing Bar

Type of reinforcing bar	S D 3 O
Allowable tensile stress determined by yielding point	1800 kg/

# Allowable stress for analysis, against cracking

Surrounding	Allowable value correspond- ing to dead load				
condition	Slab			beam	
Permanent wet	Ssa1 = 1200	Kg/ /cm²	55a1=	1400	Kg/ /un2
condition	Saz= 1400	4	5592=	1600	"
Alternate dry and wet condition	Ssa1= 800	"	054,=	1000	<i>"</i>
wet condition	Ssaz= 1000	, "	0592=	1200	"

$$S_{501}: \alpha = \frac{Sl+i}{Sd+Sl+i} \ge 0.25$$

$$Ssan: d = \frac{Sl+i}{Sd+Sl+i} = 0.25$$

Od: Tensile stress of re-bar subjected dead load

Seti: Tensile stress of re-bar subjected

train load and impact load

# 1-7 Allowable stress, subjected combined load

# (1). Track carrying structure

Combination of load	GivenExtra
D+T+1(+E)	1.00
D+T+1+C (+E)	1.00
D+T+1+C+TL(+E)	1.15
D+T+I+B(+E)	1.15
D + TE(+E)	1-15
D+S(+E)	1.50

D: Dead load

T: Train load

1 : Impact load

C: Centrifugal load

TL: Train lateral load

TE: Temperature load

B : Brake load

S : Seismic Force

E : Catenary pale

Load listed above with (+ ) expression is considered when the combined load including (+ ) brought dangerous result.

# (2). Platform structure.

Combination of load	Given Extra
D+P+EP	1.00
D + P + EP+TE	1.15
D + P + EP+S	1.50

D : Dead load

P: Pedestrian load

EP: Earth pressure

TE: Temperature load

S : Seismic Force

1-8. Allowable Reaction of pile.

 $Q = 30 \cdot \overline{N} \cdot A_{P}$ 

 $\overline{N}$ : Mean N value obtained from the N values measured within 4D vertical distance from pile bottom.  $\overline{N}$  = 25

AP: Base area of pile. AP =  $\frac{1}{4} \pi \times 0.35^2 = 0.0962$ 

Q: Ultimate reaction of pile.

D: Diameter D = 0.35 m

 $Q = 30 \times 25 \times 0.0962 = 72^{t}$ 

Under ordinary condition  $Ra = \frac{Q}{F} = \frac{72}{3} = 24 \frac{1}{P_{e}P_{e}}$ 

Under the condition of ordinary plus temporary

$$R_A = \frac{72}{2} = 36 \frac{1}{P_{ile}}$$

Under the condition of earthquake

$$R_a = \frac{72}{1.5} = 48 \frac{1}{Pise}$$

Over-turning analysis

Under Ordinary condition

Ratio of minimum and maximum reaction of pile shall be 0.3 or more.  $\frac{R \min}{R \max} > 0.3$ 

1-9. Calculation for the pile elasticity, horizontal direction

$$Kn = 0.4 \times \alpha' \times \alpha E_o Bn^{\frac{3}{4}}$$

Where

Kh : Coefficient of elasticity,

horizontal direction ( kg/cm3)

d' : Correction factor for the pile sides

 $\alpha' = 1.2$  (circular section)

d: For the permanent load d=2

For the Temporary load & =4

E. : Deformation factor of the ground.

Assumed the average N value to be

Eo = 2 kg/cm² as observed from the

boring log.

Bn: Equivalent width of the pile

$$Bh = \int D \times l_m$$

D: Diameter of pile

lm: Depth of the first nonmove pint.

$$\beta = \sqrt{\frac{Kn \cdot D}{4 \cdot E \cdot I}}$$

D: Diameter of pile

1: Moment of inertia ofpile

E: Modulus of elasticity. E=3.5×105

$K_{h} = 0.32 ( \angle E_{\circ} )^{1.103} \times E_{\circ}$	) × (E[) -0.103
$\ell_m = \frac{\pi}{2\beta}$	D = 35 cm
$E = 3.5 \times 10^{5}$ kg/mj <sup>2</sup>	
$E_0 = 2 \frac{k_2^4}{cm^2}$	

Under the condition	For the permanent load	For the Temporary load				
(∝.E) <sup>-0,103</sup>	9.91	21.29				
	62/30	62130				
D-0.310	0.332	0.332				
(E·[) <sup>-0.103</sup>	0.086	0.086				
Kn	0.091	0.195				
β	2.46 × 10 <sup>-3</sup>	2.98 × /3 <sup>-3</sup>				

1-10. P.C. Pile

# (1). Given conditions for preparing MN Diagram

	Allowable compressive stress of concrete	Allowable tensile stress of Tendon used for P.C. pile		
1.0	150 Kg/cm²	90 <sup>kg</sup> /mm²		
1.1	165 "	100 "		
1.2	180 '	110 "		
1.5	225 "	/35 "		
1.65	250 "	150 "		

Young's modulus of concrete.

Young's modulus of Tendon

$$n = \frac{Es}{Ec} = 5.7$$

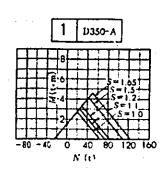
Design slandard of concrete stress

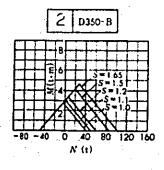
Prestressing Tendon

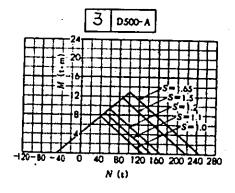
$$Spa = 0.6 Spu or 0.75 Spy$$
  
= 90  $^{49}/mm^{2}$  (\$\phi 7.0 \text{mm}^{mm}\$)

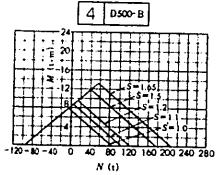
# Where, concrete stress shall be 50 > 0

Diameter of pile (mm)	Wall thick- ness of pile (mm)	pile prestressing	Cross sectional area of Tendon used for P.C. pile (cm²)	MN Dia- gram No.
3 <i>5</i> 0	65	Α	3.1	1
350		В	6.1	2
 <b>500</b>	90	A.	6.1	3
500		В	/2.2	4

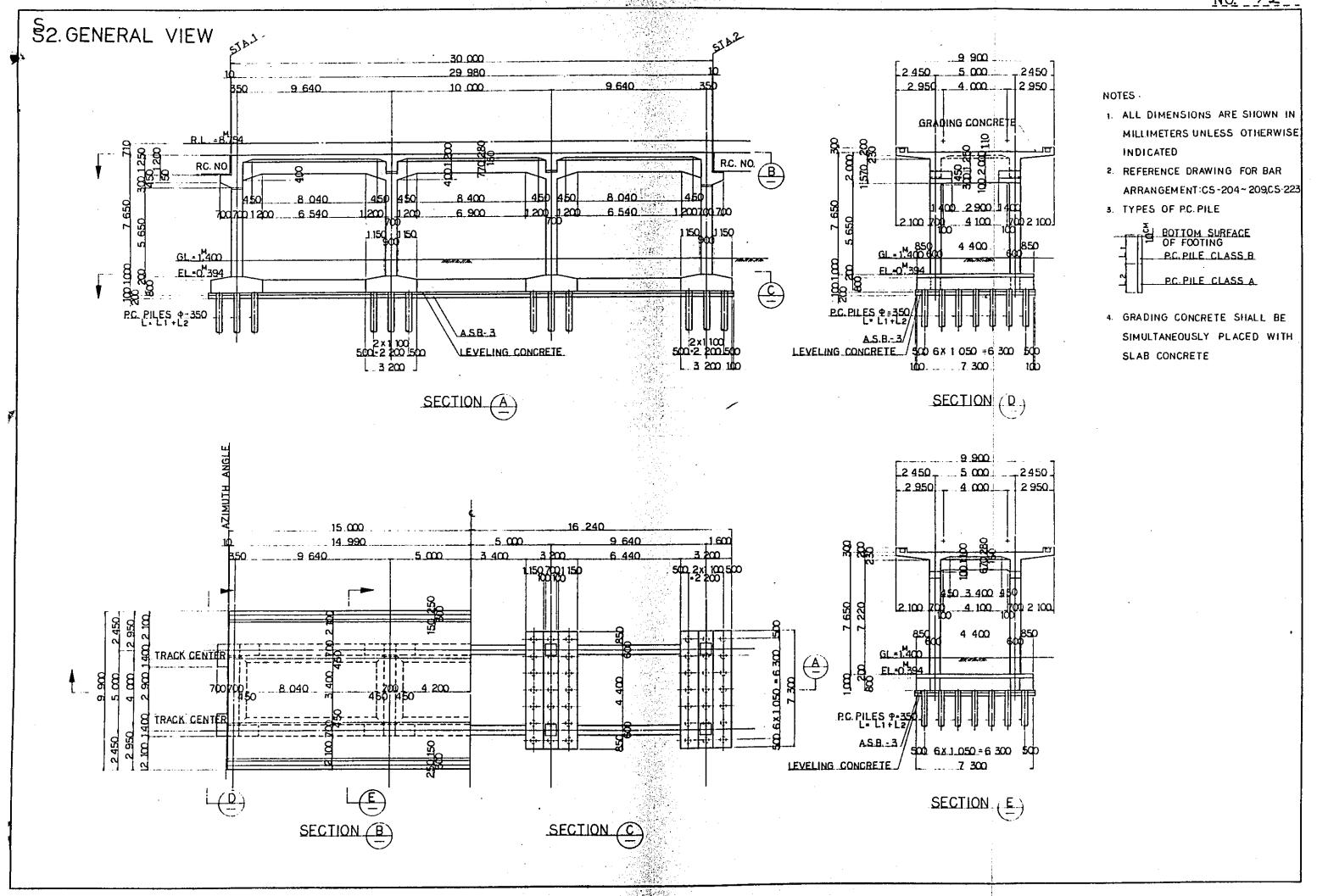








MN Diagram

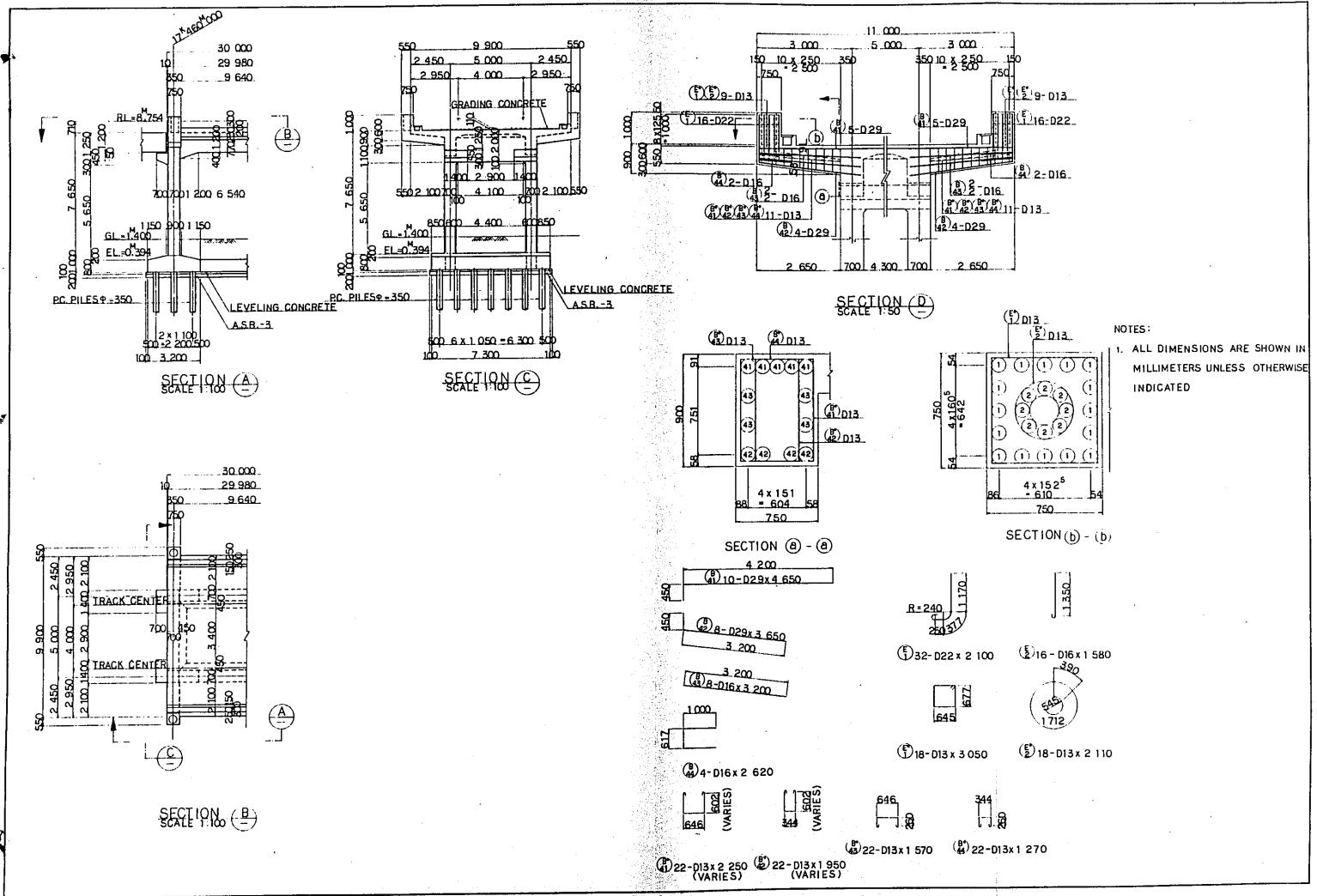


#### DIMENSION SCHEDULE

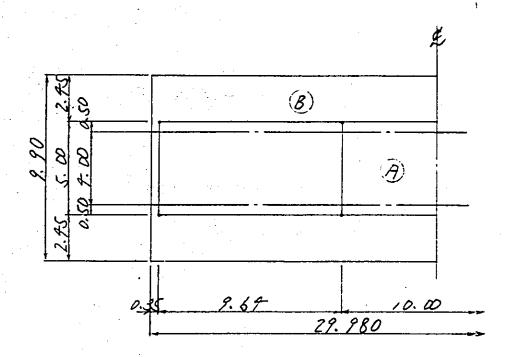
		STATION	R.L.	L1	L2	AZIMUTH ANGLE	CENGKAREN AIRPOAT LINE COORDINATE	RC. NO.
V090	STA.1	17 <sup>K</sup> 270 <sup>M</sup> 000	8. <sup>M</sup> 754	8 000	13 000	345 30 45	U.12 121 <sup>M</sup> 415 T. 1 070 <sup>M</sup> 633	R.C. 18
	STA.2	17 <sup>K</sup> 300 <sup>M</sup> 000	•				U.12 128 <sup>M</sup> 920 T. 1 099 <sup>M</sup> 679	RC.19
	STA.1	17 <sup>K</sup> 310 <sup>M</sup> 000	8 <sup>M</sup> .754	8 000	13 000	345 30 45"	U.12 131M422 T. 1 109M361	RC.19
V091	STA.2	17 <sup>K</sup> 340 <sup>M</sup> 000	•			•	U.12 138.927 T. 1 138.407	RC.20
V092	STA.1	17 <sup>K</sup> 350 <sup>M</sup> 000	8 <sup>M</sup> 754	8 000	13 000	345° 30′ 45″	U.12 141.429 T. 1 148.089	RC. 20
	STA.2	17 <sup>K</sup> 380 <sup>M</sup> 000	•			•	U.12 148 934 T. 1 177 135	RC.21
V007	STA.1	17 <sup>K</sup> 390 <sup>M</sup> 000	8. <sup>M</sup> 754	8 000	13 000	345° 30′ 45″	U.12 151 <sup>M</sup> 435 T. 1 186 <sup>M</sup> 817	RC. 21
V093	STA.2	17 <sup>K</sup> 420 <sup>M</sup> 000	,			•	U. 12 158 941 T. 1 215 863	RC. 22
V094	STA.1	17 <sup>K</sup> 430 <sup>M</sup> 000	8. <sup>M</sup> 754	8 000	12 000	345° 30′ 45″	U. 12 161 442 T. 1 225 545	RC.22
	STA.2	17 <sup>K</sup> 460 <sup>M</sup> 000	,			•	U.12 168.947 T. 1 254.591	RC.23

#### NOTES:

- ALL DIMENSIONS ARE SHOWN IN MILLIMETERS UNLESS OTHERWISE INDICATED
- 2 REFERENCE DRAWING FOR GENERAL VIEW :CS-202



## §3. Calculation of Slab



- [1] Slab for calculation

  Slab A ----- Two way Slab

  Slab B ---- Cantilever Slab
- (2) Calculation of slab (1)

  Four sides fixed span

  ldx = 5.00 0.70 = 4.30 "

  ldy = 10.00 0.70 = 9.30;

  Four sides semi fixed span

  lex = 4.30 + 0.28 = 4.58 "

ley = 9.30 + 0.28 = 9.58.

# · Span ratio

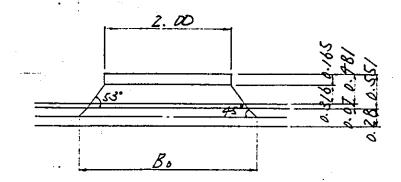
$$md = \frac{ldx}{ldx} = \frac{4.30}{9.30} = 0.46 > 0.4$$

$$me = \frac{l_{24}}{l_{eij}} = \frac{4.58}{9.58} = 0.48 > 0.4$$

From the above, the slab is considered as a two-way slab for calculation.

# 1. Load Calculations (1) Dead Load

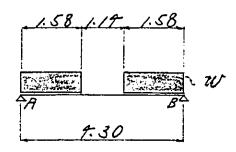
Effective width



Bo = 2.00 +1.5 × (0.316 + 0.07) + 0.28 = 2.86 m

(a) Weight of track assembly

W = 0.45 /m/2.86. = 0.157 /m2



RA = RB = 0.157 × 1.58 = 0.25 +

i) At the % point  $M = 0.25 \times 1.075 = 0.157 \times 1.075^2 \times 1/2 = 0.18^{tm}$   $W = \frac{32 \times 0.18}{3 \times 7.30^2} = 0.10^{t/2} > W = 0.09^{t/2}$ 

$$W_{2/2} = \frac{B \times 0.20}{9.30^{2}} = 0.09 \%$$

Sloping concrete 
$$2.35\% = 0.07 = 0.165\%$$

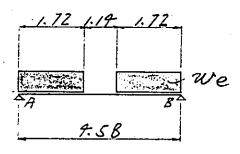
Ballast  $1.9\% \times 0.481 = 0.919\%$ 

Slab  $2.5\% \times 0.28 = 0.700\%$ 

Nd1 = 1.78 1/2

(b) Uniformly distributed load

EWd = 0.10 + 1.78 = 1.88 1/2



RA = RB = 3.73 × 1.72 = 6.42 +

i) At the  $\frac{9}{4}$  point  $M^{2}/_{4} = 6.42 \times 1.145 - 3.73 \times 1.145 \times \frac{9}{2} = 5.22^{+m}$   $W^{2}/_{4} = \frac{32 \times 5.22}{3 \times 4.58^{2}} = \frac{2.65}{3} \times \frac{4}{5} > W^{2}/_{2} = 2.11 \times \frac{9}{5}$ 

ii) At the  $\frac{9}{2}$  point  $M = \frac{9}{2} = 6.42 \times 2.29 - 3.73 \times 1.72 \times (2.29 - \frac{1.72}{2})$   $= 5.53^{+m}$   $W = \frac{8 \times 5.53}{4.582} = 2.11$ 

Reduction of impact coefficient 
$$i = 0.490 \times (1 - \frac{4.58}{200}) = 0.479$$

- 2. Bending moment
- (1) Dead Load
- (a) Shaving of load

  ldx = 4.30" ldy = 9.30"

  conefficient of load shaving in the direction of x or y

$$Cx = \frac{9.30^{2}}{9.30^{4} + 9.30^{4}} = 0.956$$

$$cy = \frac{9.30^{+}}{4.30^{+}9.30^{+}} = 0.044$$

(b) Shared Road

$$Wdx = 1.88 \frac{1}{100} \times 0.956 = 1.80 \frac{1}{100} \times 0.099 = 0.08 \frac{1}{100}$$

$$Wdy = 1.88 \times 0.099 = 0.08 \frac{1}{100}$$

(C) Tonsional Coefficient

$$9dx = 9dy = \frac{5}{18} \times \frac{9.30^2 \times 9.30^2}{9.30^4 + 9.30^4} = 0.057$$

ii) At the span center point

$$Mdx = \frac{1}{24} \times 1.80 \times 4.30^{2} \times (1 - 0.057) = 1.31^{4m}$$
 $Mdy = \frac{1}{24} \times 0.08 \times 9.30^{2} \times (1 - 0.057) = 0.27^{m}$ 

direction of  $\times$  or y

$$Clx = \frac{9.58^{+}}{4.58^{+} + 9.58^{+}} = 0.950$$

$$Cey = \frac{9.58^{4}}{4.58^{4} + 9.58^{4}} = 0.050$$

- (b) Coefficient of load sharing  $We+ix = 3.92 \times 0.950 = 3.72 \text{ fm}^2$   $We+iy = 3.92 \times 0.050 = 0.20$ 
  - (0) Torsional coefficient  $g_{\chi} = g_{y} = \frac{25}{36} \times \frac{7.58^{2} \times 9.58^{2}}{7.58^{2} + 9.58^{2}} = 0.151$
  - (d) Bending moment

    Train Load and Impact
  - i) At the support point  $Me + ix = -\frac{1}{2} \times 3.72 \times 9.58^2 = -6.50^{\pm m}$   $Me + iy = -\frac{1}{2} \times 3.92 \times 9.58^2 = -3.93^m$
  - ii) At the Span center point  $Me + ix = 1/2 \times 3.72 \times 4.58^2 \times (1 0.151) = 5.52^{+m}$   $Me + ij = 1/2 \times 0.20 \times 9.58^2 \times (1 0.151) = 1.30^n$

# (3) Combined moment

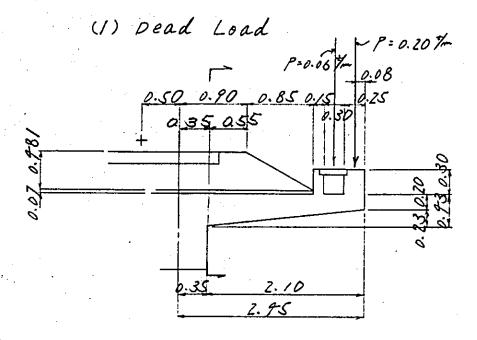
		Dead Load	Train Load and Impact	Total load
At the Support	ス	- 2.77	-6.50	- 9. 27
Point	y	-1.45	- 3.43	- 4.88
At the Span Center	Z	/.3/	5.52	6.83
Point	y	0.27	1.30	1.57

Allowable stress, safe against cracking  $EM \times 0.25 = 9.27 \times 0.25 = 2.32^{+m}$   $(Ml+i) = 6.50^{+m}$ 

There fore, (sa = 800 kg/om

# (3) Calculation of slab B

1. Calculation of Cantilever Slab



	Calculation	$\sim$	X	ル・エ
hand Rail		0.20	2.02	0.40
curb	2.5 1 × 0.30 × 0.25	0.19	1.975	0.38
Duct cover	2.5 " × 0.05 × 0.30	0.04	1.700	0.07
Cable		0.06	1.700	0.10
ballast stopped	2.5 × 0.15 × 0.30	0.11	1.475	0.16
Sloping concrete	2.35 × 0.07 × 1.40	0.23	0.700	0.16
bellast (A)	1.9 x 0.481 x 0.85 x /2	0.39	0.833	0.32
bellast (B)	1.9 × 0.481 × 0.55	0.50	0.275	0.14
Distributed load by Track weight	0.45 × /2.86 × 0.58	0.09	0.290	0.03
Slab (A)	2.5 × 0.20 × 2.10	1.05	1.050	1.10
Slab (B)	2.5 × 0.23 × 2.10 × /2	0.60	0.700	0.42
TOTAL				3.28

(4) Train Load and Impact

$$We = \frac{16}{1.5 \times 2.86} = 3.73 \%^{2}$$

$$Me = 3.73 \times 0.58^{2} \times 1/2 = 0.63 \%^{2}$$

(a) Impact Coefficient
$$l = 0.58^{m} - i = 0.587$$

$$Me + i = 0.63 \times (1 + 0.587) = 1.00^{+m}$$

Allowable stress, safe against Cracking

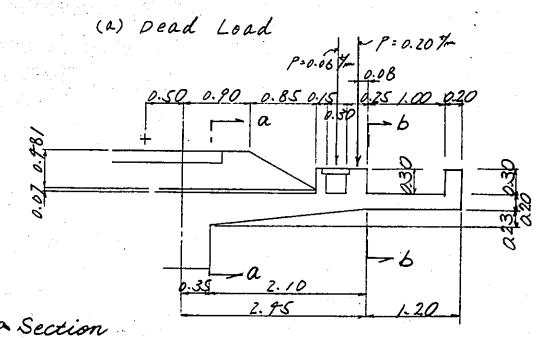
M x 0.25 = 4.28 x 0.25 = 1.07 tm > Meti = 1.00 tm

Therefore (sa = 100 //m

		A SA	lo h		B s.	la L
	Direction	· of railway	Direction 0	+ nailway	Cantilever	•
	At SUPPORT	of pailway ction At Spam Centel Point	At Support	At spam. Centerpoint	Cantillul	3220
M (tm)	(-2.77) -9.27	6.83	-4.88	1.57	3.2B	4.28
N (i)						
S (1)						
b (cm)	100	100	100	100	100	100
h (cm)	93	28	43	28	<i>9</i> 3	73
d (cm)	39.5	24.5	37.B	22.9	39.5	39.5
d' (cm)	<u> २.८</u>	3.5	5. Z	5./	3.5	3.5
As (cm²)	3:5 1018-30000 1016-300 11	019-150 de	D16-4-00 "	1	1019-300C	1019-300 ac
		= 19.11		= 6.34	= 16.17	= 16-17
p	0.00409	0.00780	0.00215	0.00277	0.00409	0.00409
As' (cm²)				4.3		
p'	•					
e=M/N (cm)						
e = M/N + u				·		
e=M/N-u						
e/h						
d/e						
d'/h					,	
d'/d			·			
Nelbd*(kg/cm²)		,				
k			·			
С .				· · · · · · · · · · · · · · · · · · ·		
j						
1/Lc	7.53	6.02	2.64	8.74		7.53
1/Ls	270	197	503	394	270	270
β= σs/σc						· · · · · · · · · · · · · · · · · · ·
$\sigma c = (k_R/cm^2)$	44.8	68.5	33.0	26./		20.7
OS (kg/cm²)	(480)	1670	1720	1180	570	790
T (kg/cm²)	(RQD)			-		
σsα (kg/cm²)	(800)	1800	4	3	1 000	1800
σca (kg/cm <sup>*</sup> )	90	90		4		90
Ta (kg/cm²)	(D)					
Combination	$D+T+I_{-}$	D+I+I	D+T+1	D+I+I	0	D+T+.
Homogram number	M-1	, ,	<u> </u>			LLCA

### (4) Calculation of the contilever slab for equipment space

### 1) Calculation of loads



	Calculation	N (e)	X (m)	N·X
hand Rail		0.20	2.02	0.90
CURB	2.5 1/2 × 0.30 × 0.25	0.19	1.975	0.38
puct lib	2.5 " × 0.05 × 0.30	0.04	1.700	0.07
cable		0.06	1.700	0.10
ballast stopped	2.5 x 0.15 x 0.30	0.11	1.475	0.16
Sloping concrete	2.35" × 0.07 × 1.40	0.23	0.700	0.16
bellast (A)	1.9 x 0.48/ x 0.85 x //2	0.39	0.833	0.32
bellast (B)	1.9° × 0.481 × 0.55	0.50	0.275	0.14
Distributed load by Track weight	0.45 x /2.86 x 0.58	0.09	0.290	0.03
Slab (A)	2.5 × 0.20 × 2.10	1.05	1.050	1.10
Slab (B)	2.5 x 0.23 x 2.10 x /2	0.60	0.700	0.42
widening of slab	2.5"×0.20 × 1.20	0.60	2.700	1.62

widening of our b	2.5 1/2 × 0.20 × 0.30	0.15	3.200	1.48
Pedestrian load	0.50 t/m² × 1.00	a50	2.600	130
total			.	668 cm

6-6 Section

	Calculation	(+)   \( \lambda \)	X cm)	N.X
widening of slab	25 × 0.20 × 1.20	0.60	0.60	аЗб
widening of curb	25 × 0, 20 × 0, 30	0.15	1.10	0.17
Pedestrians load	0.50 th × 1.00	0.50	0.50	0.25
total			٠	0.78

### Seress acting at section a-a

- 1) Pedestrian Load

  P = 0.50 1/2 × 0.50 = 0.25 +
  - M = 0.25 × 1.61 = 0.40
- 1) pead Load and Pedestrian Load  $Md+w = 668 + 0.40 = 7.08^{\pm m}$
- 3) Train Load and Impact

  We = 16 3.73 1/2

Me = 3.73 × 0.582 × 1/2 = 0.63 tm

Impact Coefficient

L = 0.58 - i = 0.587

Me+i = 0.63 x (1 + 0.587) = 1.00 tm

Resultant stress

Dead Load + Train Load and Impact (x=1.00)

M = 688 + 1.00 = 7.68 + 7.03 + 7.03 + 7.03

Pead Load + Train Load and Impact
Pedestrian Load (x-1.15)

M = (7.08+1.00) × 1/1.15 = 7.03 tm

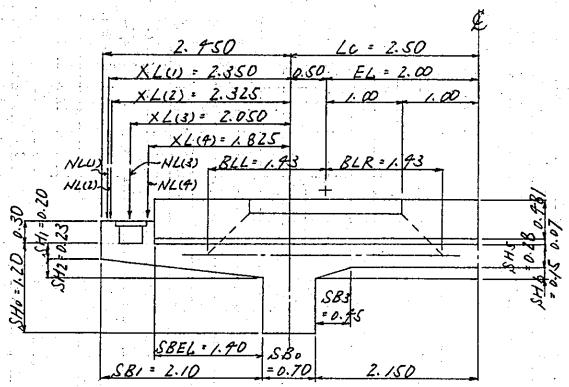
Allowable stress, safe against cracking  $M \times 0.25 = 7.68 \times 0.25 = 1.92^{+m} \times Meri = 1.00^{+m}$ The stress of th

Stress acting at section b-b  $Mb-b = 0.78^{em}$  (Dead load)

	Stres	s calcu	lation	
		•	<u> </u>	
	a-	a	6-6	
M (.tm)	6.68	7.68	0.78	
N (t)				
S (1)				
b (cm)		20	100	
h (cm)	4	<u> </u>	20	
d (cm)	3	25	16.7	
d' (cm)		3. <u>5</u>	3.3.	
As (cm²)	019-300	-22.27	016-300cc	
	016-30	) • )	= 6.62	
p	0.00	564	0.00396	
As' (cm²)				
				•
<i>p</i> .				
$e = M/N \ (cm)$				
e = M/N + u (cm)	· · · · · · · · · · · · · · · · · · ·	1 1 1 1 1 1 1 1 1 1 1 1 1 1 1 1 1 1 1		
e = M/N - u		<u> </u>		
e/h				
d/e				
d'/h				·
d'/d				
Ne/bd*(kg/cm²)				
k				
c				
j	•	· .		
1/Lc 1/Ls		72	7.62	
	26		279	
<del>                                     </del>		- 4 /		
$\sigma c = (kg/cm^2)$ $\sigma s = (kg/cm^2)$		33 /	-00	<del></del>
$\sigma s = (kg/cm^2)$ $\tau = (kg/cm^2)$	860	980	780	
$\sigma sa = (kg/cm^2)$				
σca (kg/cm²)	1000	1800	100	
Ta (kg/cm²)		90		
combination No magram	_ <i>_</i>	O+T+I	0	
number	M-1		<u>,</u>	

### St Calculation of torsional moment.

#### (1) Load Calculations



Distributed Width

Be = 2.00 + 1.5 x (0.3/6 + 0.07) + 0.28 = 2.86 m

1. Weight of elements on the slub except truck Weight

BL1 =1.75		BR1 =1.75
WAY S	gor Word	w.R.
2.45	5.00	2.45
		}

a). Distributed Load

Ballast

WL1 = WO = WR1 = 1.08 /m

- b). concentrated load
  - 1) hand rail NL(1) = 0.20 5/m
  - 2) CURB NL(2) = 2.5 1/m × 0.25 × 0.30 = 0.19 1/m
- 3) Cable and Duct lib

NL(3) = 0.06 /m + 2.5 /m × 0.05 × 0.30 = 0.10 /m

4) ballast stopper

c) Distributed loads by Track Weight

$$W = \frac{0.45}{2.86} = 0.16 \text{ fm}$$

2. Train Load (Single track)  $l = 10.00^{m}$ 

Distribution loads

WMZ = 4.2 x 2 = 8.40 t/m

- · AT END of viaduct
- · AT Intermedlate of viaduct

  LL = 10.00 m
- Points of Computing stress

  X, = 0.350 m

  X2 = 1.150 ° (1/2)

  X3 = 1.550 "

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*	** NELIKI MOMENT	r FUKUSEN	*				
059.6	0.0	p.4	0.0	0.0	2.5	4	
1.20	0.20	0.23	0.20	0.23	0.28	0.15	
0.70	2.10	2.10	0.45	1.40	1.40	-	
0.20	0.19	0.1	0.11	0.0			
2.350	2.325	2.050	1.825	0.0			1 .
0.20	0.19	0.1	0.11	0.0	٠		
2.350	2.325	2.050	1.825	0.0		-	
1.08	0.0	1.08	0.0		,	\$ \$ \$ \$	•
1.75	0.0	1.75	0.0				
0.16		0.16	0.16	2.94	2.94	2.94	2.94
1.08	•	2.0	0.35	1.55	0.0		
1.43		1.43	1.43		-		

FUKUSEN \*\* \*\* NEJIRI MOMENT

\*\*\*\* FUKUSEN-SAIKA \*\*\*\*

### STOUDL XXXXX

### TATE BARI NO NEJIRI MOMENT

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1.400	SMALL BEL = 1.400	
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MALL	SMALL HL =	
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	.20360	.11083	.00183
:	n ·	# 1E	15 =
	MAGE DANMEN NIJI MOMENT	NEJIRI DANMEN NIJI MOMENT	SLAB NO MAGE DANNEN NIJI MOMENT

SMALL K2 = -.0987

# KAJYU HENSIN NI YORU NEJRI MOMENT

NEJIRI KEISU

SMALL ED = 0.000	SMALL EL = 0.000	2 4 2 2
WD = 21.981	WL = 16.817 S	ī = .4i3
SHIKAJYU	RESSHA KAJYU III	SYÖGEKI KEISÜ SMALL II

HENSIN NI YORU NEJIRI MOMENT WA SHOJINAI

KOTEITAN MOMENT NO SANI YORU NEJIRI MOMENT

= .930 SHALL BL = 1.930 SHALL BR = 1.930 SHALL BR 0 = .93.  = .466 SHALL IL = .575 SHALL IR = .575  -11.889 HR = 11.889  8.403 HR = -8.403  3.276 HR = -3.276  1 D+L+1  9 2.780177  0 0.000 0.000 0.000  0 0.000 0.000 0.000  0 0.000 0.000 0.000	SMALL CY = .933 SMALL WLL = 2.940
SMALL IL = .575 SHALL IR = .6.403  HR = 11.889  HR = -8.403  HR = -3.276  1.	SMALL BLD =
HR HR MR 1.2.7801 2.5781 1.8871 0.000 0.0 2.578	SHALL IZ = .468
HH MR MR 2.7801 2.5781 1.8871 0.000 0.0 2.780	
780 .578 .000 .578 .578 .578	N. =-11,889
I D+L+; 2.78017; 2.57816; 1.887120 0.000 0.000 2.780 .17; 2.578 .120 1.887 .120	. H. J. H
	F = 1
	'n
	-10.086 7.129
	-9.354 6.612
	-6.846 4.839
	0.000 0.000
:	
	10.086 -7.129
	9.354 -6.612
	6.846 -4.839
	0.000 0.000

NEJIRI MOMENT NO GOSEI ( CARL & LARLE )

SHUBARI FUKUBU NO NEJIRI MOMENT BUNTANRITHU  L BARI  R BARI  ALFA L1 = .850	٠.						
R BÀRI ALFA L1 = .850 ALFA L2 = .790  R BÀRI ALFA R1 = .850 ALFA R2 = .790  X D L I D+L+I SHUBARI  1 0.000	SHUBA	RI FUKUBU NC	NEJIRI MON	TENT BUNTANR	THU		
R BÁRÍ ALFA R1 = .850 ALFA R2 = .790  X D. L. I D+L+I SHUBARI 1 0.000	<b>ب</b>		ALFA L1 =	•.	ALFA L2 =	. 790	
X     D     L     I     D+L+I     SHUBARI       1     0.000     =10.086     7.129     2.780    157       1     .350     -9.354     6.612     2.578    164       2     1.550     -6.846     4.839     1.887    120		BARİ	ALFA R1 =		ALFA R2 =	790	
X         D         L         I         D+L+I         SHUBARI           A         0.000         =10.086         7.129         2.780        177           1         .350         -9.354         6.612         2.578        164           2         1.550         -6.846         4.839         1.887        120	BARI		*				
1 .350 -9.354 6.612 2.578164 2 1.550 -6.846 4.839 1.887120		×	<b>a</b>	۰	H	D+L+I	SHUBARI NO
1 .350 -9.354 6.612 2.578164 2 1.550 -6.846 4.839 1.887120	MT.A		-10.086		2.780	177	
1.550 -6.846 4.839 1.887120	MT 1	.350	-9.354	6.612	2,578	164	14
	MT 2	1.550	-6.846	4.839	1.887	120	- 00

BUNTAN

,	140	560.	0.000
.177	191:	.120	0.000
-2.780	-2.578	-1.887	0.000
10.086 -7.129 -2.780	-6.612	-4.839	000.0
10.086	9.354 -6.612	6.846	0.000
0.000	.350	1.550	MT C 4.825
# P	H .	MT 2	N 1 C

R BARI

\*\*\*\* TANSEN-SAIKA \*\*\*\*

KAJYU HENSIN NI YORU NEJRI MOMENT

SHIKAJYU		= QN	UD = 21.981	SMALL ED = 0.000	0.00
RESSHA KAJYU	KAJYU	# 7	UL = 8.408	SMALL EL = 2.000	000
SYOGEKI	KEISU	SYOGEKI KEISU SHALL II =	.413	\ \ :	
	<b>*</b>	a :		H	0+1+1
MT A	0.000	0.000	9.135	3.769	12.905
MT 1	.350	0.00	8.473	3.496	11.968
HT 2	1.550	0.000	6.201	2,558	8.759
MT	4.825	0.000	0.000	0.000	0.000

KOTEITAN MOMENT NO SANI YORU NEJIRI MOMENT

SHALL HLL = 2.940         SHALL HRR = 11.889         SHALL HRR = 2.940         SHALL HRR = 11.889         SHALL HRR = 2.940         SHALL HRR = 11.889         SHALL	SHIKAJYU (CHUKAN SLAB)		SMALL CY # .933	. 933	SMALL 1107 = 1.764		
EKI KEISU         SHALL BLD = .930         SHALL BL = 1.930         SHALL IR = 1.930         SHALL IR = .575         SHALL IR = .575           EKI KEISU         SHALL IZ = .466         SHALL IR = .575         SHALL IR = .575         SHALL IR = .575           EKI KEISU         HL = -11.889         HR = -11.889         HR = -1.601           HIKAJYU         HL = 1.675         HR = -1.601           HOGEKI KAJYU         HL = 1.675         HR = -1.601           A 0.000         -10.086         3.159         1.414         -5.513           2 1.550         -6.846         2.144         .960         -3.742           C 4.825         0.000         0.000         0.000         0.000           1 0.000         10.086         -3.742         -3.742           A 0.000         0.000         0.000         0.000           1 0.000         0.000         0.000         0.000           1 0.550         -4.750         4.406           2 1.550         -2.695         -1.267         4.406           2 1.550         -2.695         -9.27         3.224           2 0.000         0.000         0.000         0.000	RESSHA KAJYU	<b>.</b>	SMALL WIL = 2	2.940	SMALL WLR = 2.940	SMALL WRL = 2,940	SMALL HRR = 2.9
EKI KĒISU EN HOMENT NO SA HIKAJVU HIMAJVU HIMA	SAIKAHABA	י נט	SMALL BLD =	.930		H	SMALL BRD = .9.
EN MOMENT NO SA  HIKAJYU  ESSHA KAJYU  HOGEKI KAJYU  X  A 0.000 -10.086 3.159 1.414  1 .350 -9.354 2.930 1.311  2 1.550 -6.846 2.144 .960  C 4.825 0.000 0.000 0.000  A 0.000 10.086 -3.970 -1.366  1 .350 9.354 -3.682 -1.267  2 1.550 .6.846 -2.695927  C 4.825 0.000 0.000 0.000	SHOGEKI KEISU		SMALL 12 =	.468	#	11	
HIKAJVU  ESSHA KAJVU  HOGEKI KAJVU  X  A 0.000	SHITEN MOMENT NO SA	-					
ESSHA KAJYU  HOGEKI KAJYU  X  D  L  I  A 0.000	SHIKAJYU			889	MR = 11.889		
NR = 1.675   NR = 1.675   NR = 1.666KI KAJYU	RESSHA KAJYU	;		579	MR = -4.824		
X     D     L     I       A     0.000     -10.086     3.159     1.414       1     .350     -9.354     2.930     1.311       2     1.550     -6.846     2.144     .960       C     4.825     0.000     0.000     0.000       A     0.000     10.086     -3.970     -1.366       I     .350     9.354     -3.682     -1.267       I     1.550     .6.846     -2.695    927       C     4.825     0.000     0.000     0.000	SHOGEKI KAJYU			675	. 16		•
X     D     L     I       A     0.000     -10.086     3.159     1.414       1     .350     -9.354     2.930     1.311       2     1.550     -6.846     2.144     .960       C     4.825     0.000     0.000     0.000       A     0.000     10.086     -3.970     -1.366       I     .350     9.354     -3.682     -1.267       2     1.550     .6.846     -2.695    927       4.825     0.000     0.000     0.000			:				
0.000       -10.086       3.159       1.414         .350       -9.354       2.930       1.311         1.550       -6.846       2.144       .960         4.825       0.000       0.000       0.000         0.000       10.086       -3.970       -1.366         .350       9.354       -3.682       -1.267         1.550       .6.846       -2.695      927         4.825       0.000       0.000       0.000	×	٥	-1		0+41		
1 .350 -9.354 2.930 1.311 2 1.550 -6.846 2.144 .960  C 4.825 0.000 0.000 0.000  1 .350 9.354 -3.970 -1.366  1 .350 9.354 -3.682 -1.267  2 1.550 .6.846 -2.695927	A 0.000	0.086	3.159	1.414			
2 1.550 -6.846 2.144 .960 C 4.825 0.000 0.000 0.000 A 0.000 10.086 -3.970 -1.366 1 .350 9.354 -3.682 -1.267 2 1.550 .6.846 -2.695927 C 4.825 0.000 0.000 0.000	1 .350	9.354	2.930	1.311		and the special specia	
C       4.825       0.000       0.000       0.000         A       0.000       10.084       -3.970       -1.366         I       .350       9.354       -3.682       -1.267         2       1.550       .6.846       -2.695      927         C       4.825       0.000       0.000       0.000	2 1.550	6.846	2.144	096"			
A 0.000 10.086 -3.970 -1.366 1 .350 9.354 -3.682 -1.267 2 1.550 .6.846 -2.695927 C 4.825 0.000 0.000 0.000	C 4.825	. 000.0	000.0	000.0			÷
A 0.000 10.086 -3.970 -1.366 1 .350 9.354 -3.682 -1.267 2 1.550 .6.846 -2.695927 C 4.825 0.000 0.000	Ξ.	:		- 1		1 1 1 1 1 1 1 1 1 1 1 1 1 1 1 1 1 1 1 1	
1 .350 9.354 -3.682 -1.267 2 1.550 .6.846 -2.695927 C 4.825 0.000 0.000 0.000	A 0.000	0.086	-3.970	-1,366			
2 1.550 .6.846 -2.695927 C 4.825 0.000 0.000 0.000	1 .350	9.354	-3.682	-1.267			
c 4.825 6.000 0.000 0.000	2 1.550	6.846	-2.695	- 927	i	:	
	C 4.825	0.000	0.000	000.0			

NEJIRI MOMENT NO GOSEI (SLAGLE LAACK)

							•
OHS	BAR	I FUKUBU NA	SHUBARI FUKUBU NO NEJIRI MOMENT BUNTANRITHU	IENT BUNTANI	RITHU	,	
	نـ	L BARI	ALFA L1 =	.850	ALFA L2 =	.790	
	Œ	R BARI	ALFA R1 = .850	.850	ÁLFÁ RZ =	. 790	
L BARI	±	-					
	-	*	<b>-</b>		H	0+1+1	SHUBARI NO BUNTAN
L H	∢	000.0	-10.086	12.295	5.183	7.391	
ž	<b></b>	350	-9.354	11.403	4.807	6.855	5.830
E	ÇI	1.550	-6.846	8.345	3.518	5.017	3.962
M	MT C	4.825	0.00	0.000	0.000	0.000	0.000
R BARI	Ξ		•			•	
F	∢	0.00	10.086	5.166	2.403	17.655	:
 TH		350	9.354	4.791	2.229	16.374	13,925
T.W.	64	1.550	978-9	3.506	1.631	11.983	6,463
M	Ü	4.825	0.000	0000	0000	0.00	0.000

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0		0		<u> </u>		0	0
** NE.	NEJIRI MOMENT	FUKUSEN	*	.,,			
10.00	5.0	7.0	0.0	0.0	2.5	4	
1.20	0.20	0.23	0.20	0.23	0.28	0.15	
0.70	2.10	2.10	27.0	1.40	1.40		
0.20	0.19	0.1	0.11	0.0			
2.350	2.325	2,050.	1.825	0.0			
0.20	0.19	0.1	0.11	0.0			1
2.350	2.325	2.050	1.825	0.0			
1.08	0.0	1.08	0.0	; ;	!	•	į
1.75	0.0	1.75	0.0		-		
0.16	0.16	0.16	0.16	2.94	2.94	2.94	2.94
1.08	2.0	2.0	0.35	1.55	0.0	:	
1.43	1.43	1.43	1.43				

\* \*\* NEJIRI MOMENT FUKUSEN \*\*

\*\*\*\* FUKUSEN-SAIKA \*\*\*\*

## TATE BARI NO NEJIRI MOMENT

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SMALL	SMALL	

	= .20360	IT = .11083		SMALL K2 =1022
The second secon	IJI MOMENT I		SLAB NO MAGE DANNEN NIJI MOMENT IS	SMALL K1 = .7368
	MAGE DANMEN NIJI MOMENT	NEJIRI DANMEN NIJI MOMENT	SLAB NO MAGE	NEJIRI KEISU

# KAJYU HENSIN NI YORU NEJRI MOMENT

		i
SMALL ED = 0.000	SMALL EL = 0.000	
WD = 21.981	WL = 16.817	SMALL 11 = .408
SHIKAJYU	RESSHA KAJYU	SYOGEKI KEISU

# HENSIN NI YORU NEJIRI MOMENT WA SHOJINAI

OTEITAN MOMENT NO SANI YORU NEJIRI MOMENT

SHIKAJY	SHIKAJYU (CHUKAN SLAB)	:	SMALL CY =		941	SMALL HDY = 1.780				
RESSHA KAJYU	KAJYU	,	SMALL 1	SMALL 1111 = 2.940	076	SMALL WLR = 2.940	SMALL URI	SMALL URL = 2.940	SMALL UR	SMALL WRR = 2.940
SAIKAHABA	BA		SMALL BLD =		.930	SMALL BL = 1.930	SMALL BR	= 1.930	SHALL BRD =	0 = .930
SHOGEKI KEISU	KEISU		SMALL 12	-1 - 11	894.	SMALL 11 = .575	SMALL IR	575		
SHITEN	SHITEN NOMENT NO SA	æ		*-		-				
SHIKAJYU	AJYU		독	=-12.029	029	MR = 12,029				
RESSI	RESSHA KAJYU		¥	U	8.893	MR = -8.893			· · · · · · · · · · · · · · · · · · ·	
SHOG	SHOGEKI KAJYU	-	뒫	ij	3,482	MR = -3.482	;			
L BARI		- 14		:	÷			‡		
	*	۵	-		. <del></del>	D+L+1	,			
M TM	000.0	-10.092	7	7.461	2.921	.289			. :	
MT 1	.350	-9.386	-	6.938	2.716	.269		:		
MT 2	1.550	-6.964	is.	5.148	2.015	.200				
AT C	3.000	0.000		0.000	0.00	0.000				
R BARI	r		· .	:					:	
MT A	000.0	10.092	-7	-7.461	-2.921	289				
H I	.350	9.386	9-	-6.938	-2.716	269				
MT 2	1.550	996.9		-5.148	-2.015	200		;		
MT C	5.000	0.000	,0	000.0	0.000	0.000				

NEJIRI MOMENT NO GOSEI (LOUBLE LAGER)

SHUBARI FUKURU NO NEJIRI MOHENT BUNTANRITHU

	;	-										
			SHUBARI NO BUNTAN	•	.229	.158	000.0			229	158	טטי ט
.790	.790		0+1+1	. 289	.269	.200	0.00		289	- 269	-,280	00000
ALFA L2 = .790	ALFA R2 =	•		2.921	2.716	2.015	0.000		-2.921	-2.716	-2.015	0.00
.850	850	5.	 	7.461	6.938	5.148	0.000		-7.461	-6.938	-5.148	0.000
ALFA L1 =	ÁLFÁ ŘÍ 🖺			-10.092	-9.386	-6.964	000.0		10.092	9.386	796.9	0.000
L BARI	R BARI		×	0.000	.350	1.550	5.000	,	000.0	.350	1.550	5.000
7	œ	L BAR1		HT A	MT 1	MT 2	D TM	R BARI	HT A	MT 1	MT 2	MT C

### \*\*\* TANSEN-BAIKA \*\*\*\*

KAJYU HENSIN NI YORU NEJRI MOMENT

; ;	· .				
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1 1	. 4	0.0	0:	0.0	0.000
SHALL	, •	,			
KEISU	×	0.00.0	.350	1.550	5.000
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pr.				i	
				:	
	a.	SMALL 11 = .408	SHALL I1 = .408  0.000 10.000	GEKI KEISU SMALL I1 = .408  X	GEKI KEISU SHALL I1 = .408  X

KOTEITAN MOMENT NO SANI YORU NEJIRI MOMENT

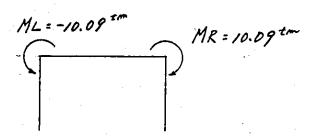
SHIKAJ	SHIKAJYU (CHUKĀN SĻĀB)	SLAB)	SMALL CY =	176	SMALL 469 = 1.780		· .		•
RESSHA KAJYU	KAJYU		SMALL 11LL = 2.940		SHALL WLR = 2.940	SMALL WRL = 2.940	SMALL	SMALL WRR = 2.9	
SAIKAHABA	ABA		SMALL BLD = .930		SMALL BL = 1.930	SMALL BR = 1,930	SMALL	BRD = .93	
SHOGEK	SHOGEKI KEISU		SHÂLL ÎŽ 🚊468	;	SHALL IL = .575	SMALL IR = .575	. :		
SHITEN	SHITEN MOMENT NO SA	SA	:	•					
SHI	SHIKAJYU		ML =-12,029	.029	MR = 12.029				
RES	RESSHA KAJYU		.π π	3.754	MR = -5.138		;		
SHO	SHOGEKI KAJYU		Æ 1.	1.757	MR = -1.725				
L BARI		:	•						
	×	٥	پ	<b></b>	D+L+1				
M T A	0.000	-10.092	. 3.291	1.471	-5.330	,			
. H	350	-9.386	3.061	1.368	-4.957		:		
MT 2	1.550	-6.964	2.271	1.015	-3.678	•	•		
M	5.000	0.000	0.000	0.000	0.800				
R BARI				:		1 1 1 1 1 1 1 1 1 1 1 1 1 1 1 1 1 1 1 1			
A TM	0.000	10.092	-4.169	-1.450	4.473				
Z FE	.350	9.386	-3.878	-1.349	4.159				
MT 2	1.550	496.9	-2.877	-1.001	3.086	: : : : :			
MT C	\$.000	0.000	0.000	0.000	0.000	·			

NEJIRI MOMENT NO GOSEI (Sungle track)
SHURARI FUKUBU NO NEJIRI NOMENT BUNTANRITHU

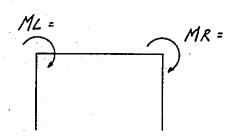
			SHUBARI NO BUNTAN		6.924	4.770	0.000		•	14.677	10.111	0.000
062.	790		D+C+1	8.754	8.141	0,0.9	0.000		18.557	17.258	12.804	0.000
ALFA L2 =	ALFA R2 =	•	eț.	5.556	5.167	3.833	0.000		2.635	2.450	1.818	. 000*0
.850	.850		_	13.291	12.360	9.171	000.0		5.830	5.422	4.023	0.000
ALFA L1 =	ALFA R1 =		۔ م	-10.092	-9.386	-6.964	0.000		10.092	9.386	496.9	0.000
BARI	BARI		×	0.000	.350	1.550	5.000		0.000	.350	1.550	3.000
	oc	L BARI		M TM	HT 1	MT 2	H C	R BARI	MT A	HT 1	MT 2	AT C

- 3. Rigid frame analysis on transverse section O-O of viaduct

  (AT. END of viaduct)
  - a) Dead Load

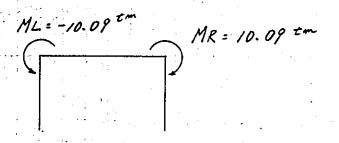


b) Train Load and Impact Load

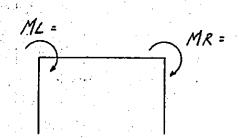


	single track	double track
ML	(12.295+5.183) = 17.98 +m	(7.129+2.780) = 9.91 tm
MR	(5.166 + 2.903) = 7.57 tm	(-7.129-2.780) =-9.9/tm

- 4. Rigid frame analysis on transvense section 2-2 of viaduct
  (AT Intermediate of viaduct)
  - a) Dead Load



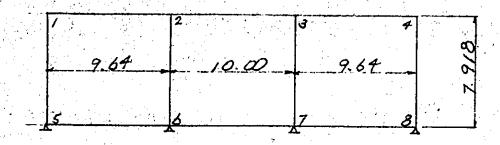
b) Train Load and Impact Load



	Single track	double track
ML	(13.291+5.556)×2 = 37.69 ±m	(7.961+2.921) × 2 = 20.76 +==
MR	(5.830+2.635)×2 = 16.93 +m	(-7.461-2.921) × 2 = -20.76 + m

§ 5. Rigid frame analysis on longitudinal direction of elevated structure

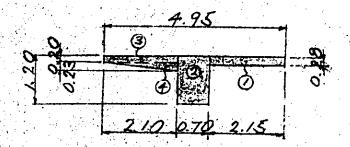
- (1) Elements for rigid frame analysis
  - 1. Conligulation and dimension



2. Cross-sectional anea and moment of Inextia

Of the member

(1) Member (1-2 2-3.3-4)



effective width

be= 9.64 × 1/2 + 0.35

= 5.17\*\*

	b (m)	h (m)	A (m)	y (m)	A.y (m)
$\bigcirc$	2.150	0.280	0.602	0-140	0.08428
2	0.700	1.200	0.840	0.600	0.50400
3	2.100	0.200	0.420	0.100	0.04200
4	2.100	1/2 × 0.230	0.242	0.278	0.06728
Σ			2.104		0.69756

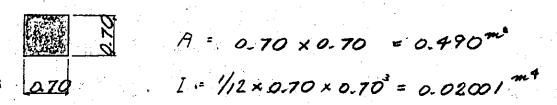
$$J = \frac{0.69756}{2.104} = 0.332^{m}$$

	b (m)	h (m)	A (m)	y (m)	Io (m4)	A. J. 2 (m+)	Io + A. J. = (m
<u> </u>	2-150	0 280	0-602	0/92	0.00393	· ·	
<u>②</u>	0.700	1.200	0.840	0.268	0.10080	0.06033	0.16113
	2-100	a 200	0.420	0.232	00140	202261	0.02401
<b>4</b> )	2.100	1/2 × 0230	0.242	0.054	0.0071	0.00071	0.00142
$\Sigma$			2.104		0.10884	0.10584	021268

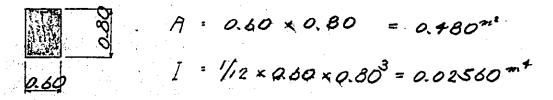
CYOSS-Sectional Area A = 2.104<sup>m</sup>

Moment of Inertia I = 221268<sup>m+</sup>

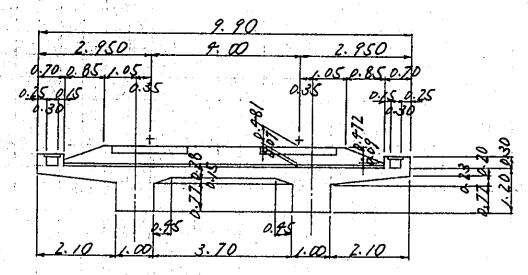
#### (2) Member (125. 2-6.3-7.428)



### (3) Member (5-66-7.7-8)



### 3. Reaction of T-beam Saperstructure



Length of Longitudinal beam

L = 9.980 m

Width of Cross beam

b = 0.70 m

1) Phistributed load

0.95 g/m × 10.00 × 2

= 9.00 1

2) Ballast (A)

1.9 1 × 0.481 × 6.80 × 10.00

= 62.15 "

3) Ballast (B)

1.9" x 0.772 x 0.85 x /2 x 10.00 x 2 = 7.62"

4) Ballast (C)

1.9" × 0.85 × 0.009 × 10.00 × 2 = 0.29"

15) Cross beam

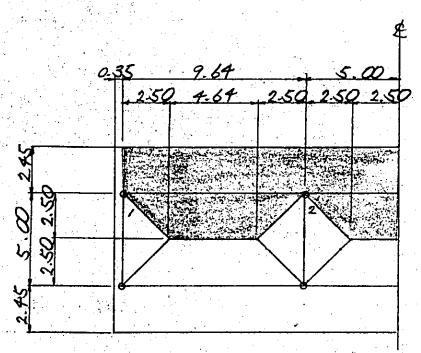
2.5 th × 0.70 × 0.92 × 3.70 × 2

= 11.91+

ERL = 237.55 t

Reaction Rd = 237.55 x 1/2 = 118.78 =

- (2) Calculation of loads
  - Dead Load
  - (1) Member (1~2~3~4)



(a) Disthibuted Load (A)

Weight of Slab, grading Concrete and Ballast

Slab

2.5 This × 0.28 = 0.700 This

grading concrete 2.35° , 0.07 = 0.165°

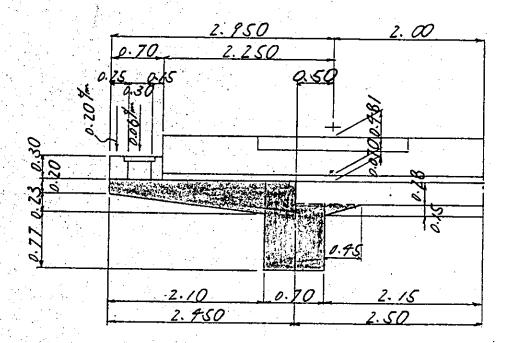
Ballast

1.9° × 0.481 = 0.914"

Wd = 1.78 t/m2

Wd1 = 1.78 /m x 2.50 = 4:45 t/m

### (b) Distributed load



1) handrail		=	0.20%
2) cunb	2.5 1/2 × 0.25 × 0.30		0.19
3) ballast stopper	2.5° × 0.15 × 0.30		0.11"
4) Duct Cover	2.5" × 0.05 × 0.30		0.09"
5) cable			0.06
6) Track Weight			0.95"
7) ballast	1.9 1/2 × 0.481 × 1.75		1.60"
8) grading concrete			0.29 *
9) Slab	2.5" × (0.20 + 0.43) × /2 × 2.10	5	1.65"
10) Slab Haunch	2.5" × 0.45 × 0.15 × 1/2	=	0.08
11) Lon gitudinal beam	2.5" × (1.20 × 0.70 - 0.28 × 0.35)		1.86
			1534

### (c) Concentrated load of elements acting at joint, P1 as Shown below

1) Distributed load (A)

= 5.56 +

2) Distributed load (B)

= 1.56 4

3) Distributed load (c)

= 1.61"

4) Slab Haunch

= 0.14"

5) Longitudinal beam Haunch

= 0.42 \*

6) Cross beam

= 6.47.

7) Beam for Bridge Support

1.47

8) Deficit of column Weight

= -2.09"

9) Deficit of Longitudinal beam Weight

= -0.56

### 10) Reaction of Theam Superstructure

118.90 1/2

= 59.45 t

P1 = 74.08 =

· soint PZ

1) Distributed load

1.78 / x 2.50 x 2.50 x 1/2 x 2

= //./3<sup>-±</sup>

2) Transversal beam

2.5 / × 0.70 × 0.82 × 2.15

= 3.09.

3) Slab Hounch

2.5° × 0.45 × 0.15 × 1/2 × 1.70 × 2

= 0.29.

4) Longitudinal beam haunch

2.5" × 1.20 × 0.40 × 1/2 × 0.70 × 2

= 0.89"

5) Deficit of column weight

2.5 \* × 0.70 × 0.70 × (0.332 - 0.28)

= 0.06"

6) Deficit of Longitudinal beam Weight

- (1.86 + 0.08) 1/m × 0.70

= -1.36"

P2 = 14.05 t

(d) moment at Joint Caused by beam of Bridge support
and T-beam bridge

 $M = 59.45 \times 0.71 + 2.5 \%^{3} \times (0.45 \times 0.70 \times 0.7$ 

(2) Member (5~6~7~8)

1) Distributed load

Bracing beam 2.5 1/2 × 0.60 × 0.80 = 1.20 1/2 Earth pressure 1.8° = 0.60 × 1.20 = 1.30°

W = 2.50 /m

Column Weight

8 = 2.5 1/m × 0.70 × 0.70 = 1.23 1/m

- 2. Train Load and Impact
  - (1) Train Load (Single track)

KS - 16

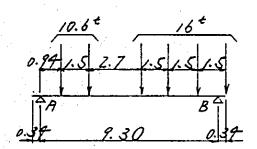
(a) Distributed load acting on Vigid-frame

Span l = 9.73 m

WMZ = 4.20 × 2 = 8.40"

WS = 5.60 × 2 = 11.20"

(b) Reaction of T-beam Superstructure



$$RB = \frac{1}{9.30} \times \{10.6 \times (0.94 + 2.44) + 16 \times (5.14 + 6.64 + 8.14 + 9.30)\} = 54.12^{4}$$

- (2) Impact coefficient
  - (a) Within rigid frame Section

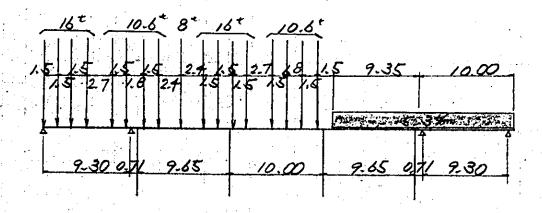
    l = 9.73 m io = 0.433

For the case of double track reduction is made follwed formula.

- Reduction of impact coefficient  $i = 0.433 \times (1 \frac{9.73}{7.00}) = 0.412$
- (b) Within T-beam bridge section  $l = 9.30^{m} io = 0.437$ Reduction of impact verificient  $i = 0.437 \times (1 \frac{9.30}{200}) = 0.417$
- (3) Train Load + Impact Load
  - (a) Within Figid frame Section  $WM1 = 10.20 \times (1 + 0.412) = 14.40\%$   $WM2 = 8.40 \times ( " ) = 11.86$   $WS = 11.20 \times ( " ) = 15.81$
  - (b) Realtion of T-beam bridge and joint moment

 $R(l+i) = 54.12 \times (1 + 0.417) = 76.69^{t}$  $M(l+i) = 76.69 \times 0.71 = 54.45^{th}$ 

### 3. Brake load



15% of the train load

- (a) Within elevated structute Section  $T = H_1 = (10.6 \times 6 + 8 + 16 \times 4 + 5.3^{44} \times 9.35)$   $\times 0.15 = 27.77^{t}$
- (b) Within T-beam bridge Section At Supporting Point A:  $T_2 = (16 \times 4 + 10.6 \times 2) \times 0.15 = 12.78^{t}$  $H_2 = 0.5 \cdot T_2 = 0.5 \times 12.78 = 6.39^{t}$

at Supporting Point B  $T_3 = 5.3^{t/m} \times 10.00 \times 0.15 = 7.95^{t}$   $H_3 = 0.5 \times 7.95 = 3.98^{t}$ 

(c) Total brake load acting within elevated structure

 $ZH = 27.77 + 6.39 + 3.98 = 38.14^{+}$  $ZH = 38.14 \times 1/2 = 19.07^{+}$ 

Brake load acting at

the Supporting point  $H = 19.07 \times 1/4 = 4.77^{t}$ 

4. Force of temperature change and/or Drying Contraction

Temperature rise

+10°C

· Temperature drop + Drying contraction
-15°C

5. Dead load + Seismic torce Kh = 0.1

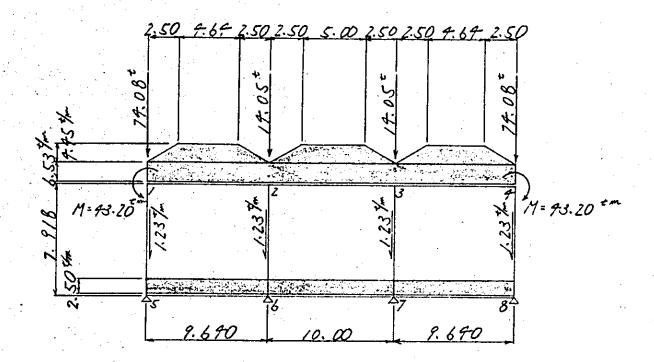
 $H = \{6.53^{tx} \times 29.28 + (£64 + 9.64) \times 1/2 \times 4.45^{tx} \times 2 + (5.00 + 10.00) \times 1/2 \times 4.45^{tx} + 74.08^{tx} \times 2 + 14.05^{tx} \times 2 + 1.25^{tx} \times 7.918 \times 1/2 \times 4\} \times 0.10$   $= 48.39^{t}$ 

Seismic force acting at

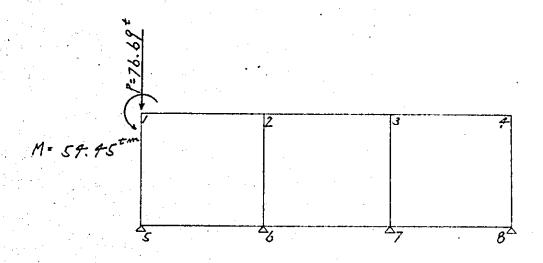
The Supporting point  $H = 48.39 \times ^{1/4} = 12.10^{t}$ 

# (3) Loading diagram

### 1. Case 1. Dead load

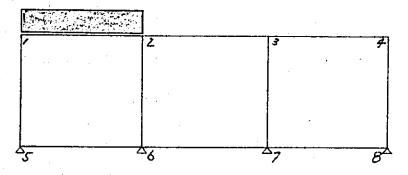


# 2. Case 2 Train load + Impact



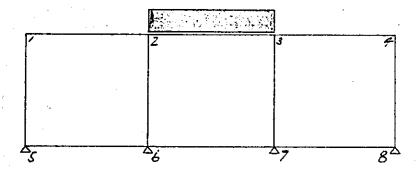
# 3. case 3. Train load + Impact

WMI = 14.40 1/2 WS = 15.81"



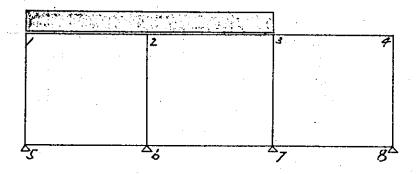
4. CWET. Train load + Impact

WM, = 14.40 5/m WS = 15.81"

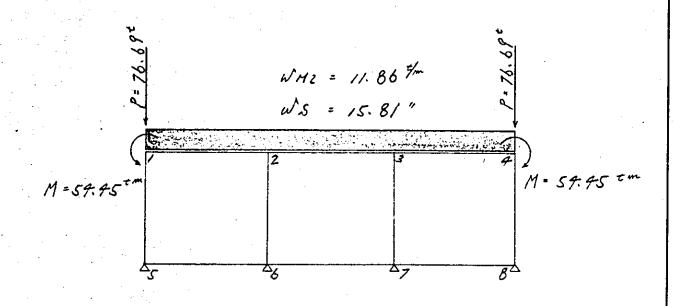


### 5. CASES Train load + Impact

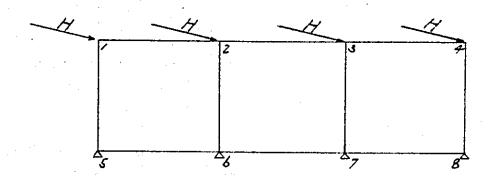
WHZ = 11.86 4m NS = 15.81"



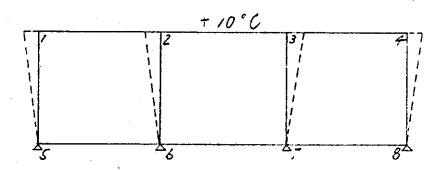
6. CORB Train load + Impact



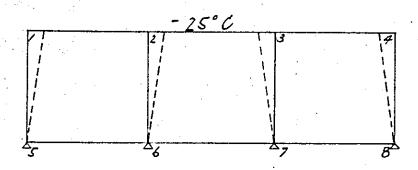
### 7. COSET Brake load



8. case 8 Temperature rise

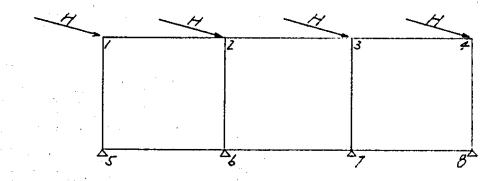


9. Case 9 Temperature drop + drying contraction



10. UDE 10 Seismic load

H = 12.10 +



# (4) Combination of loads 1. Basic load

		<u> </u>
Case No.	Kind of load	Loading Pattern
	Dead load	c y
2	Train load + Impact	G T T T T T T T T T T T T T T T T T T T
3	Train load + Impact.	
4	Train load + Impact	
5	Train Load + Impacx	
6	Train load + Impact	\$ <u></u>
7	BRAKE Load	
8	Temperature	+10°C
9	Temperature + Contraction	−25°C
10	Seismic load	

### 2. Combined loads

		<u> </u>
Case No.	combination of loads	~
11		1. 0000
12	1 + 2 + 3	1.000
/3	/ + 3	4
14	1 + 4	4
15	1 + 5	4
16	1 + 6 + 7	0.8696
17	1+6-7	<b>,</b>
18	/ + 8	•
19	1 + 9	>
20	1 + 10	0.6667
21	1-10	

### 3. Critical cases

No.1 case 11 Crack

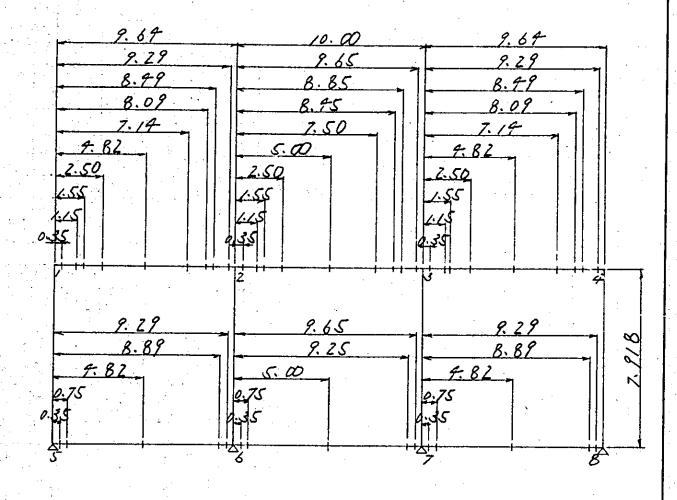
No.2 case 12 ~21 Synthetic

No.3 case 12 ~17 tooking

No.4 case 18 ~19

No.5 case 20 ~21

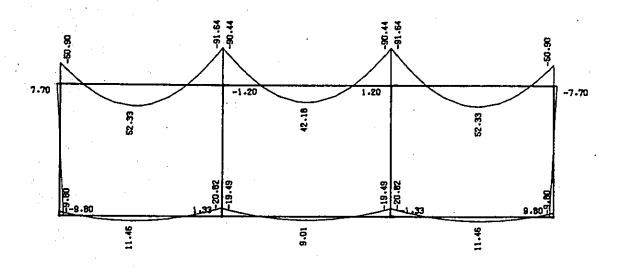
# 4. Point of computing stresses

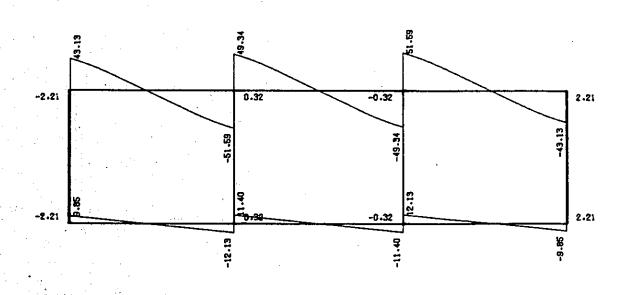


VIADUCT OF DOUBLE TRACK (3 \* 10 = 30) LI

CASE / (PEAD LOAD)

BENDING MOMENT

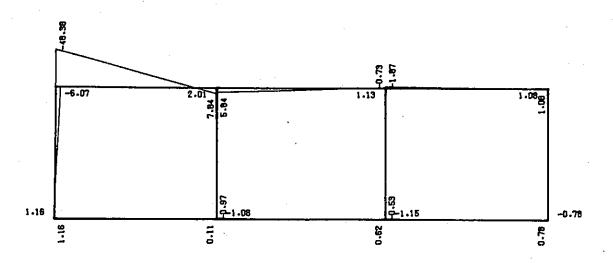


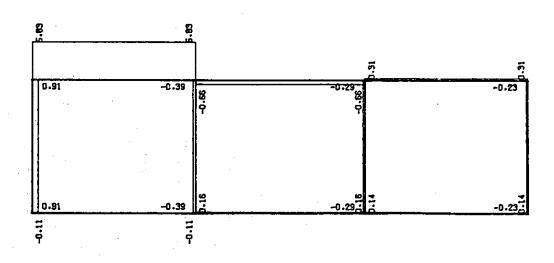


VIADUCT OF DOUBLE TRACK (3 \* 10 = 30) LI

CASE 2 (TRAIN LOAD + IMPACT LOAD 1)

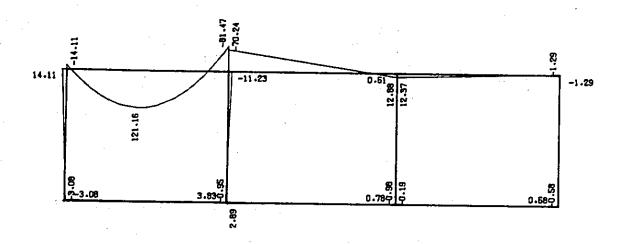
BENDING MOMENT

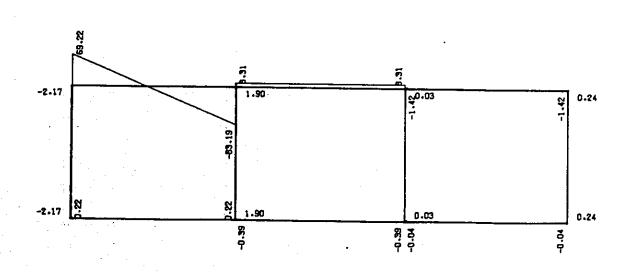




VIADUCT OF DOUBLE TRACK (3 \* 10 = 30) LI CASE 3 (TRAIN LOAD + IMPACT LOAD 2)

BENDING MOMENT

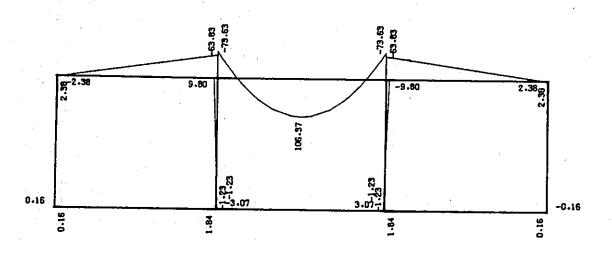


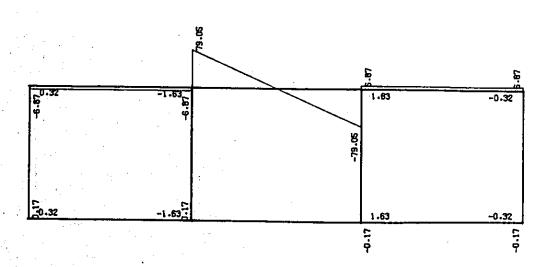


VIADUCT OF DOUBLE TRACK (3 \* 10 = 30) L1

CASE 4 (TRAIN LOAD + IMPACT LOADS)

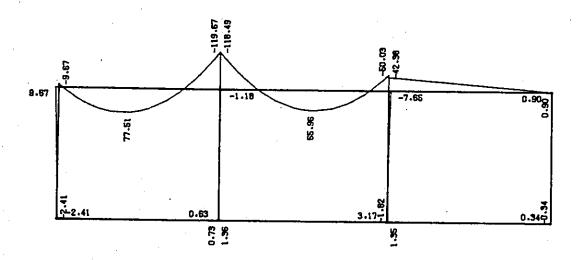
BENDING MOMENT

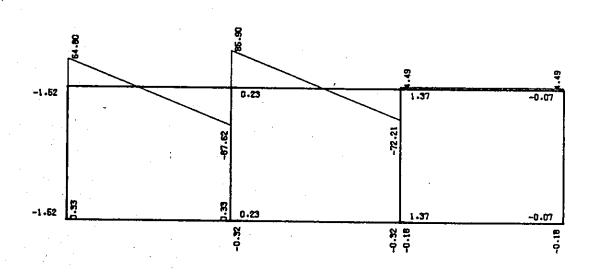




VIADUCT OF DOUBLE TRACK (3 \* 10 = 30) LI CASE S (TRAIN LOAD + IMPACT LOAD 4)

BENDING MOMENT

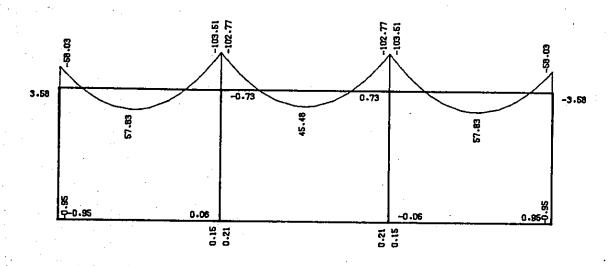


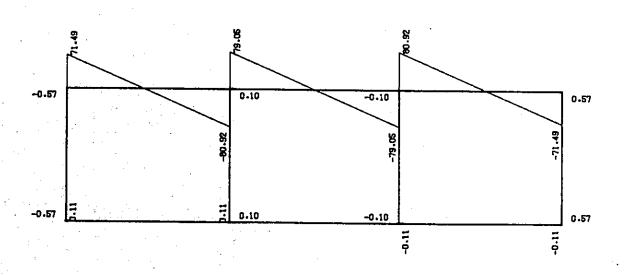


VIADUCT OF DOUBLE TRACK (3 \* 10 = 30) L1

CASE 6 (TRAIN LOAD + IMPACT LOADS)

BENDING MOMENT

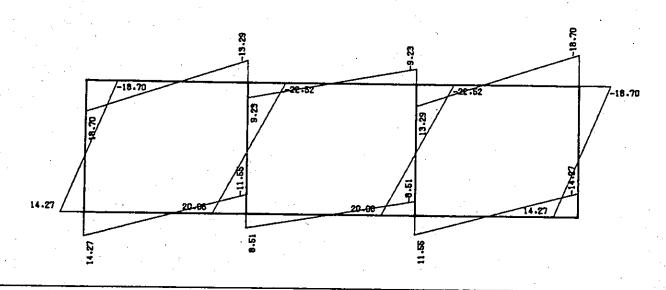


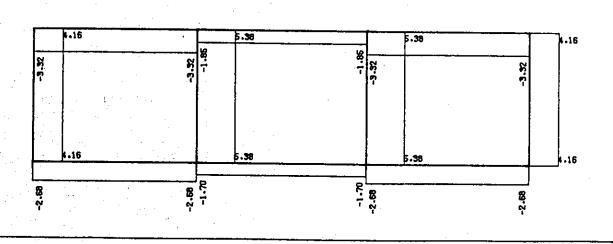


## VIADUCT OF DOUBLE TRACK (3 \* 10 = 30) LI

CASE 7 (BRAKE LOAD)

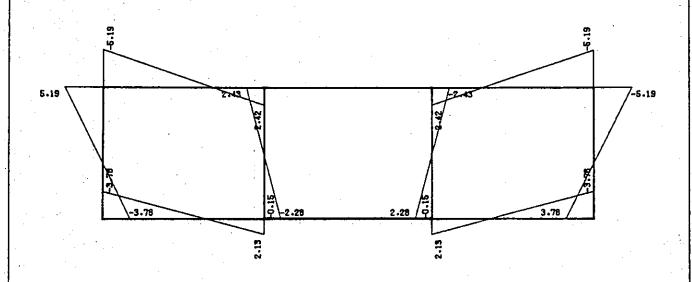
BENDING MOMENT

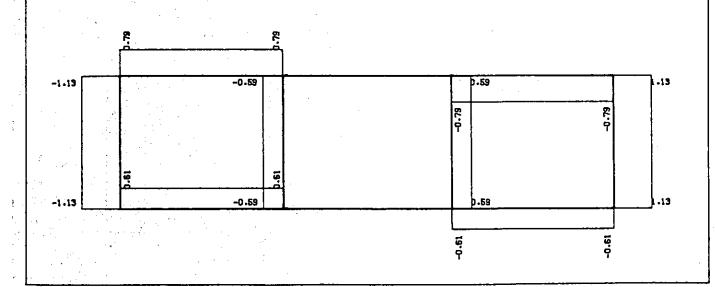




# VIADUCT OF DOUBLE TRACK (3 \* 10 = 30) LI

CASES (TEMPERERATURE RISE)
BENDING MOMENT

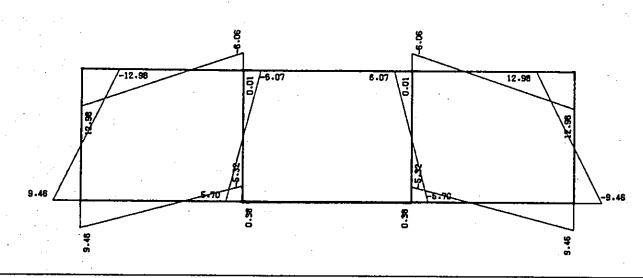


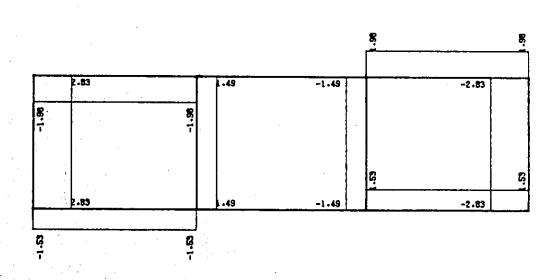


VIADUCT OF DOUBLE TRACK (3 \* 10 = 30) LI

CASE 9 (TEMPERATURE DROP + DRYING CONTRACTION)

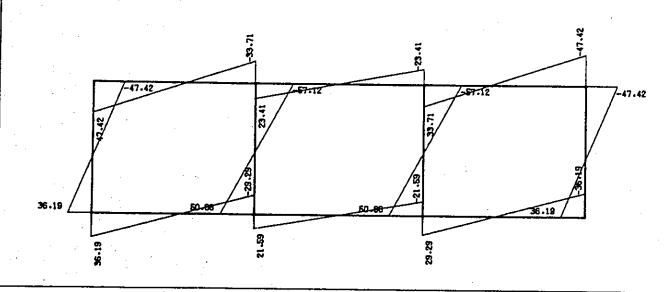
BENDING MOMENT

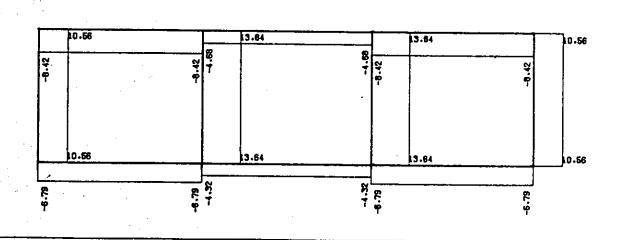




VIADUCT OF DOUBLE TRACK (3 \* 10 = 30) LI CASE 10 (SEISMIC LOAD)

BENDING MOMENT





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11   S1 K1	1.0000	3 1.0000	2 1.0000	2 1.0000	2 1.0000	3 1.0000	3 1.0000	2 1.0000	2 1.0000	2 1.0000	0000
LOAD SS N	. ~		1.0000		_	- 8696	s	.a	۰.		

	-		MOMENT MAXINUM	<b>.</b>				ì			4-
								É	מבמו הומות	5	
		-CASE-		2	N	<b>i</b>		-CASE-	·	3	
= = HENBER	7.7	ं <b>।</b> जून	2 ) 6 = =			= = MEMBER	ER 1 (	1 - 2	# 15 C		
ITAN	0.00	( 11 )	83	43.135	굯	ITAN	0.000	( 11 )	-50, 897	m	
<b>1</b>	35	7	36.21	40.740	2.21		.35	(11)	-38.213		Ž
N I	1.150		-6.061	10 4 4 4 CO	-2.210	~	1.150	(11)		•	-2.210
7	٠. د	•	10.	50.875	77.7		. 55	ᅻ	7.013	<u>~</u>	2
<b>3</b> W	בינ	7	31.09	742.12	7	. <del></del> (	2	( 11 )	31.898	3	-21
Λ,	Š.		51.64	ارا	7.2	ا	• 82	( 11 )	51.642	Ň	-21
ا م	5	1	12.28	52	12.2	٠	. 14	t 11 )	12.288	0	12.
	S.		-26.62	<b>.</b>	77.7	~	. 09	(11)	-20.627	Ņ	7
<b>*</b> 0 6		٠,	-37.08	ų.	12-2	•	6.	(11)	-37,083	0	2
-	Ţ.		· ·	77.00	7:		62.	( 11 )	-73.996	6	-21
X 4 10	ō	~	11.65	ż	7	NALO	40.	11	-91,639	40	2
HAX	4.284	( 11 )	52.331	1,654	-2.210	HAX	4.284	( 11 )	52,331	1.654	-2.210
	~		H 11			30737	7 0			7.5	
1	,		•				2	73 I	11		
ITAN	0.000		-90.43	2	8.9	I TAN	0	(11)	_	49,338	- 5
	.350	-	-73,58	46.943	89		35	-			
~	. 15	17	-38.46	40.651	5	~	.15	-		'n	
מי	1.550	( 11	1 -22.912	37.078	-1.890	m	1.550	(11)	-22, 912	37.078	- 00
•	55	11	7.86	27.450	1.89	•	•50	-	-	27.450	
<b>ن</b>	8	11	42.17	000	83	in.	• 00	-1		000	20
9	.50	-	7.86	2	6	9	• 50	~1	-	-27.450	8
~		-	22.91	37	6	~	•45	( 11 )	-	-37.078	٠.
eco (	. 85	-	8.45	ġ,	8	<b>~</b>	. 65	(11)	•	0.55	٠.
	65	( 11 )	73.58	٥	5		•65	( 11 )	m	-46.943	
スないつ	8	~	90.43	•	89	CIAN	0	( 11 )	* ·	9.33	-1.898
HAX	5.000	( 11 )	42.179	000-	-1.890	HAX	5.000	( 111 )	42.179	000	-1.690
= = MEMBER	E E	i Po	= = 9 ( 4		•	= = MEMBE	ER 3 (	3 1 19	" " "		
ITAN	000.0	-	91.63	51.587	-2.210	ITAN	0.000	( 111 )		51.587	-21
<b>~</b>	.35	+1	73,99	ġ.	7	<b></b> 1	S	(11)		49.193	21
2	LO.	( 11 )	í.	42.901	7	~		(11)	-37.083	42.901	N
m	. 55	#	-20.62	÷	2.21	m	. 55	( 11 )	•	39.328	2.21
<b>.</b>	.50	=	12.28	ດໍ	2.21	.3	.50	-	•	29.700	2.21
5	. 82	11	Š		7	2	82	-	•	4.226	.21
۰	:	=======================================	31,89	2	7	Φ.	4.	( 11 )	•	-21.247	2.21
~	-03	=======================================	7.01		2.21	~	• 03	(11)	•	-30.875	2.21
æ	64.	11	9	-34.448	걾	<b>8</b> 0	£	( 11 )	•	-34.448	2.21
	. 29	┥,	36.21		굺		•29	(11)	•	-40.740	2.21
NATO	9.	++	£.		2	LTAN	3	11 )	-	3.13	.21
r z x	5, 356	(11)	52,331	-1.654	-2.210	HAX	5,356	( 11 )	52,338	-1.554	-2.218
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		<b>2.</b>	MOMENT HAXIHUM	£.	:	· .		•	MOMENT MINIMUM	:: :::	•	
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ITAN	0.000	( 11 )	-9.803	•	\$							
	350		964.9-	9.049	0.00	24.1	0.000	::	-9.803	9.847	<b>.</b>	
2	.750		:	:	t i		0.00	::	00.4.70	D + D + S	6	
	4.820	( 11 )	11.173			. M		::	-5.059	8.137	<b>.</b>	
	8.890	[ ]	-12,363	!			• •	1	11.173	21.14		
<b>5</b>	9.230	(11)	-16.714				• •	1 +	12.353	-10.423	-	
JTAN	0.79.6	- 11	-20.821	:		JIAN	9.540	11	-20.021	-12.133	56	
HAX	4.284	, 11	11.458	.078	0.000	HAX	4.284	11	11.458	.078	0.00	:
E = HEMBER	in.	9	¥ = 9 ( ¥	;	?	= HEHBER	ER 5 (	9	7 ) 6 = =			:
TTAN	טיי טיי	-	100,000		: C							:
				10.602	0.00	₫ .	200.0		19,490	11.400	0.000	
<b>~</b> ∙	.750	(11)	-11.581	9.690	0.000		1750		-11,581	209.01	2000	
m	5.000	( 11 )	ᇊ	0.000	0.000	ı m	5.000	1 = 1	9.010	0.000		
J (	9.250	1	-11.581	-9.690	0.000	•	9-250	11	-11.581	-9.690		:
v :	9.650			-10.602	0.00	<u>ι</u> Λ	9.650	11 )	-15.640	. 0	0.00	
	10.001		19.490	-11.400	0.000	Neto	10.000	11 )	-19.490	•	0.000	
MAX	5.000	(-11)	9.010	0.000	0.000	HAX	5.000	11 1	9.010	0.000	0.00	:
# # NE HBER	9	- ~	8 ) (5 11			BH HE HE	ER 6 (	~	11 11 11 11 11 11 11 11 11 11 11 11 11	: : :	!	:
ITAN	0.000	( 11 )	-20.821		0.000	ITAN	0000	-	-20, 821	11 64	•	
(	350	( 11 )	-16.714	11, 335	0.00	-	.350	( 11 )	-16.714	11.335	•	
N F			-12.363		0.000	2	.750	11 1	-12,363	10.423		: : : : : : : : : : : : : : : : : : : :
) • <b>3</b>			11.17.5	!		m .	4.820	11:	11.173	1.143	_	
. PV	9.290	11	-6.496			FU	0.00		650-5-	å,	•	
NATO	9.640	( 11 )	-9.803		0.000	UTAN	9.640	13	-9.803	7. 10° 84° 4	0.00	
MAX	5.356	( 11 )	11.458	078	0.000	MAX	5.356	( 11 )	11.458	078	0.000	
# = HEHBER	2 (	ा च	5 ) C s =			. = HE HBE	.R 7 C	1	5 1 C = x		‡ .	
I TAN JI AN	7.918	111	7.697	-2.210	-117.215	I TAN JAN	0.000	111	7.697	-2.210	-117.215	
T T NEMBER	<b>6</b> 0	I P/	= # 3 C 9			BH3K = =	. 9 2	2	6 ) C = =	•		
LIAN	0.000	(11)	-1.203	.320	-114.975	I TAN	0.000		-1.203	320	-114.975	Vo.
							•	•	٠	200+	-17.471-	•

		2	MOMENT MAXIMUM	I.					HOH	HOMENT MINIMUM	5	
		LCASE	N3N	2			!		-CASE-	H		· N
= HEMBER 9 ( 3 -	6	~	# # J (			11 11	HEHBER	6	= HEHBER 9 ( 3+ 7 ) C ==	H ()		<b>!</b> .
ITAN	0.000	0.000 (11) 7.918 (11)	1.203	320	320 -114.975	15	ITAN	0.000	0.000 ( 11 ) 1.203 7.918 ( 11 ) -1.331	1.203		320 -114.97 320 -124.71
# = HEMBER 10 ( .4 =	10 (	-	1 U G			#. . II	MEMBER	10 (	= MEMBER 10 ( 4 - 8 ) C = =	= = 0		
ITAN JIAN	0.000 7.916	0.000 (11) 7.918 (11)	-7.697 9.803	2.210 2.210	2.210 -117.215 2.210 -126.954	117	ITAN	0.000	0.000 (11) -7.697	-7.697 9.803	2.210	2.210 -117.21

,	, <b>,</b>									; !	:	:	•		;		•			:	:	:											÷				
	N		21	2.21	2.21	2.21	;;	7.7	2.21	17	-2.210	3	***	1.69	) C	2.89	1.89	1.89	1.69	. 89	-1.690	;	.21	2.21	2.51	-2.210	2.21	2.2	2.5	2.23	2.21		. 00	<u>=</u> ;	0.00		3
	0			3	2	۵.	5.5	10	~	2	-49.193	•	- 1	, . ,		7.07	7.45	8:	Λ P-	65	-40.337	1	1.58	9.19	2.90 2.90	29.700	4.22	1. 24.	2	7.	3,13			3:		0.42	,,
-	X	# # D	-50.897	-36.213	-6.061	7.013	7	12,288	8		-73.996		, !	30. TV	9	22.91	89	- :	22.91	38.46	-90.436	H 9 C	91.63	73,99	3	12.268	\$	6	2 0	7	5	: : : : : : : : : : : : : : : : : : :	9	5 :	10.00	9	
	-CASE	1 - 2	( 11 )	( 11 )	<b>.</b>	٠,٠			( 11 )	(11)				1:			111	~	717	711			( 11 )	(11)		11	(11)	- : - :		( 11 )	( 11 )	9 - 5	( 11 )	- - -	 : ::		•
•		R 1 (	0	.35	150	ָהָ הַיּ			•00	6	0.5.9	^	, '6		15	55	- 50	9 5		· •	10.000	υ 10 10 10 10 10 10 10 10 10 10 10 10 10	0	5.	ָנֻ בַּרָ	2.500	• 62	7 5		29	.6		0	5	1.820	9	
•	į	BHJW = =	ITAN	T	N٢		r tr	: •••	<b>~</b>	<b>40</b> (	NATI	HENBE		4	; 4 N	מיו	ا جي	6	۰ ۸	<b>40</b> (	JIAN	E # MENBER	ITAN	¢	is m	<b>.</b>	<b>.</b>	+ ص	- ec	· \$	JAN	= = MEMBES	ITAN	(	u m	<del>.</del>	۲
•	1112111		2	2.21	2:	7.0	7.5	2.21	2.21	22	-2.210	i	- 6	- 0	1.89	1.89	1:89		1.89		1.89		2	Z:	į s	-2.210	2	2 5	72	2	21		.00	5		00.	•
	0	ř	3.13	72.0		٥,٥	22	9.70	9.32	8°.9	-51,587	7							-37.076	-40.651	-49.337		1.58		25.0	29,700	4. 22	21.24		10.74	~		-		-1.143		٠
		H 12 -	•	ų,	٦,	- «	9	Ş	• 62	37.0	-91.639	" و		5 ~	E	Ň	- 8e	7	. 22	8	; ;	11 11	91.63		2	12.288	1.6	- ·	6.06	N	0.09	# # # # # # # # # # # # # # # # # # #	.80	9,	1.17		
	CASE -	1 = 2			н.	· -	: =	-	(11)				, , , , , , , , , , , , , , , , , , ,	: :	٠-	=		Ξ:			(11)	i J N		٦.	: :	111	= :	Ξ:	=======================================	**	( 11 )	9	(11)	= :		( 11 )	
		1 -	0.000	350	÷.		2	=	6	5	9.6	~		2 6	5	55	200	֓֞֜֜֜֜֜֝֓֓֓֓֜֜֜֓֓֓֓֓֜֜֜֓֓֓֓֡֓֜֜֜֡֓֓֓֓֡֓֜֜֜֡֓֡֓֡֓֡֓֡֓֜֜֡֡֡֡֡֡	4	8.850	00	n w	0	2.	. 55	2.500	- 8.2	. e	, ,	29	9.	ت ع		M 1	200	, 0	`
	<b>!</b>	E E HEMBE	ITAN		NJ P		r M	: 		<b>*</b> 0 C	JEAN	HE MBE			1 10	m	ي ج	۸ م	o <b>~</b>	<b>40</b> (	JI AN	= = HENBE	ITAN	f	V F	, ) . <del></del>	rv :	4 0	- ec	•	71 AX	38K3H = ±	IIAN	<b>-</b> → (	u m	<b>.</b>	,

SHEAR MINIMUM

		v	SHEAR MAXIPUN			1	•	SHEAR MINIMUN	HUH	
		-CASE-	H	8	N			-CASEH		N
= HENBER	2 (	. •	== 9 ( 2		1	= = NEMBER	BER 5 (	= 9 1 2 - 9		
ITAN	0.00	(11)	-19-490	11.400	0.000	ITAN	00000	(11) -19.49[	7	
	.350	( 11 )	-15.640	10.602	0.000	<b>T</b>	350		10	6
~ ~	5.000		-11.581	9.698	000-0	<b>N</b> P	.750	~		0.00
3.3	9.250		-11.581	9.69	000.0	7 4	0.00	11 ) 3.010		<b>-</b>
Ē	9,650	( 11 )	-15.640	-10.602	00000	, IQ	9.650	. ~	1	
JTAN	10.000	( 11 )	-19-490	-11.400	0000	JTAN	10.000	_	Ŧ	
± = MEMBER	9	- ~	1 1 2 6 8	•	: :	= = MEMBER	BER 6 (	7 - 8 ) 6 = =		
ITAN	000 0	( 11 )	-20.821	12,133	000-0	ITAN	0.000	(11) -20.821	12.	0.00
-	.350	( 11 )	-16.714	11, 335	0.000	-	TU.		11.	0.00
~	.750	( 11 )	-12.363	10,423	0.000	<b>~</b> :	. 750	(11) -12,363	10.	0.00
m	4.820		11.173	1.163	000	<b></b>	4.820	( 11 ) 11.173	÷	0.000
<b>.</b> 7 U	8.890	~ ;	-3.059	-8.137	0000	a≱ ti	80 0 0 0 0 0 0 0	~ -	ရုံ (	0.00
JIAN	9.640		-9.803	-9.847	000.0	JTAN	9.640	6	748.6-	000.0
= = MEMBER	7 (	<b>1</b> ₩	5 ) C = =			= 1518	BER 7 (	1-5)0==		
MAL	0.00	111	7,697	21	-117,215	ITAN	.00	(11) 7,697	-2.2	-117,215
UTAN	7.918	( 11 )	-9.803	-2.210	-126,954	JIAN	7.918	• • .	-2.210	-126.954
= = MEMBER	80	- 2	= = 0 ( 9		•	至安 中心	HBER 8 (	2 - 616 ==		
ITAN	0.000	(11)	-1.203	. 320	-114.975	ITAN	000-0	•	.320	-114.975
N	6.918	( 11 )	1.331	. 320	-124.714	JIAN	91	(11) 1.331	• 32	-124,714
= = MEMBER	) 6	ا س	= = 3 ( L		:	= = HENBER	BER 9 (	3-706==		
ITAN	0. 660	111	1.203	320	-114.975	LIAN	0.000	(11) 1,203 (11) -1,331	320	-114.975
= = MEMBER		1				= = MEHBER	10	) ( 8 -		
ITAN	0.00	( 11 )	-7.697	2.210	-117.215	ITAN	0.00.0	(11) -7.697	2.210	-117.215
NATO	7. 918	(11)	9.803	ಸ	-126,954	JIAN	7.918	(11) 9,803		-126.954

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-N			<b>.</b>	<b>.</b>	<b>.</b>	• ,	ď	<b>.</b>	5						0.0	000.0			-117.215		-114.9	-124.714		1111	-124.714		
0	•			204-01	3.030	000.0	-9.690	-10.602			12,133	11,335	10,423	1.143	-8,137	-9.049	, , , ,		-2.210		.320	.320		127°=	320		
H	II II	149		3*0*CT	100.17	0 TO	-11.561	19.66	•	,   	-20:821	-16.714	-12,363	11-173	040.5	-9.803	ار ار ن		-9.803	# # O	-1.203	1.331	# # **	1.283		1	اا اا ج
-CASE	9 - 7					1					~	~ ·		-,-			1 - 5)		111	6 9 - 2	(11)	( 11 )	3 - 7)	(11)	11)	1	+
	, . <b>S</b>	0.00	350	750	5.000	200	7	10.800	æ		000.0	350	200		0.00	9-640	~	0.000	7.918	₩	0.00	7.918	) 6	0.000	7.918	10	,
	H MENBER	NATI	<del>-</del> 4	7 2	ı iya		, r	JTAN	" " NEMBER		Z 4 -	- c	v m	7 -		JIAN	= = MEMBER	TAN	JTAN	= = NEMBER	ITAN	NaTo	= MEMBER	ITAN	LIAN	= = HEMBER	
N	•	: ``		0.000				0.00		6	200			0000	0.000	0.000	. •	-117,215	-126,954		-114.975	-124.714		-114.975	-124.714		
0		11.400	10.602	069 6	0.00	-9.690	-10.602	ä		12 112	11.100	10.423	1,143	-8.137	-9.049	-9.847		-2.210	-2.210		.320	125.		320	- 320		
·	11 3 C	-19.490	-15.640	-11.581	9.010	-11.581	-15.640	-19.490		-20.821	-16.714	-12,363	11.173	-3.059	-6.496	-9.803	= = 0 (	69	-9.803	= = 0	-1.203	1.001	# # O	-20	-1.331	= = 0	
-CASE-	2 - 9	( 11 )		( TT )	(11)	_	_	_	7 - 8	( 11 )	( 11 )	( 11 )	( 11 )	(11)	(11)	( 11 )	1 - 5	( 11 )	( 11 )	2 - 6	(11)	***	3 - 7	(11)		89 1 37	
	2 (	0.000	065.	.750	5.000	9.250	9.650	10.000	9	0.000	350	.750	4.820	8.890	9.290	9.640	۷ (	0.00	7.918	9	0.000		) 6	0.00°	7.518	10 (	
	= MEHBER		r+ (		:		ľ	LEAN	= = MENBER	ITAN	-	~				OT AN	= HEMBER	ITAN		= = MEMBER	ITAN		# # MEMBER	NATI		= = MEMBER	

MOMENT MINIMUM

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	1	-CASE	3		;			•			
					1	i		-CASE	X	0	N
# # MENBER	-	. T	11 11 11			= MEMBER	.R 1 C	-	2 ) 6 = =		
ITAN	•		7	23, 147	-2.500						,
<b>–</b>	350	( 20 )	5.511	21.550	-2.500	-		7:	-113	118-183	4
	. 15	-	80.	65.483	-4-3B2	• •	•	ì	-81.	95.668	2
•	. 550	-	72.35	75.586	-4.382	. P		7	-53.	28.578	3
	• 500	-	2	50.939	782	•		5	-16.	26.195	3
	82	_	71.12	-11.213	14. 482	<b>.</b>	<b>-</b> (		3.6	19.777	44.
	77.	( 13 )	76.80	-71.166	7 7 7	,	2:	<b>.</b>	502	-11,694	. 33
	60	21	20,	-20.509	705.1		- 6	<b>.</b>	- 4 to	-36.568	-89
	4.9	( 21 )	A. 70	-22 001	K 1 1		֓֞֓֓֓֓֓֓֓֓֓֓֓֓֓֓֓֓֓֓֓֓֓֓֓֓֓֓֓֓֓֓֓֓֓֓֓	3	-73.B	-46.196	.83
	2			-27 485		10 (	5		-93.016	-49.769	.89
	4		- 20 er	007.00			• 29	( 12	-170.391	-131.277	73
		;	9	701.07	77.1	NATO	3	( 15 )	-211.309	-139,206	-3.735
MAX	4,284	( 13 )	173,491	3,134	-4.382	HAX	4.284	( 14 )	25.282	-5.214	-1.890
= = MEMBER	2 (	2 - 3	H 50	٠		HENDY II	69	•	1		1
									9		
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	, c	٠,	3,5	•	5.19	•	•50		-41.58	^	~
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	٠.	Ŧ .		•	3.19	9	.59		-2,55	-1	-
	, ,	- 6	52.7	91.62	3,19	~			-47.29	81.2	
	ю,	7	13.62	23.98	1.26	€0	. 85		-76.52	80.8	
ַר בּ	യ	~	4.54		1.26	5	59	16.		707	,,
-	0	( 51 )	• • 68	-29.772	-1.260	LTAN	10.000	15.	-175.0404	F36 - 211-	10.04
								:		74674	•
χ α χ	5.000	( 14 )	148,550	000	-3.195	HAX	5.556	( 13 )	16.427	2.212	-2.159
= #EHBER	m	3 1 2	== 9 (			= = MEHBER	e e e	ı m	II		
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	•					<b>3</b> 1	.50	- 14	~	36.568	÷
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	4 6	• 6		<b>:</b>	7 (	ּפֿ	<b>.</b>	( 50 )	~	-19.777	447
	<b>,</b>	1 6		٠,	3.0	~	-03	( 02 )	₹.	-26.195	
		7 ;	47747	٠.	-2.500	<b>00</b>	64.	< 50 >	$\sim$	-28.578	7445
	Ju	7.0		∴,	3		• 29	(16)	9	-95,668	-1,891
Z	0	v		ri.	20	JTAN	9	( 16 )	-110.978	-102,562	-1.891
KAX 5	.356	( 17 )	93,690	-1.814	-2.947	7	~		•		
						< <u>*</u>	Γ.	**	292.62	5.214	-1.890

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[17]	4.820	3	N	696	2	<b>,</b> r		3	-22.172	9.954	•	
	8.890	2		2 120	, c	, ) ,	700	7	2.149	3.767	•	
	9.29		•	800	9 6		0.00 m	20 1	-24.375	1.47		
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τ	2.0			-3.550	00	NATO	9.640	( 50 )	-33.411	•	000-0	
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u ir	5.000	7 2 7	210.4	2,002	0.00	۱ ہ	. 750	72	-19.955	9,339	•	
ن (	2000	3 6		0 0	3 (	·	2.000	( 51 )	6.007	2.879		
ru	7.670	7.	ב מ	20 0	200	-3* I	9.250	( 20 )	-19.955	-9.339	-	
	0000			2	9	-	9.650	<b>6</b> 02 )	-23.812	2,6.6-		
24 - 7	10.00		D	N	С С	UTAN	10.000	( 50 )	-27,387	-10.479	000.0	
HAX	5. 000	( 13 )	9.965	-, 386	0.000	H A K	5.000	( 21 )	6.007	2.879	0.00	
= = NEMBER	8 6 C	7 - 8	" 9 0			= = MEMBE	.R 6 (	7 -	3 6 = #			
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ī,	9.290	{ 21 }	18.213	-1.504	0000	r un			2	ř	•	
JTAN	9.640		7.59	-2.036	8	JTAN	9.640	( 50 )	-30.665	-11.094	0.00	
K A X	8.569	( 23 )	19.902	408	0.00.0	MAX	4.284	( 22 )	6.697	6.105	0.000	
= MEMBES	2	1 - 5	# = 0 (			E = MEMBE	7 (	+	2 0 6			
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	<b>.</b> c	9	ا د د	,		H HEMBE	_ ∞ ∝	2 :	# D C 9			
NATI	00.00	( 21 )	5	-8.880	-74.164	ITAN	0.00	( 20 )	-38.883	9,307	-79,143	
24 7	5		4.81	9, 307	-85.636	UTAN		( 51 )	-33,035	-8.880	-80.657	

MOKENT MINIMUM

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1CASEM2N		8.880 -74.154 -9.307 -85.636		8-514 -83.758
3	:	6.680	•	8.514
H	# # D	-37.279	H .	-36.749
-CASE	3 - 7	0.000 ( 20 ) -37.279 7.918 ( 21 ) -34.810	60 1	0.000 ( 20 ) -36.749
	6	0.000	10 (	0.00.0
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N		-9.307 -79.143 8.880 -80.657	:	-72.536
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	11 U	38.883	11 11	26.486 30.665
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r	55:	N (	75.69	10.25	ž		35				2.20	
u m	1.550	7 -	410	91.315	-3.469	<b>~</b>	1.150	( 50 )	21.124	. 6	2.5	
) <u>-</u>	.50	(12)	1 M	: ^	,	M3 &	un (			97	2.50	,
S.	.82	12	35		, ,	<b>.</b>	, ,		38.	55	2.50	
s	.14	2	'n	. 5	3	n 4				3	3.73	
~	65•	21	2	20.60	3	) <b>. ^~</b>			ָ מַמָּ	-77.79	3.73	
∞ :	6.	N.	8.70	22,99	3	· «	Ì			3 6	5 5	
•	53	2	82	2	3	• • • •	.29		70.3	131.27	3.5	
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	15	-	88.73	80	8	· ~	15	200	, v	•	2.50	
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	1 a	3 8	מיייי	;;	9	~	45	( 14 )	2	91.6	3, 19	
- o	9.550	3.5	-13-628	128.436	-1.250	æ (	8.850	- 1 11			-3.195	
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ית: יים	5	( 21 )	٠. ح	55	2.50		.29	( 16 )	86	1.0	3 =	
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5	. 29	ď	6.801	• 02	2	Ľ	29	( 16 )	23.66			
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 MAXIMUM
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3.560 3.028 2.420 -3.767 -9.954 -10.562 -6.514 -9,307 37.279 38.883 21 0.000 350 750 4.820 8.890 9.290 0.000 0.000 7.918 0.000 8 6 0.000 10 -72.536 12.780 11.478 5.291 -.896 -1.504 5.567 9,307 8.880 8.514 24.375 -24.375 -24.375 9.749 18.693 18.213 17.594 -38.883 34.810 ## ပ ں 0.000 .350 .750 4.820 8.890 9.290 0.000 **-**60 0.000 7.518 **)** 000 918 0.000 10 MEMBER

AXIAL HINIMUM

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	:	,	74.382	) F	-4.382	8	-4.382	38	20	38	8	-4.382			7,	7	,,	;		,,,	10.190	; ,	•	-3,195		•	2.9	2.9	5.5	6.	5.0	2.9	<u>ت</u> ر	,,	,,	7.6.2-	1		5	2 6	3 6	9 6		000.0	00
0			164-423	85.483	75,586	50.939	-11.213	-73,366	-98.013	-107.910	-126.850	-134.778	•		? :		7 0	7 4	ם יו	20			120.1	128.			-	Cuι	Λ.	•	-	Δο	00	40	и с	-96.792			9,950	4	'n	7.6		-11,222	2.02
H	9 (	-65.040	-29,360	1088	354	838	71.125	807	- 981	40.219	29.608	173.107	9 (	1	3 6	, A.	,	JN		, ,			22.8	164.	# # 9		701	-146.695	974.01		10.00	20.05	26 72	27.4.6	54 172	-78.463	1	•	•	aç	CE - 7-	9.798	-13,286	-17.592	-21.660
-CASE-	1 - 2	( 13 )	13	( 13 )	m			,	( 13 )	C 22	13	113 7	2 - 3	1	11.		1 1 1	1 1	71	, c	1 11	( 14 )	14 )	( 14 )	- F						- 7.			17.		( 17 )	, ,		(12)	( 12 )	( 12 )	1 12	( 12 )	( 25 )	( 112 )
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	= MEMBER	ITAN	<b>+</b>	<b>ω</b> ,	m		D, F	۰,		<b>20</b> G	TAN	<u>.</u>	= HEMBE	ITAN		~	כיו	: <b>.</b>	<u>ب</u>	٠	~	€0	er	UTAN	HENBE	TAN	•	40	, 1*		•	ک ۱	~	. ac	o	JTAN	* * MEMBE		ITAN		~	רט	<b>.</b>	S.	JTAN
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	, :	35.792	33, 710	26.239	107 27	10. 70 P	, .	ı,	120.02		-46.578			.90	. 82	35,350	ž	. 87	• 00	3.87	32.24	35,35	40.82	2.90		46.578	61.	39.024	5.91	7.56	5.39	6.75	5.13	23	33.71	5.79			96	15	2	1.03	0.31	-11.222	Z• t 2
<del>H</del>	II	-32.973	N	, ,	•			ď			-84.951		H H U	.9	3.98	33.44	9.91	5.8	89	6.8	5	3 44	96,	8.63	9 (	-84.951	9.01	-35.543	54	9.70	7.91	34.731	4.72	4.04	.83	2.97	= = 9 (		1.73	ñ.	4.90	9.7	3.28	-17.592	90.
-CASE-	1 - 2	( 19 )	517	7 0	٠.۵	5	-	19					۳ ۱ ۷	7	13	( 19 )	13	13	5	5	61	5	. 61	( 61 )	3 1 5		σ	( 19 )	19	19	19	6	13	19	_	-	5 - 6		15		7	15	21	75.	71.
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O. NATI	000	(12)	-17.576	11,173	0.00	TTAN	•			1	1
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<b>~</b> ↓	.750	( 75 )	-9.837	o,	000.0	~	.750	(12.)	-9-837	9.453	
	000	127	9.791	227	0.000	כיו	5.040	( 12 )	9.791	- 227	
	052*	( 75 )	-11.764		0.00	<b>.</b>	9.250	( 21 )	-11.764	-9.917	0.000
JIAN 10	0.00	121	-19.843	-11.627	0.000	5 JAN	10.000	215	-15, 913	-10.829	0.000
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ITAN 0		( 12 )	-21.546	12.228	ט-ט-ט	TAN	6		ċ		
-	350	(12)	-17.406	11.430	0.000			77	-71.546	12.228	0000
	.750	(12)	-13.017	10.518	0.00	• ~	756	77	-13.012	17.650	0000
	• 820	( 12 )	10.909	1.239	0.000	m	4.820	(12)	10.909	0 0 0 0	0000
<b>3</b> □	8.890	( 15 )	-2.933	-8.841	000.0	<b>.</b>	8.890	( 75 )	-2, 933	-8.04	
ហៈ	- 590	( 15 )	-6.332	-8.953	0.00	Lín	062.6	( 21 )	-6,332	-8-953	
	079	( 12 )	•	-9.751	0.000	JIAN	9.640	( 21 )	-9.605	-9.751	0.000
= = MEMBER	) (	1	= =, 0 ( 4			= = MEHBER	~		= = 0 ( S		
IIAN 0. Jian 7.	. 918	( 50 )	17.594	5.567	-72.536 -79.029	I TAN J TAN	0.000	( 12 )	15.739	-3.469	-262.158
= = HEMBER	9	2	== 3 ( 9		,	= MEHBER	80	2 - 2	= = 0 ( 9		:
ITAN G.	.918	( 21 ) ( 21 )	37.279	-8.880 -8.880	-74.164 -80.657	ITAN	0.000	(15)	-2.385	875.	-249.697
= = MEMBER	<b>5</b>	ы 1	== 0(4			= = MEHBER	) 6	l M		•	
ITAN G	.916	( 20 1	-37.279	8.830 8.830	-74.164 -80.657	ITAN	0.000	( 27 )	21, 264	-5.041	-206.643
= HEMBER 1	10 (	3 1 3*	= = 3 ( 3			= = MENBER	10 (	3	8) ( 8		
ITAN 0. JIAN 7.	. 918	( 21 )	26.486 -17.594	-5.567	-72.536 -79.029	I TAN JTAN	0.000	( 16 )	-26+061 21,759	6.039	-217,113

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N												-3.735		}		,	7		CT :	2.15		ᅻ.	2.15	9	2.05	-2.054	Z. U.2	-2.159					•			-	_		_		-1-891	1.890
0			201.01	79.197	24.007	14,379	-11.094	-35.558	-46,196	692.64	-131.277	-139.206	-5.214			6	? c	֓֞֜֜֜֜֜֜֜֜֓֓֓֓֜֜֜֜֓֓֓֓֓֓֜֜֜֜֜֓֓֓֓֜֜֜֜֓֓֓֓֡֓֜֜֡֓֡֓֜֡֓֡֓֡֓֡֡֡֡			2	15.0	24.15	81.28	167.68	-106,359	110.67	2.212	-		7	11.	<b>.</b>	7	6.5		ņ	;	-79-197	5.6	٠.	5.214
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X	) II	-48.521	29.36	<b>→</b> ~			75.80	3 8	90.0	407.00	3		173.491	# II 9 C		-154.835	22.89	39.81	53	69.23	52	.23	23	35.11	9	77.55		146.550	u 19 (	79.26	62.12	26,34	-10.453	7.32	82	8.11	72	2.4	6.03	100	3	99.690
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	9.650	( 11 )	-6.536	-7.739	0.00	· c	6.5		907-71	-10.076	0.00
NATO	10.000	( 25 )	-9.366	-8.433	0.00	JTAN	10.000	13.	-20.466	-11.786	000.0
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<b>→</b> 0	120		-63.556	12.085	0000		.350	(16)	-5.212	7.428	
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·	0.00		27.7	14.040	0.00	4	8.890	( 16 )	-14.073	-9.504	
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<b>∡</b>	7.0.4		3.055	-6.333	0 00 0	JTAN	9.640	( 16 )	-21.759	-10.991	0.00
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2		11,173	10.375	9.463	227	-9.917	-10.879	-11.627		12.228	11.430	10.518	1.239	-8-041	-6.953	-9.751		-1.890	-1.890			1.833		286	286		1.890
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MOMENT HAXIMUM

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SHEAR MINIMUM	H	.II II 	S,	•						35.	-69.017	ی د		3	98		3	20,0	9	36		-63.988	3.64		•		-30.929		•	•	٠	Ň.	į,	-48,775		9.0	N	**		11.5	1	-18.693	
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		· ·	38.197	30-643	27.536	19,164	-2.988	25	ĕ,	9	-42.091			'n,	40.822	i,	ů.	;	٠,	Š	'n	-40.822	ţ.	•	46.578	÷	39.024	ŝ	٠.	S.	<b>1</b> 6.		•	-35.792			.09	0	. 60	9	8.53	-4.323	•
AR MAXINUM	X	11 11 15 C	-48.775	9	3	ð	3.70	6	6.89	30.92	ב ל	H	. !	8.63	65.98	* 0	19.91	\$ 9 0 4	4	9.9	33.65	-63.980	78.63		84.96	9.01	35.54	15.0	2	<del>;</del> ;	23	۷:	***	-32,973	• •	יי ט	-11.815	-8.752	-5,550	966.6	-4.30	-12.872	7407
SHEAR	-CASE	1 - 2 1 - 3	( 18 )	٦, -	•	18	-	-		<del>ا</del> ا	200	N	:	<b>-</b> 4 .	4	7	٦.	r 0	٧ +	۰ ۵	++	(191		3 1 19	्.न	6	( 61 )	19	19	6	5	7	r <b>c</b>	( 13 )		5 1	-1	<b>6</b> 0	-	-	⊶.	- 2	4
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SHEAR MINIMUM

SHEAR MAXIMUM

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ITAN	0.000	( 19 )	-16.620	4, 917	5		- 4	:	•		
	.350	( 19 )	~	; ;		LIAN	000-0	٠ .	-17.080	9.913	00000
~	.750	( 19 )	-9.742	8.426		H 6	- C.		-13, 732	9,219	0.000
:	5.000	( 19 )	8.164	000		<b>u</b> r	200	- ·	-10.203	8.426	0.000
	9.250	( 19 )	-9.742	-8-475		7	0000	-1 ·	7.704	000	0.000
	9.650	( 19 )	-13.271	-9.219			067.6	- ·	-10.203	-8.426	0.000
	0.000	( 19 )	-16.620	-9.913	0000	JTAN	10.000		-13.732	-9.219	000.0
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	000 00	_	-22.730	11.883	000	TIBN		•	•		
	.350	u	-18.693	11,189		-	3 5	97	-16.256	10.017	0.00
~	750	J	-14.376	10.396	00.00	4 0	200	201	-12.872	9,323	0.000
	4.820	( 13 )	11.516	2,327		- د		707	105.6-	8.530	000.0
	3.890	( 19 )	4.565	-5.743		7 .3	000	10	966 - R	. 461	
r.	9.290	( 19 )	2.110	-6.536		ru	0.0	21.	-5.550	-7.509	0.000
	9.640	( 19 )	299	-7.230	0.00	NATO	9.640	( 18 )	-11,815	-8.402	0.00
= = MEMBER	7 (	1 + 5	= = 3 ( 5			= = MEHBER	~ ~	+ 1	= = O		•
ITAN	0.00	( 19 )	-4.593	.542	-100.212	ITAN		( 18 )		0	
Z		( 14: )	299	. 542	-108.681	- OTAN	7.918	( 18 )	-11.815	-2, 908	-111.086
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ITAN	0.00.0	( 19 )	•	1.570	-101.700	ITAN	ם כני			ć	
	. 918	( 19 )	6.111	1.570	-110.169	JIAN	.91	( 18 )	1.065	238	-99.295
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HOMENT MINIMUM

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3	9,250	2	4	7 2 2	9 6	· .		{ 51 }	6.007	2.879	
ī	9.650	21	2.9	100	3 6	، س	9.250	( 50 )	-19,955	-9,339	
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rAx	8.569	( 23 )	18,902	408	0.000	MAX	4.284	( 21 )	6,697	6,105	
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					t t		14.	1 77	33. 03	-8-880	-80.657

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		:	34,369	32,773	28.578	10.777	202.	-14.190	-20,609	-22, 991	-27,186	-28.782	•			₩,	30.224		N	NI	15.17	21.59	<b>*</b>	71.07			40.004	•	34.213	უ•	<b>ب</b> ر	JU	١.	. 5	S				60	5.0	,	200	-3.02B	3.56		
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	.750	<u> </u>	24.375	11.478	•			6.801	3.028	000.0
[ 21 ] 18.693	4.820	_		5.291			•	1,691	2.42	0.000
[ 21	8.890	_		896				5.149	-3.767	0.000
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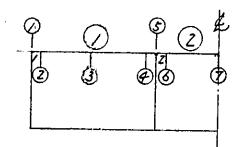
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	-7	( 20 )	5.04	27.84	2		2	25	Ÿ	21.42	
	νOι	( 22 )	7.66	25	8	. 🗪	, a	֓֞֞֜֓֓֓֓֓֓֓֓֓֓֓֓֓֓֓֓֓֓֓֓֓֓֓֓֓֓֡֓֓֓֓֓֓֓֡֓֓֡֓	•	<u> </u>	
o :	9.650	20 )	63	3	2	<b>o</b>	65	22	, ,	30.62	
-	•	( 20 )	2.90		26	J. ATAN 1	8	20 )	32	-36.015	1.250
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	6	20	8.24	Ġ	647	· ·		7.0	38.856	55	2.5
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	ζ;	02 )	3.7	32	- · · · ·	6	29	77	5,511	. n	,,
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ITAN	0.000	( 20 )	1.399	4.722	0.000	NATT		. 00			1
<b>-</b> 4∫	350	~;	2.958	4.190	0.000	•	2 15		7. C	22/**	0.000
∾.	.750	( 02 ( 20 )		3.582	0.000		757		0000	200	0000
<b>m</b>	2.000	, · · .		-2.879	0.000	, m	200.5		4.013	2.052	0.000
<b>.</b>	9.250	( 50 )	-19.955	-9,339	000.0		0.25		700.00	5.8.2	0.00
ر د	9.650	( 50 )	-23.812	276 -6-	0.000	, M	9.650		-13. 405 -03. 810	D 20 0	0.000
	10.000	( 20 )	-27.387	-10.479	8.	JTAN	10.000	( 20 )	-27.387	-10.479	0.000
= = MEMBER	9	7	8 3 6 =			量。	HENBER 6 (	- 6	     -		
ITAN	0.000	( 50 )		3.560		TTAN	6			. I	
-1	.350	( 20 )	6.801	3.028		-	•	~ ~ ~ ~	2.048	3.560	0.000
~	.750	( 20 )	7.891	2.420	0.00		750		5. 801 7. 801	3.028	0.000
m,	4.820		5.149	-3,767	-	; P	1000		160.7	024.2	0.00
3	8.890	( 20 )	-22.772	-9.954	_	) <b>.</b>	8.89		641 °C	73.76	000.0
	9.290		-26.875	-10.562	-	, PU	9.290	25	26.875	- 4. 4. 5. 4. 1. 1. 1. 1. 1. 1. 1. 1. 1. 1. 1. 1. 1.	00000
Z Z Z	9.640		-30,665	-11.094	္	JIAN	9.640	( 20 )	-30.665	-11.094	0.000
= = MEMBER	7	-	5 ) C = =			= # MEHBER	BER 7 (	1 - 5	= = O		
ITAN	0.000	( 20 )	-26.486	5.567	-72,536	TIAN			20.4		1
JAN	ij	( 02 )	7.59	295.5	-79,029	JTAN	7.918	( 21 )	-30.665	-6.514 -8.514	-83.758
H HEMBER	60	1 2	€ = 0 (9	•		= = HEMBER	BER 8 (	2 - 2	" " " O ~	•	
ITAN	0.000	( 21 )	37.279	-8.880	•	ITAN	0.830	. 02	-38.882	202	
JIAN	91	( 12 )	3.03	-8.880	-80.657	JTAN	7.918	( 02 )	34.810	9.307	-85.636
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O LE II	6	1	ا د د			,	٠. ا	1	0	300.F-	-85.636
		ı	<b>.</b>			= 1E 78E	BER 10 (	æ □ -#	" " " O		
LIAN	0.000 7.918	( 21 )	26.486	-5.567	-72.536 -79.029	LIAN	0.000	( 20 )	-36.749	8.514	-83.758

(6) Calculation of upper beam



(1) Calculation of Compressive stress caused by bending
(a) Summary of stresses

(i) At the support point

		•					Pi	CK UP 1	VO. 2
			(	$\mathcal{D}$			(2	)	
		0	co.	<b>S</b>	Co.	<u>©</u>	co.	0	Co.
Combined		-113.39	12	-21/3/	ا ا کرنا	-208.92	15		   
stress	Bottom		! <del>-</del>		   <del></del>	<del></del>			 
Dead A		-52.20	11	-71.87	77	-90.44	//		

(Note) Co. - Combination

(ii) span moment

				2	
		3	CO.	Ø	co.
combined	Bottom	17/13		248.55.	19
Mmin	u,		 		

(iii) Transit point to haunch

		<u> </u>				(2	)	
	2	co.	$\mathscr{P}$	co.	6	co.	0	co.
combined stress bottom	72.35	/3				-		

- (b) Allowable stress of upper beam, safe against cracking
  - (i) At the support point 1

    Dead load  $Md = -50.90^{+m}$  (case1)

    Train load + Impact  $ML = \frac{(-48.38-19.11)}{=-62.49}$   $EM = -1/3.39^{+m}$

$$d = \frac{62.49}{1/3.39} = 0.55 > 0.25$$

There fore (sa = 100 m/m

(ii) At the support point 2,3

Dead load

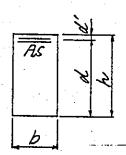
ML = -91.69 tm (Casel)

Train load + Impact ML = -119.67 " (Cases)

EM = - 24631 tm

There fore (sa = 1000 //m

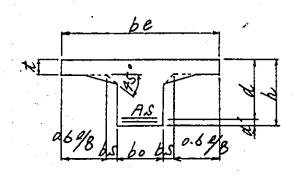
- (C) Cross Section
  - (i) Cross Section at the support point



(effective height used for shearing

Stress Calculation)

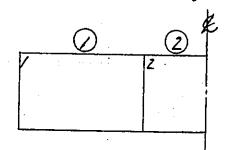
(ii) Effective width of T-beam Compression fibre



(6	O Bendin	stress	calcula.	tion		
				•.		•
	0	0	2	3		
M (tn	-50.90	-1/3.39	72.35	171.13	· .	
N (	()		, , , ,	,,,,,,,,,,,,,,,,,,,,,,,,,,,,,,,,,,,,,,,		
S (	()					
- b (сп	1) 70	"	245	245		
h (cn		"	120	120		
d (cr		1.	113.8	110.9		
ď (cr	1) 9.2		6.2	9.1		
	032 ( 2	·	032 - 7	2326 9		·
As (cm	71.48	3	55.59	103.25		
p	0.00677	4	0.00199	0.00380		
					·	
As' (cm	"			•		
p						
e = M/N (cr	n)					
e = M/N + u		7 - p				
e = M/N - u	7)					
e/h (t	<i>y</i>		(28)	(28)		
d/e						
d /h			·		· .	
d /d ( */a	ソ		(0.246)	(0.252)		
He/bd*(kg/cm	2) 3.198	7.123	2.280	5.679		
k				0.287		
С	-					
j			•	0.907	·	<u>,                                      </u>
1/Lc	6.32	4	9.96			
1/Ls	169	. 3	591			
β= σs/ σc				_		<u> </u>
σc (kg/eπ	(2)	45.0	22.7	44.3		ļ
OS (kg/cm	540	1200	1230	1650		
T (kg/cm	i <sup>2</sup> )					
osa (kg/cm	1) 1000	1800	1800	1800		
σca (kg/cn		90	90	90		
τα , (kg/cπ						
number	M-1	"	M-47.M-1	M-47.48		
Combination		P+T+I	D+T+1	D+T+L		LICA

		7	<u> </u>	1		•	<del>-</del>
					<u>2</u> )	· <del>- </del>	
34 /	$\geq$	<u>©</u>	<b>S</b>	9	<u> </u>	<del>                                     </del>	_
· · · · · · · · · · · · · · · · · · ·	tm)	-91.64	-211.31	148.55	<u> </u>		
<del></del>	(1)	•				_	
	(1)						
<del></del>	cm)	70	4	250			
<del></del>	cm)	160	"	120			
	cm)	150 B	4	110.9			
<u>ď</u> (	cm)	9.2	. 3	9.1			
As (c	m²)	032 < 3	,	032 < 5			,
		95.30		103.25			
p		0.00903		0.00372			
As' (c	<sub>m²)</sub>				,		
	/				·	· <u> </u> _	
p <sup>'</sup>		•					
e = M/N (c	m)						
e = M/N +	m) u		,				
e = M/N -	in)	,	•				
e/h (1	ارب			(28)			
d/e						<u> </u>	
ď/h							
d'/d (5/4			····································	(0.252)		-	
Nelbd" kg/cr	n²)	5. 757	13-275	4.831			
k	_	3.707	70.270	0.285			<u>·</u>
c	7			0.200			
<i>j</i> .	+			0.907		<del>                                     </del>	-
1/Lc	$\dashv$	C 74		V. FUE	<u></u>	<del> </del>	
1/Ls	$\dashv$	5.79	<i>'</i>			-	
3= os/ oc	-+	128	<u>, , , , , , , , , , , , , , , , , , , </u>			<u> </u>	
TC (kg/cm	,2)		7/ 7	270			<u> </u>
7S (kg/cm		740	76.Z	37.9			
(kg/cm		740	1700	1430			_
	+	-10	<u> </u>	2.50			
	+	1000	1800	1800	· 		<del> </del>
TCQ (kg/cm	<del> -</del>		90	90		<u> </u>	·
C (kg/cm							
omo gran um ben	$\perp$	M-1	<u> </u>	M-47.48		<u></u>	
mbinatu	m	D	D+T+I	0+ [+[			1

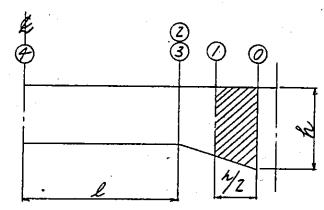
(2) Calculation of Shearing Stress



(a) Summany of shearing stresses

For examining Section 0, shearing stress

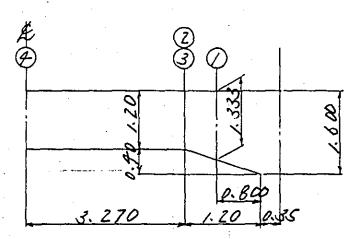
at Section 1 is used



N.				<del></del>	T	·	<del></del>		
		(,		2					
	Left supposit	CO,	Right support	CO;	Left.Support	co.	Right Support	00,	
0	110.26	12	-131.28	15.	127.31	15			
0	91.32	(/	-1/2.34	7	108.37	7			
2	56.77	2	702.44	5	98.47	5			
3	56.77	43)	-102.#1		98.47				
<b>F</b>	-5.38		-15.87		6.85	_			

(Note) CO. - Combination

(b) Shearing stress caused by bending



(i) Correction for shearing stress  $S = So - \frac{M}{d} (tand)$ 

Where, M: Bending moment (tm)

d: Effective height (m)

d: An angle of elevation of the member

So: Shearing, stress caused by bending

	20 (t)	M (tm)	d (cm)	tanx	S (t)
$\bigcirc$	112.34	-85.73	1.Z9 K	//3	89.31
2	102.44	-48.25	1.108	//3	87.92
3	102.44		1.108		102.99
$\mathcal{G}$	15.69	<u></u>	1.108		15.64

Combination 15 (1+5)

$$T = \frac{S}{b \cdot d}$$

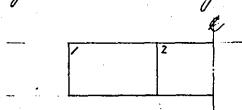
Where

- (C) Shearing stress caused by tonsional moment
  - (i) Tonsional moment

Refer the result of computer analysis on torsional moment.

(Without the beam for electric pole,

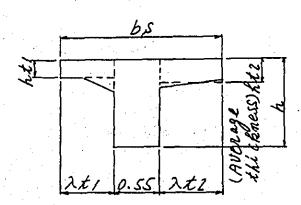
Single track loading)



	, ,	Distance	Ma	Mt (tm)		
	point	(m)	O-Omember			
0	column front	0.350	13.93	14.68		
$\bigcirc$	1/2 Point	1.150	11.44	12.15		
2	Transit pt.	1.550	9.46	10.11		
3	11	1.550	9.46	10.11		
4	Mid - Span	5.000	0	0		

Value in () represents 1-2 member

### (ii) Effective width



One side effective width of Projected flange subjected to the torsional moment is calculated followed the equation.

1 = 3. Rt

Intermediate part  $\lambda t_1 \leq \frac{l_0}{2}$ Cantilever part  $\lambda t_2 \leq l_C$ 

Where,

At: One Side effective width of projected flange(m)

ht: Thickness of projected flange (m)

lb: nex clearance between girdens (m)

lc: Projecting length of cantilever slab (m)

hti = 0.28 m htz = 0.315 m (Average thickness)

\(\lambda t/ = 3 \times 0.28 = 0.840 m < \frac{\lb}{2} = 2.150 m

Atz = 3 x 0.315 = 0.945" < lo = 2.100 "

Column front ho = 1.600 m

4/2 point h1 = 133"

Transit pt. of haunch hz = h3 = 1.200 m

(iii) Shearing stress caused by torsion on T section Torsional shearing stress is calculated followed the equation.

 $Tti = \frac{MT}{Lt} \cdot bi \cdot \eta i$ 

Where, Thi: Shearing stress of concrete Calculated
on each rectangular Section (\*%)

MT: Torsional moment ( M/m)

bi: Shorter side of each nectangular

Section (cm)

Ni Referred Table - 40.(2).

It: Tonsional moment of inentia (ont)

ai: Longer Side of each rectangular Section

Ri: Referred Table - 40. (2).

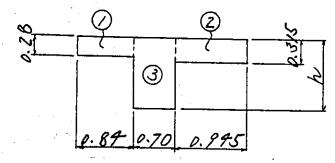
### Table - 40 (2) Coefficient qi

2/6	1.0	1.2	7.5	1.75	2.0	2,5	3.0	4.0	5.0
ri	0.675	0.759	0.89-8	0.895	0.930	0.968	0.985	0.997	0.999

### Table - 40.(3). Coefficient ki

a/b	1.0	1.2	1.5	1.75	2.0	2.5	3.0	4.0	5.0
ki	0.190	0.166	0.196	0.219	0.229	0.249	0.263	0.28/	0.292

### (i) Tonsional moment of inentia



		a (m)	b (m)	2/6	k	[t=k.a.b3 (m4)
	$\bigcirc$	0.890	0.280	3. 000	0.263	0.263 × 0.840 × 0.283 = 0.00 +85
	2	ĺ				0.263 × 0.945 × 0.3/5 = 0.00777
	Column front	1.600	0.700	2.286	0.240	0.240×1.600 × 0.7003 = 0.13/71
(3)	R/Z Pt.					0.223 x 1.333 x 0.700 = 0.10196
	Transit Pt. Of haunch	1.200	0.700	1.714	0.211	0.211 × 1.200 × 0.7003 = 0.08685

Column front

1/2 Point

Transit point of haunch

# (ii) Tonsional moment beared by longitudinal beam (for Calculation of axial ne-bans) $Mt = MT \cdot \frac{It3}{\Sigma It}$

Column front

M/Z point

Transit point of haunch

$$Mt2 = Mt3 = 10.11 \times \frac{8.685 \times 10^6}{9.947 \times 10^6} = 8.83 tm$$

(iii) Toesional shearing stress of longitudinal beam

$$\frac{a}{b} = \frac{133.3}{70} = 1.904$$

$$Tt/ = \frac{12.15 \times 10^5}{11.458 \times 10^6} \times 70.0 \times 0.917 = 6.81$$

Transit point of haunch

$$Mt = 10.11$$
 tm

$$\frac{a}{b} = \frac{120}{70} = 1.714$$

$$7t2 = \frac{10.11 \times 10^{5}}{9.947 \times 10^{6}} \times 70.0 \times 0.888 = 6.32$$

(d) Combined Shearing Stress

Tb (3.9 % --- Diagonal tension re-bars not are not required Calculation

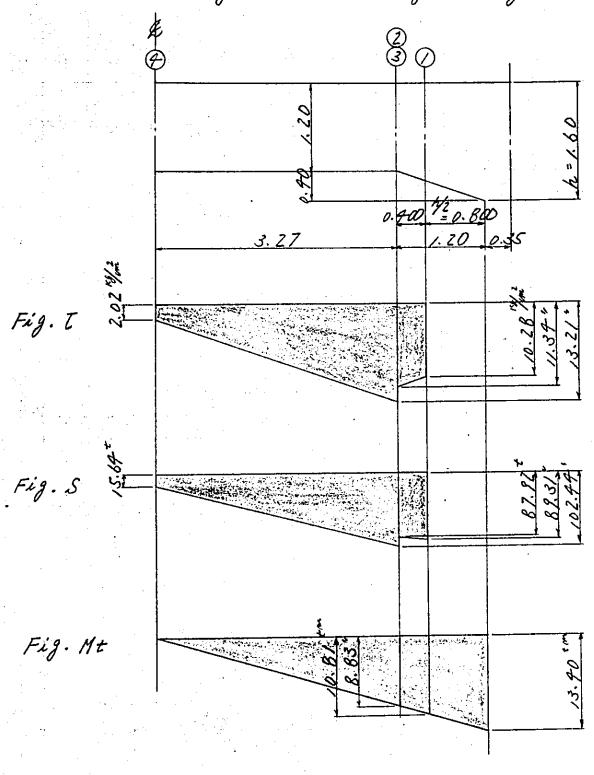
Tb <17" --- Diagonal tension ne-bans are calculated.

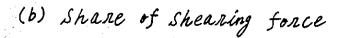
Tb + Tt <17 ×1.3 = 22.1 1/m --- Tonsion is considered

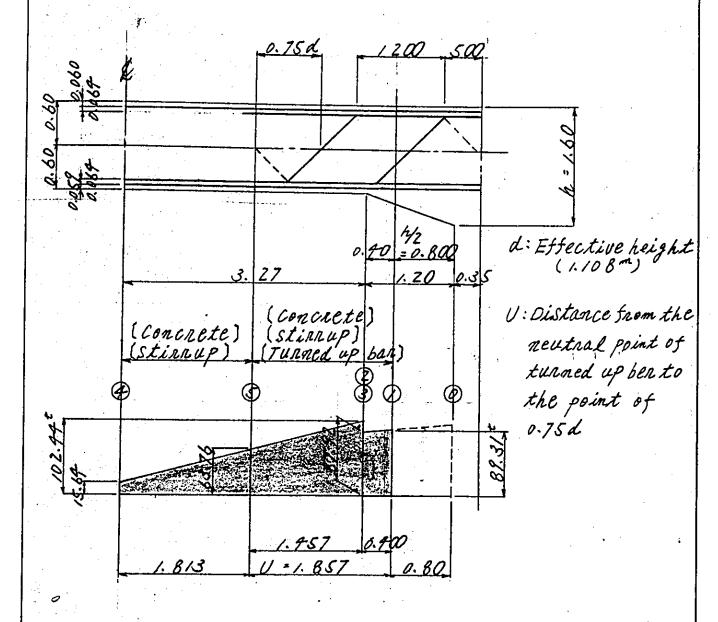
The Tt 1/2 point 10.28 + 6.81 = 17.09% < 22.1%Thansit pt. 11.34 + 6.32 = 17.66% < 0.0% 1.3.21 + 6.32 = 19.53% < 0.0%Span Center 2.02 + 0 = 2.02% < 0.0%

Calculated as above, diagonal tension re-bases are examined.

## (3). Colculation of diagonal tension re-bans (a) Shearing stress caused by bending







$$U = (0.50 + 1.20 + 0.600 - 0.060 - 0.064 + 0.75 \times 1.108)$$

$$- (0.35 + 0.800) = 1.857^{m}$$

$$Sd \text{ at point } U$$

$$Sd = \frac{102.44 - 15.64}{3.270} \times 1.813 + 15.64 = 63.76^{t}$$

- (C) Shearing force beared by concrete
  - (i) Area F subjected total shear within the range of U $F = (87.92 + 89.31) \times 0.900 \times 1/2 + (102.99 + 63.76) \times 1.457 \times 1/2 = 156.52$  tm
  - Ui) Area Fe subjected shear beared by concrete

    Sc = 1/2 · Tc · b · d

Where

SC: Shearing fonce beared by concrete (+)

TC: (CK = 240 1/m. TC = 3.9 15/mi

b: Width of cross section of member (um)

d: Effective height of member (om)

Sc1 = 1/2 × 3.9 × 70 × 110.8 × 10-3 = 15.12 t

SCZ = 1/2 ×3.9 × 70 ×129.1 × 10-3 = 16.99"

:. Fo = 15.12 x 1.457 + (15.12 + 16.94) x 0.400 x /2

= 28.44 to

- (d) Shearing force beared by Stirrup

  Annange Stirrups D13-2 sets in 15.0 cm ctc.
  - i) Tonsional shearing stress

Where,

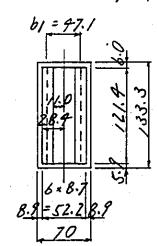
Mt: Tonsional moment (tm)

A: ctc distance of stirrup (om)

Av: Gross cross Section of Coupled Stinnups (in)

bi. hi: Length of short long side of stirzup

### (i) At h/z point



Mt: 10.81 +m

8: 15.0 cm

AV: 1.267 × 9 = 5.07 cm²

h1: 121.4 + 3.2 + 1.3 = 125.9 cm

 $b_1: \frac{11.0^2 + 28.4^2}{11.0 + 28.4} \times Z = 47.1$  om

$$\int St_1 = \frac{10.81 \times 10^5 \times 15.0}{0.8 \times 5.07 \times 47.1 \times 125.9} \times \frac{28.4 \times 2}{47.1}$$

= 8/3 Kg/cm < 1800 Fg/cm

(ii) At the transit point to haunch

$$Mt = 8.83$$
 tm

$$\int Stz = \frac{8.83 \times 10^{5} \times 15.0}{0.8 \times 5.07 \times 47.1 \times 1/2.6} \times \frac{28.4 \times 2}{47.1}$$

$$= 743 \frac{19/01}{0.8} \left( 1800 \frac{15/01}{0.8} \right)$$

(iii) At U point

$$Mt = \frac{B.83}{3.270} \times 1.813 = 4.90 \text{ tm}$$
 $h_1 = 1/2.6 \text{ cm}$ 

$$\int St3 = \frac{4.90 \times 10^5 \times 15.0}{0.8 \times 5.07 \times 47.1 \times 112.6} \times \frac{28.4 \times 2}{97.1}$$

$$= 9-12 \quad \text{Key } < 1.800 \, \text{leg/cm}^2$$

ii) Bending Shear beared by Stirrup

In the case when combined with torsional moment, allowable shearing Stress is

as 20 percent increased.

$$\int SQ = 1800 \times 1.2 = 2160 \frac{\text{Fg/m}^2}{1.15 \cdot \text{S}}$$

$$SV = \frac{(Sxa - (Txt) \cdot Av \cdot d}{1.15 \cdot \text{S}}$$

(i) At 
$$h/z$$
 point  
 $(2/60 - 8/3) = 1.3 + 7^{8}/m^{2} < 1800^{8}/m^{2}$   
 $d = 124.1^{000}$   
 $SV = \frac{1347 \times 5.07 \times 124.1}{1.15 \times 15.0 \times 10^{3}} = 49.13^{\pm}$ 

(ii) At the point transit to haunch
$$(2/60 - 743) = /4/7 \quad \text{Mon}^{2} < /800 \quad \text{Mon}^{2}$$

$$d = /10.8 \quad \text{om}$$

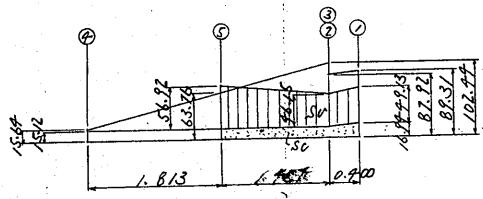
$$SV = \frac{14/7 \times 5.07 \times 1/0.8}{1.15 \times 15.0 \times 10^{3}} = 46.15^{\pm}$$

(iii) At U point  

$$(2/60 - 4/2) = 1748^{-\frac{K}{2}} (1800^{\frac{K}{2}})^{\frac{1}{2}}$$

$$d = 1/0.8^{cm}$$

$$SV = \frac{1748 \times 5.07 \times 1/0.8}{1.15 \times 15.0 \times 10^{3}} = 56.92^{\frac{1}{2}}$$

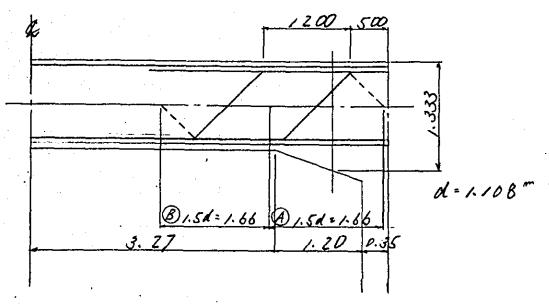


Anea Fv fon the shear beared by stirrip  $Fv = (56.92 + 46.15) \times 1.457 \times 1/2 + (46.15 + 49.13)$   $\times 0.400 \times 1/2 = 94.14^{t}$ 

$$1/2 \times (F - Fc) = 1/2 \times (156.52 - 28.44) = 69.04^{\pm}$$
  
 $(Fv = 99.14^{\pm})$ 

(e) Shear beared by turned up bar

(i) Shear beared by A



Effective range of turned up bars is assumed as the distance of 0.75d of each arrangement.

Sin 8 + COSB = 1.919

Where,

(sa: Allowable tensile stress of bar ( )

As: Cross Section of turned up bor (cm)

0: Elevation angle of turned up bar with the axis of member 0=950

S: Spacing of turned up bars in axial direction of member (m)

Fb.A = 1800 x 15.88 x 129.1 x 1.414 = 43.62 tm

Therefore, the average resisting shearwithin 1.5d distance will be

 $Sb.A = \frac{Fb.A}{S} = \frac{43.62}{1.66} = 26.28^{\pm}$ 

(ii) Shear beared by B

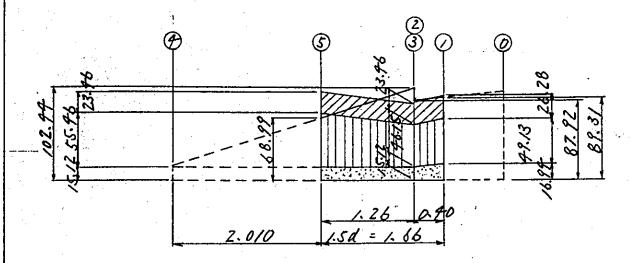
AS = D32 - 2 = 15.88 om

L = 110.8 om

Fb.B = 1800 × 15.88 × 110.8 × 1.414 = 38.94 +m

Sb.B = 38.94 = 23.46 t

### (f) Resultant resisting shear (i) Section A

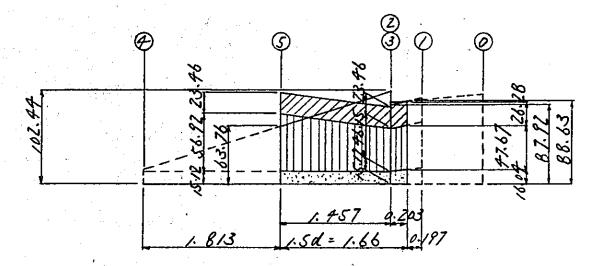


Area of acting Shear

Area of resisting shear

$$FU + FV + Fb = 25.46 + 83.07 + 40.07$$
$$= 148.60 + 5 = 143.45 + 5$$

#### (ii) Section B



Area of acting shear

Area of resisting shear

### (g) Calculation of bans in oxial direction

Required bans are calculated followed the equation

$$AS = \frac{Mt (b_1 + h_1)}{0.8 \cdot (Sa \cdot b_1 \cdot h_1)}$$

Where

引动机 化复数分类医解放性 人名英格兰人姓

As: Bans in oxial direction

Mt: Torsional moment

Tsa: Allowable stress of bar

bi. hi: Length of short/Long side of Stirrup

i) At Column front

Mt: 13.40 tm

TS: 1800 59/m2

b1: 28.4 x 2 = 56.8 cm

h1: 160.0 - 6.0 - 5.9 + 3.2. + 1.3 = 152.6 cm

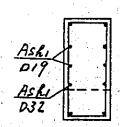
AS = 13.90 × 105 × (56. B=+ 152.6) = 22.98 om

Required ban arrangement for Shorter Side

$$ASb_1 = 22.98 \times \frac{56.8}{2 \times (56.8 + 152.6)} = 3.05$$

Required ban arrangement for longer side

$$Ash_1 = 22.48 \times \frac{152.6}{2 \times (56.8 + 152.6)} = 8.19$$
 on



(i) Top Side (refer the calculation of bending steess)

... Use main bans for this purpose as well.

$$As = \frac{1800 \times 1.2 - 1700}{1800} \times 95.30 = 24.35$$

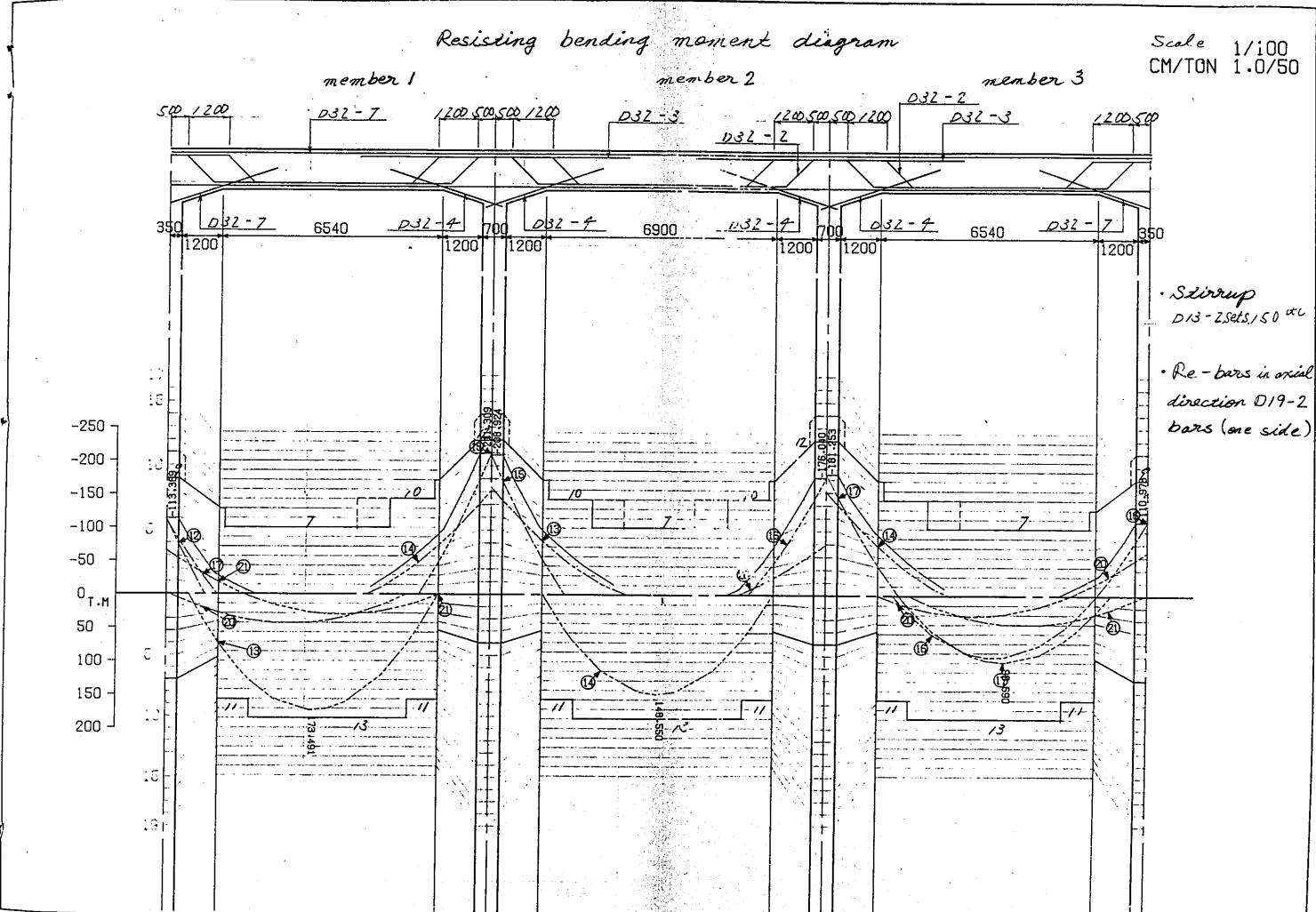
 $AS' = D32 - 12 = 95.30 \text{ cm}^2$ 

(ii) Bottom Side (refer the same). Use haunch bar for this purpose as well

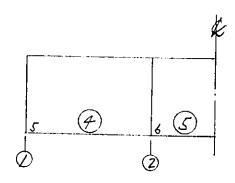
AS -032 - 4 = 31.77 ont > 3.05 ont

(iii) Side (One Side)

AS = 019 - 2 > 13.67 cm > 8.19 cm



### (7) Calculation of buried beam



(1) Summary of stresses
(a) Bending moment

						Pi	CK UP	NO.2	
		$\mathscr{P}$				<u>(5)</u>			
	Ø	co.	2	co.	2	co.	0	co.	
TOP	-30.67	12/	-33.4/	20	-27.39	21			
Bottom	.18.69	20	6.80	2/	9.97	20			
Dead	-9.80	//	-Z0:82	11	-19.49	11		1	

### (b) Shearing force

	$\mathscr{P}$	<b>S</b>	Co. Left. 1/2 co.		
Lett 4/2 Point	CO. Left 42 CO.	Left 1/2 co. Le	st. ½ co.		
9.95	21 =11.48 20	9.91 17			

(Note) co. - Combination

(6	) stres	s Calc	ulation			
		<b>(4)</b>				
	1) Bottom		2 TOP			
M (tm)	*		-33.41			
N (i)	70.07	20.02	35177			
S (1)						
b (cm)	60	60	60		· · · · · · · · · · · · · · · · · · ·	
h (cm)	80	80	80		·	
d (cm)	65	72.9	72.9			
d (cm)	15	7./	7.1			
4- (- 1)		P32-5	032-5.			,
As (cm²)	=39.71	=39.7/	=39.71		<u>-</u>	
p	0.01018	0.00908	0.00908			
As' (cm²)						
AS (cm-)				•		
p						
e = M/N (cm)						
e = M/N + u						
e=M/N-u				-		
e/h				,		
d/e						
d'/h		·				<del></del>
d'/d						
Ne/bd*(kg/cm²)	7.37	6.53	10.98	•		
<u>k</u>						
c ·						
j						
1/Lc	5.53	5.73	5:75			
1/Ls	119	127	127			
β= σs/ σc						
σc (kg/cm²)	40.8	· · · · · · · · · · · · · · · · · · ·	60			
OS (kg/cm²)	890	830	1330			
T (kg/cm²)						
OSO (kg/cm²)	90		90			ļ
oca (kg/cm*)	1800	1600	1800	•		
Nome Jalin						<u> </u>
number	M-1	<u> </u>	"			
combination	2 + 0	D	D+5			1100

- (3). Required minimum bar annangement
  - (i) Top of the support point

Therefore,

(ii) Bottom of support point

$$AS = \frac{18.69 \times 10^{5}}{1800 \times 0.875 \times 65} \times \frac{4}{3} = 24.34 \text{ cm}^{2}$$

There fore

- (4) Shearing Stress
  - (a) Shearing stress caused by bending

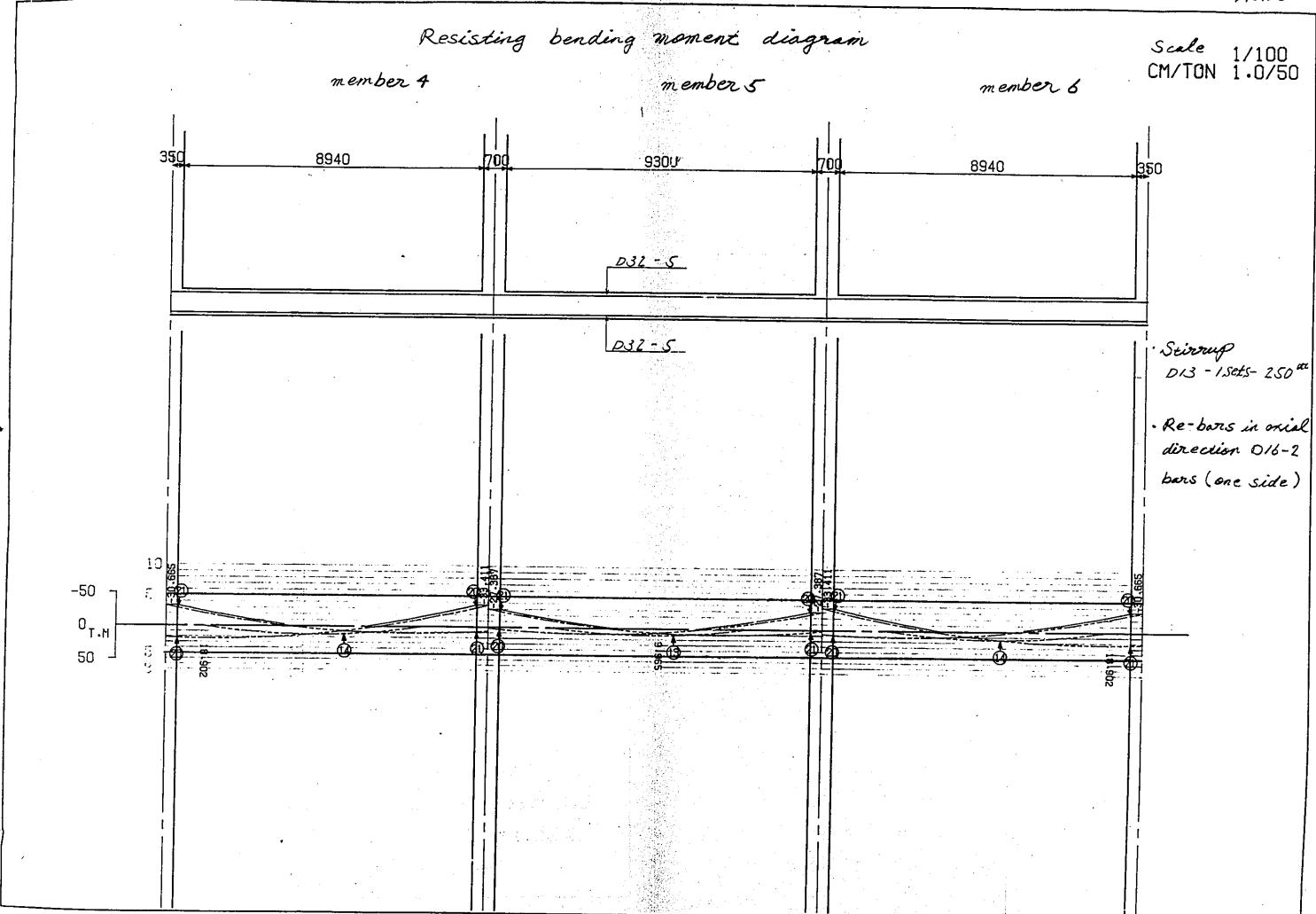
$$T = \frac{11.98 \times 10^{3}}{60 \times 72.1} = 2.65 \frac{10^{2}}{100} (3.9)^{100}$$

Therefore stinnup

Use D13 - One set in 25 cm utc.

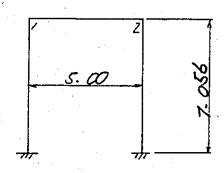
Ban annangement in axial direction

USE D16-2 bans (One side)



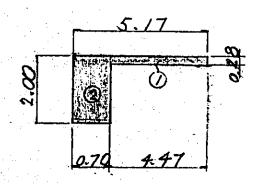
- § 6 Rigid frame analysis on transversal section (O-O) of elevated structure
  - (1) Elements for rigid frame analysis

    1. Conliquiation and dimension



2. Cross-Sectional area and moment of Inertia of the member

(1) Member (1~2)



effective width

be = 9.64 × 1/2 + 0.35

= 5.17 m

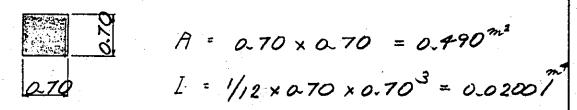
	b (m)	h (m)	A (m²)	y (m)	A. y (m)
$\bigcirc$	4.470	0280	1.252	0-140	0-17528
2	0 700	2-000	1.400	1.000	1. 40000
Σ			2-652		1.57528

$$y = \frac{1.57528}{2.652} = 0.594^m$$

	•						
	b (m)	h (m)	A (m)	y (m)	I. (m4)	A.y. 2 (m4)	Io + A. yoz (mt)
$\bigcirc$	4.470	0.280	1.252	0.454	0-00818	0. 25806	0-26624
2						0-23077	l
Σ			2.652		0.47485	048883	0.96368

CYOSS-Sectional Area  $A = 2.652^{m^2}$ Moment of Inertia  $I = 0.96368^{m^4}$ 

### (2) Member (1~3.2~4)



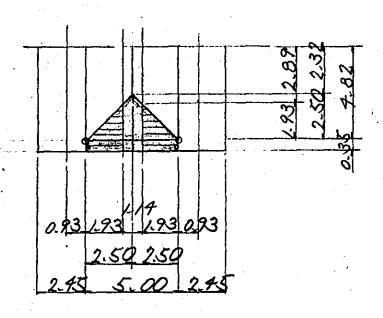
(3) Oxial height

h1 = 7.650 - 2.584 = 2.056

(2) Calculation of loads

1. Dead Load

(1) Member (1-2)



(a) Disthibuted Load (A)

Weight of Slab grading Concrete and Ballast

Slab  $2.5 \frac{1}{100} \times 0.28 = 0.700 \frac{1}{100}$ grading concrete  $2.35^{\circ} \times 0.07 = 0.165^{\circ}$ Ballast  $1.9^{\circ} \times 0.481 = 0.919^{\circ}$ Wd =  $1.78 \frac{1}{100}$ 

Wd2 = 1.78 m x 0.35 = 0.62 m Wd2 = 1.78 x (0.35 + 2.50) = 5.07 m

- (b) Distributed load (8) of Track assembly

  Distributed width  $b = 2.00 + 1.5 \times (0.316 + 0.07) + 0.28 = 2.86^{m}$   $w' = 0.45 \% \times \frac{1}{2.86} = 0.16 \%$   $w' = 0.16 \% \times 0.35 = 0.06 \%$   $w' = 0.16 \% \times (0.35 + 1.93) = 0.36^{m}$
- (c) Distributed load (c) of CROSS beam and haunch of Slub

Beam  $2.5 \% \times 0.70 \times 1.72 = 3.01 \%$ Haunch  $2.5 \% \times 0.45 \times 0.15 \times 1/2 = 0.08$ 

Wd5 = 3.09 /m

- (d) Concentrated load of elements
  acting at joint P1. P2 as shown below
- 1) Weight of elements on the slub except the weight of track assembly (2.94-0.45 1/m) × (0.35+4.82) = 12.87<sup>t</sup>
- 2) Distributed load (A)

1.78 x (2.32 + +.82) × /2 × 2.50 = 15.89

3) Distributed load (B)

a16 x (289+ 4.82) × 1/2 × 1.93 = 1.19

4) Distributed load (C)

a16"x (482+a35) x a 93 = a77

5) Cantilever slab

2.5 x (0.20 + 0.43) x /2 x 2-10 x 5.17 = 8.55

6) Haunch of Slab

25 x 0.45 x 25 x 1/2 x 402 = 0.34

7) Longitudinal beam

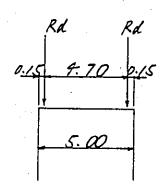
 $2.5' \times 0.35 \times (1.20 + 0.92) \times 2.06 = 3.82'$ 

B) Haunch of longitudinal beam

25 × 1.20 × 0.40 × 1/2 × 0.70 = 6: 100

$$2.5 \times 0.70 \times 0.70 \times (0.594 - 0.28) = 0.38$$

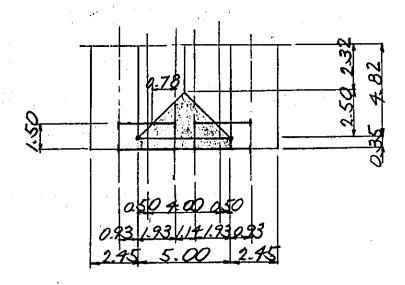
### (e) Reaction of T-beam superstructure

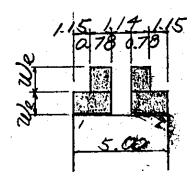


Column Weight

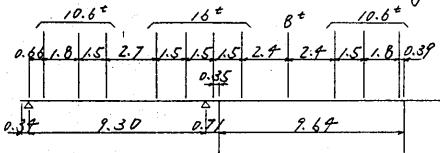
- 2. Train Load and Impact
- (1) Train Load (Single track)

  KS-16
  - (a) Distributed load acting on rigid frame





(b) Concentrated loads Coused axial loads acting on longituduial beams and T-beams of the bridge



(i) Concentrated load caused by axial load acting on longitudinal beam

$$R0 = \frac{1}{9.64} \times \{10.6 \times (0.39 + 2.19 + 3.69) + 8.0 \times 6.09 + 18.0 \times 8.49 \} + 16.0 = 42.04^{t}$$

$$P = 42.04^{t} - 5.59^{t} \times (0.78 + 1.93) = 26.89^{t}$$

(ii) Concentrated load Caused axial load acting on T-beam bridge

$$RB = \frac{1}{9.30} \times \{10.6 \times (0.66 + 2.46 + 34=4) - 16 \times (0.66 + 8.16)\} = 33.57^{t}$$

- (2) Impact coefficient
  - (a) Within Yigid frame Section

    l1 = 5-00<sup>m</sup> i01 = 0.480

    l2 = 9.73 i02 = 0.433

For the case of double track reduction is made tollwed formula.

• Red tion of impact coefficient

ii = 0.480× (1 -  $\frac{5.20}{200}$ ) = 0.468

i2 = 2433× (1 -  $\frac{9.73}{200}$ ) = 0.412

Reduction of impact coefficient

is = 0.437 × (1 - 
$$\frac{9.30}{200}$$
) = 0.417

· concentrated load Subjected Single track loading

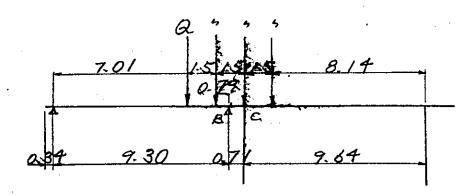
# · concentrated load occured subjected Single track loading

$$\frac{4.35}{4.70} = \frac{p}{4.70} \times 0.35 = 0.0745.p$$

$$\frac{a}{4.70} \times 4.35 = 0.9255.p$$

$$Pa = 47.57 \times 0.0745 = 3.54^{t}$$
 $Pb = 47.57 \times 0.9255 = 44.03^{t}$ 

# 3. Train Lateral Load $Q = 16 \times 0.15 = 2.40^{t}$



$$R_{B} = \frac{1}{9.30} \times 2.40 \times (7.01 + 8.51) = 4.01^{\pm}$$

$$R_{C} = \frac{1}{9.64} \times 2.40 \times (8.14 + 9.64) = 4.43^{\pm}$$

$$8.44^{\pm}$$

4. Force of temperature change and/or Drying Contraction

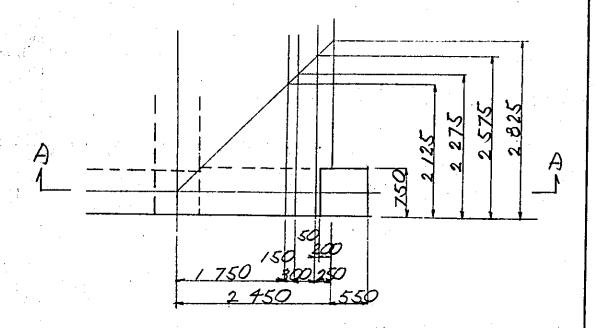
- · Temperature vise
- Temperature drop + Drying Contraction
  -15°C

5. Dead load + Seismic force Kh = 0.1

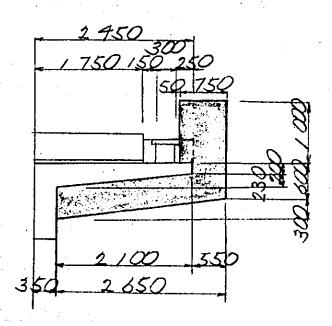
 $H = \left\{ 3.09^{\frac{1}{12}} \times 5.00 + (0.62 + 5.07)^{\frac{1}{12}} \frac{1}{2} \times 2.50 \times 2 + (0.06 + 0.36)^{\frac{1}{12}} \times \frac{1}{2} \times 1.93 \times 2 + 43.56^{\frac{1}{12}} \times 2 + 59.55^{\frac{1}{12}} \times 2 + 1.23^{\frac{1}{12}} \times 7.056 \times \frac{1}{2} \times 2 \right\} \times 0.10$   $= 24.52^{\frac{1}{12}}$ 

6. Load of electric pole,

Support of electric pole and related installations



Section A-A

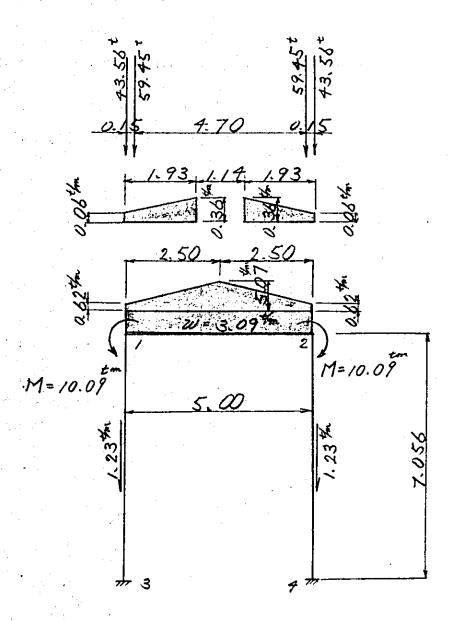


electric pole load

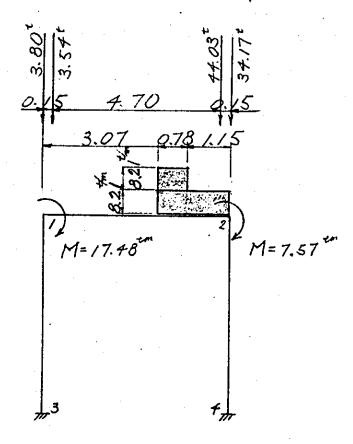
M=5.00<sup>tm</sup> N=2.00<sup>t</sup> H=0.50<sup>t</sup>

			· · · · · · · · · · · · · · · · · · ·	1
	Calculation	N (t)	X (m)	M(tm)
Electric pale		į		±5.00
		0 2.00	2,625	5.25
handrail	0.20 × 1.975	(0.40)	2.350	0.94
Curb	2.5 × 0.30 × 0.25 × 1.95	(0.37)	2325	0.86
ballase stopper	2.5" x 0.15 x 0.30 x 2.20	Ī		•
Duck Cover	2.5 × 0.05 × 0.30 × 2.425	)		
Cable	0.06 × 2.425	l " :	2-050	0.3/
ballast	1.9 × 0.481 × (0.375+2125)	,	·	
	×1/2 × 1.75		1.079	2-16
grading concrete	2.35 × a07 × ( a375 + 2.125	)		
	× 1/2 × 1.75	(0.36)	1-079	0.39
Slab	2.5 x 0.20 x (0.75+ 2.825)			
	×1/2 × 2-10	(1.88)	1-603	3.01
<b>.</b>	2.5 × a 23 × (0.75 + 2.825)			
	× 1/4 × 2-10	(1.08)	1.327	1.43
Support of	2.5 x {0.60 x 2.65 - (0.20 +0.45	)		
electric pele		· 1.74	2238	3.98
	2.5 x 0.30 x 2.65 x /3 x 0.75	0.75	1.2 <b>3</b> 3	0.92
<b>,</b>	25" x 0.75 × 0.75 × 1.00		2.625	
		5.90°		+ 28,59° - 19,59°
			<u> </u>	<del></del>

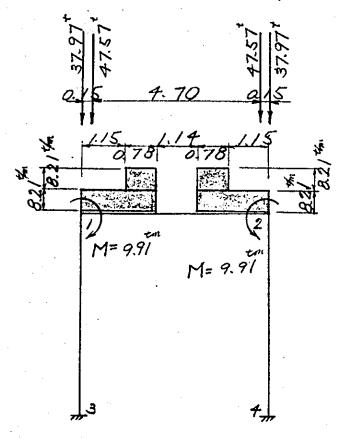
# (3) Loading diagram 1. case 1 Dead load



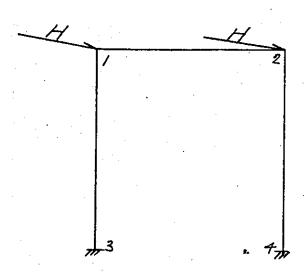




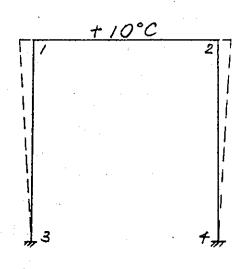
3. case 3 Train load + Inpact



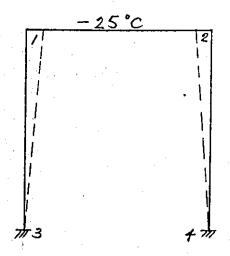
4. case 4 Train laxeral load  $H = \frac{8.44}{2} = 4.22^{\pm}$ 



5. case 5 Temperature rise

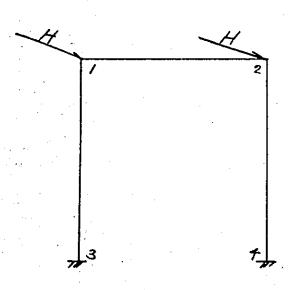


# 6. COSL6. Temperature drop + drying Contraction

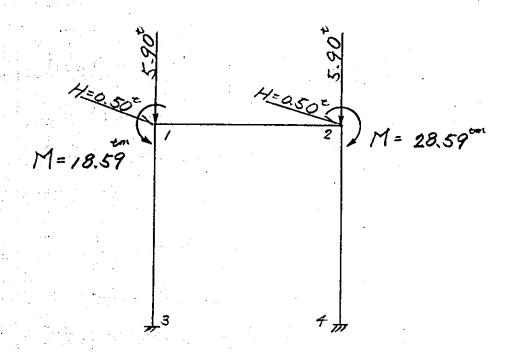


7. case 7 Seismic Road

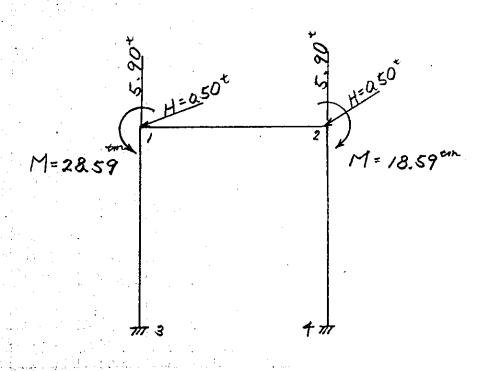
$$H = \frac{24.52}{2} = 12.26^{t}$$



## 8. case 8 Electric pale load



## 9. use 9 Electric pale load



# (4) Combination of loads

#### 1. Basic load

Case No.	Kind of load	losding Pattern
1	Dead load	
2	Train load + Impace	
3	Train load + Impact	
	Train Lateral load	
	Temperature	+ 10°C
6		- 25°C
7	Seismic Load	· Ti
8	Electric pole load	*
9		
9	Electric pole Road  Electric pole Road	

#### 2. Combined loads

Cool No.	combination of loads	<b>X</b>
10		1.000
11	1 + 2	,
12	/ + 3	5
13	1+2+4	0.8696
14	1 + 3 + 4	٠,
15	1 + 2 - 4	4
16	1+3-4	7
17	1 + 5	· 3
18	1 + 6	5
19	1 + 7	0.6667
20	1 - 7	<b>5</b>
2.1	1 + 8	1-0000
22	1 + 9	,
23	1 + 2 + 8	
24	1 + 3 + 8	5
25	1+2-9	,
26	1+3-9	<b>,</b>
27	1+2+4+8	0.8696
28	1+3+4+8	4

cose No.	Combination of loads	2
29	1 + 2 - 4 + 9	0.8696
30	1+3-4+9	4
3/	1+5+8	4
32	1 + 6 + 8	,
33	1 + 5 + 9	4,
34	1 + 6 + 9	67
35	1+1+8	0.6667
<b>૩</b> ૪	1-7+9	4

#### 3. Critical case

No.1 cose 10 (Without the load of electric pole)
the cose of analysis safe
against cracking

No.2 case 11~20 ( " ) the case of analysis of beams.

No.3 case 21~22 (With the load of electric Pole)
the Cash of analysis safe
against craking

No.4 case 23-26 (") the case of analysis of beams.

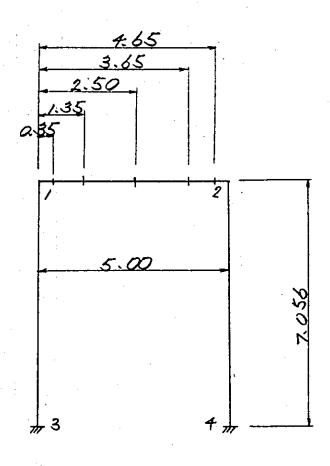
No.5 case 11 ~ 20 the case of analysis of column.

No.6 case 11 ~ 16

No.6 case 13 ~ 30 the case of analysis of foundations

No.7 case 35~36 the case of siesmic analysis of foundations

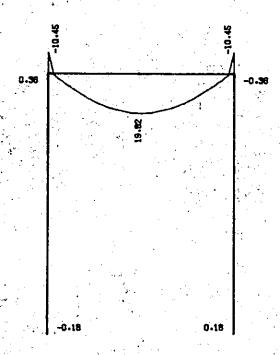
## 4. Points of computing Stresses



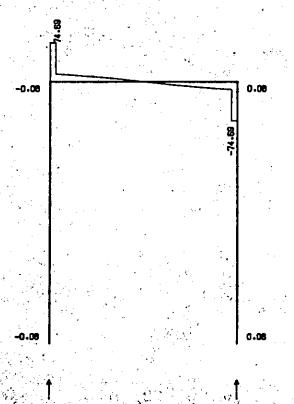
(s) stress diagram VIADUCT OF DOUBLE TRACK (3\*10=30) C1

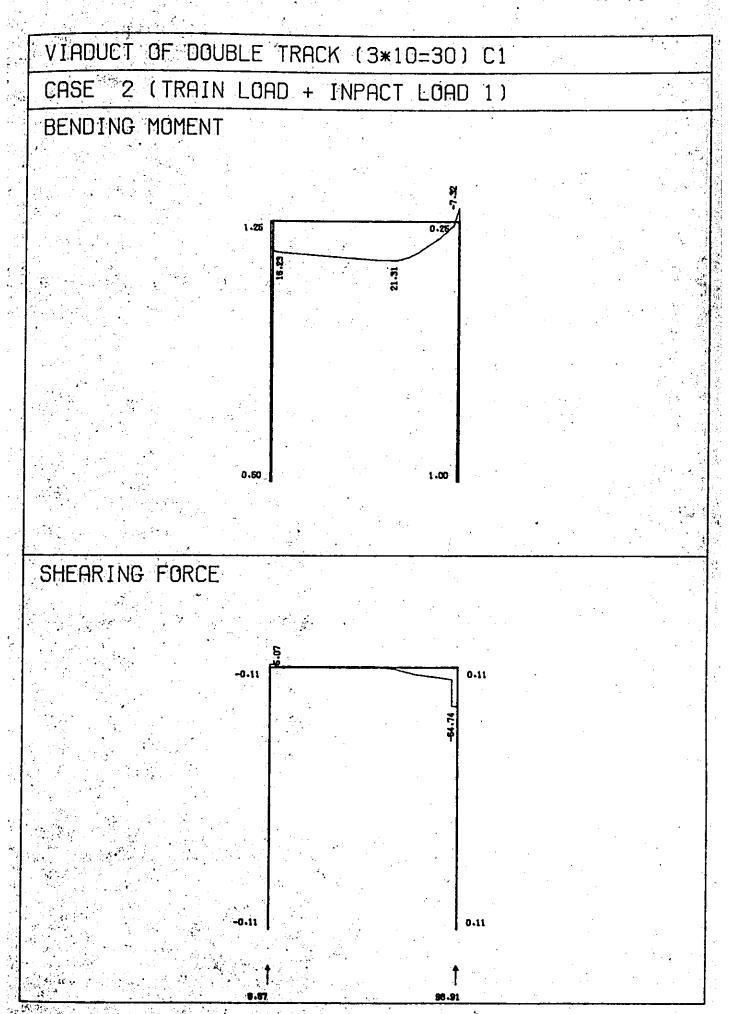
CASE 1 (DEAD LOAD)

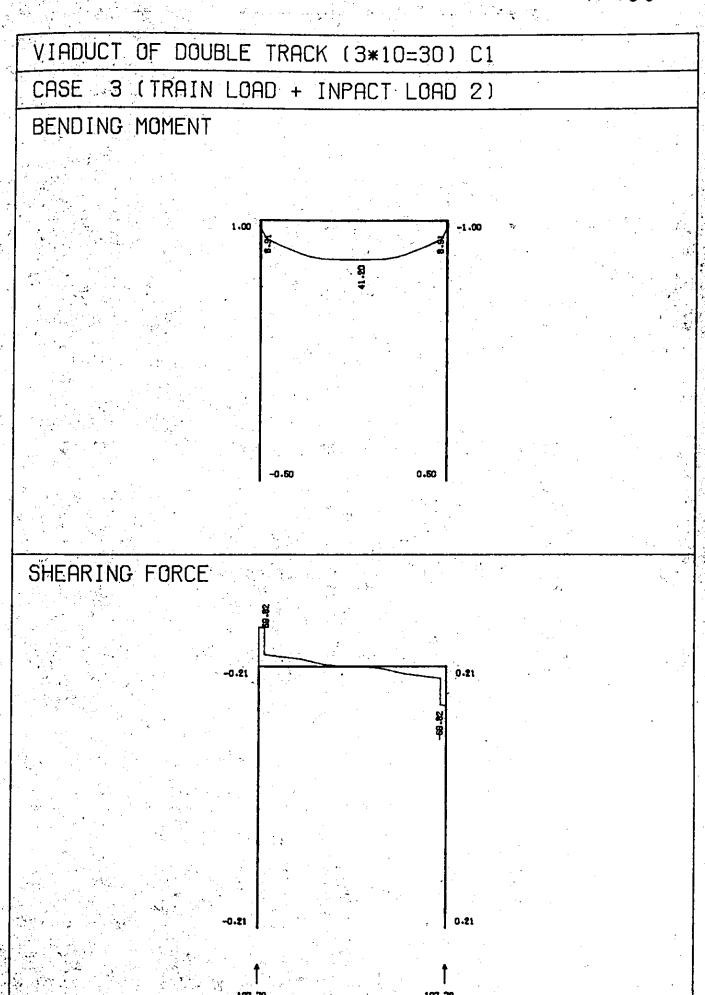
BENDING MOMENT

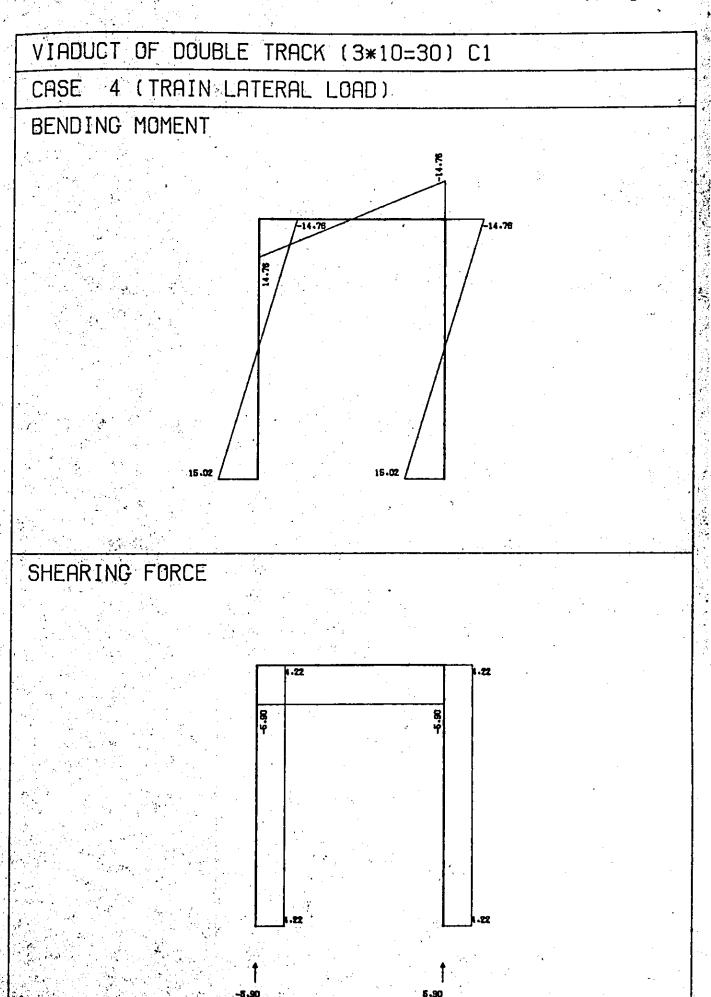


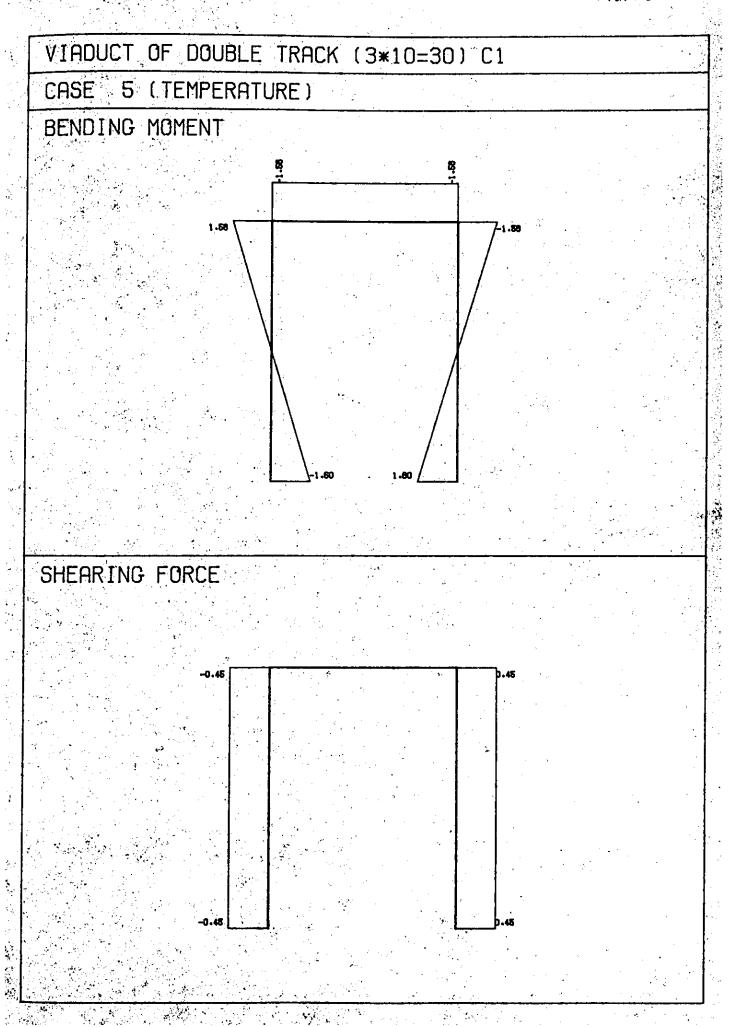




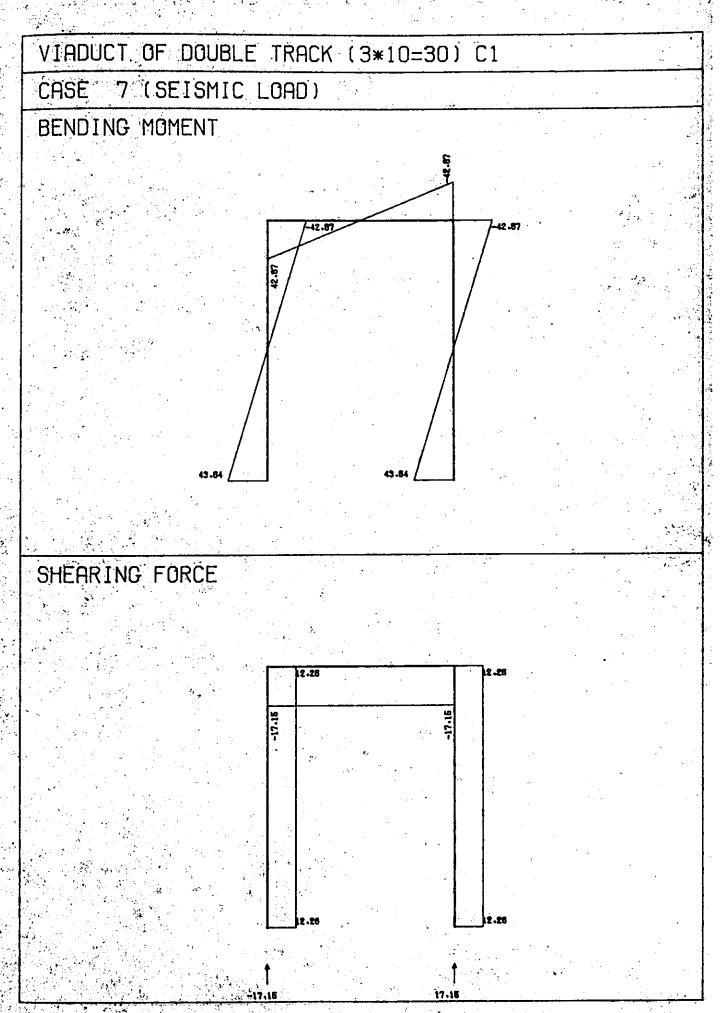


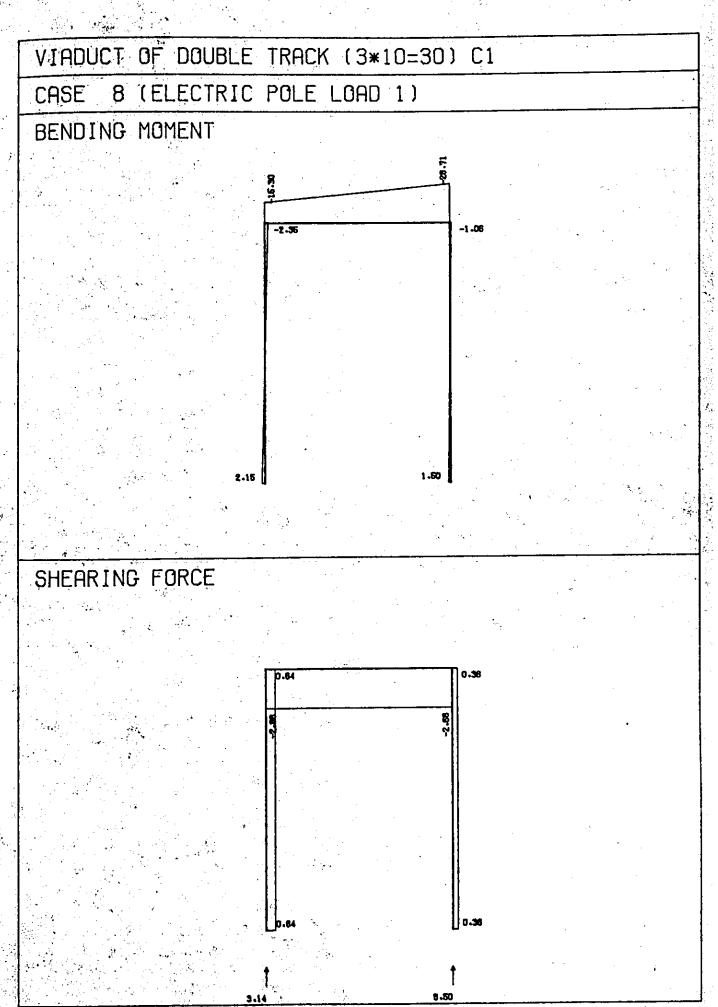


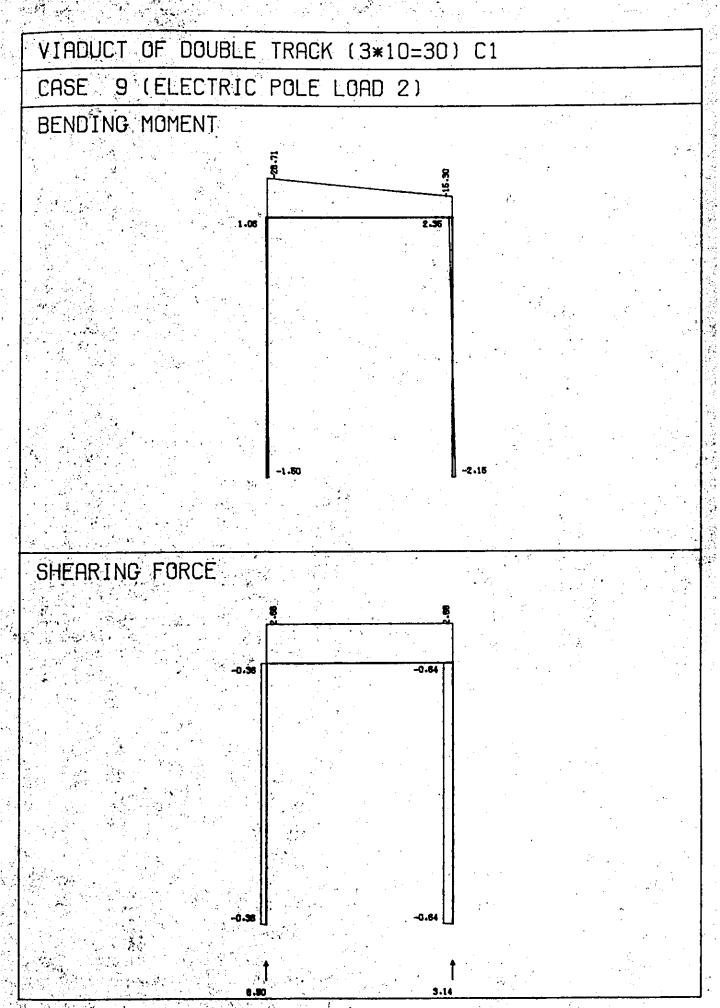




VIADUCT OF DOUBLE TRACK (3\*10=30) C1 CASE 6 (TEMPERATURE + SCHRINKAGE) BENDING MOMENT SHEARING FORCE







ETRACK (3*10=30) C1  20 20 22 22 26 26 27 28 28 29 29 29 29 29 29 29 29 20 29 29 20 29 20 29 20 29 20 29 20 29 20 29 20 29 20 29 20 29 20 29 20 29 20 29 20 29 20 29 20 29 20 29 20 29 20 29 20 20 29 20 20 20 20 20 20 20 20 20 20 20 20 20		0=8	+0+15 +4+85					1.14	-0.62 2.5	ĸ						-R-21 0.78	1.	N		и	
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(m)	LINEAR	-1.230	-1.230	0.00	0.00	
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TITLES. VIADUCT OF DOUBLE TRACK (3+10=30) C1

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σ	6.850		. 715	-74.106	075	or	4.850	÷	.715	-74.106	-•075
JTAN	5.000	101	-10.445	-74.693	520*-	JAN	2.000	( 10 )	-10.445	-74.693	520*-
MAK	2.500	( 10 )	19.621	000	075	HAX	2,500	( 10 )	19.821	000	075
HEMBER	2 (	1 - 3	3 (			= = MEMBER	2	1 - 3)	# D	•	•
11AN LIAN	0.000	( 10 )	.355	075	-116.253	ITAN	0.000	( 16 )	.355	675	-118.253
= = MEMBER	3 6	7 - 7	# 9 C			= = MEMBER	٠ ٣	2 - 4 )			
ITAN	0.000	( 10 )	355	.075	-118.253 -126.932	ITAN	0.000	( 10 )	355	. 075 670.	-118.253 -126.932

			SHEAR MAX	MAXIMUM				Ń	SHEAR MINIMUM		
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= = MEMBER	<b>.</b>	<b>.</b>	2 ) 6 =	ıı		= HEMBER		 	2 ) G		•
NAT	0.000	200	1 -10.44	•	075	ITAN	000.0	( 10 )	-10.445	74.693	075
พท	350	- C - C	3.563	15 14,656 33 13,805	075	NM	150		715	14.656	1075
3 W (	2.500	22	19.82		075	<b>3 In</b>	1.350	010	14.821	8.390	. 075
۰ ۸ ۵	മഹ	22	14.821		075	9	3,650		14.821	18.490 1000	675
5 6 7 8 E		225	715		075	. <b>20</b> GT	4.850	222	.715	-14.656	075
= MEMBER	2	# T	3 ) C =	= -(4,693	075	JIAN = HEMBER		( 10 )	-16.445	-74,693	075
ITAN	7.056	( 10 ( 10	355	5 - 075 8 - 675	-118.253 -126.932	ITAN	3.	~ ~	, ' <b>'</b>	075	-118,253
# # MEMBER	3 (	2	) C 4			= = MEHBER	3 6	- 2	# D C	-	
LTAN	0.000 7.056	( 10	1.355	.5 .075 .8 .075	-118,253 -126,932	I TAN JIAN	0.00C 7.056	( 10 )	355	. 075 . 075	-118.253

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		-CASE-	N	0				-CASE-	H	0	
= = MEMBER	-	r	216 ==	•		= = MEMBER	 	4 +4	2)6==	•	
ITAN	0.000	( 10 )	-10.445	74.693	075	ITAN	0.000	( 10 )	-10,445	76.643	a.
	.150	( 10 )	. 715	74.106	ı	•••!	.150	( 10 )	.715	74.106	
~	.150	( 10 )	. 715	14.656			.150	( 10 )	.715	14,656	•
m	.350	C 01	3,563	13.805	075	m	350	( 10 )	3.563	13,805	
<b>.</b>	1.350	( 01 )	14.821	8.390		4	1.350	( 10 )	14.821	8.396	
	2.500	( 10 )	19.821	000.	075	<b>I</b>	2.500	( 10 )	19.821	000	
<b>S</b>	3.650	( 01 )	14.821	-8.390		ພ	3,650	( 10 )	14.821	-8,395	
^	4.650	( 10 )	3, 553	-13.805	075	~	4.650	( 10 )	3,563	-13.805	
•••	4.850	( 70 )	. 715	-14.656		<b>6</b> 0	4.850	( 10 )	.715	-14.656	
σ	4.850	( 01 )	- 512	-74.106	•	<b>6</b>	4.850	100	.715	-74.106	
LTBN	5.000	( .10 )	-10.445	-74.693		JTAN	5.000	( 37 )	-16.445	-74.693	673-
# # MEMBER	2	t et	3 ) ( = =	•	:,	= HEMBER	2 2	। स	3 2 0 11		
LIAN	0.000	1000	.355	075	-118.253	ITAN	0.000	( 10 )	.355	675	-118,253
F # MEMBER	m m	2	* 3 0 7	-		= = MEMBER	3 6	.6	# " O		
TTAN UTAN	0.000	( 10 )	355	.075	-118.253	LTAN	7.056	( 16 )	355	.075	-118.253

HOMENT MINIMUM

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:	ITAN	0.00.0	1.	· ·	ี่นั	. 616	38,366	050		ITAN	0.000	( 20 )	-35.544	61.230	
	-4	150	1	_	2	. 460	118.953	250		-	.150	( 20 )	-26.388	50.838	
	N	.150	1	_	2	. 460	25,689	250		: N	. 150	( 20 )	-26.388	21,203	050
,	10	.350	1	-	ř	. 422	23, 721	250		מיז	350	02	-22.204	20.636	0.00
	3	1.350	-	2	53	259	17,913	268		· •	1.350	( 20 )	-3,266	17,025	0.00
	S.	2.500	7	~	61	. 021	000	288		5	2,500	( 20 )	13,215	11. 432	056
	ص	3,650	7	_	53	• 259	-17.913	288	•	م	3.650	( 14 )	-3.266	-17.025	050
	~	4.650	1	<u>-</u>	ž		-23,721	250		~	4.650	(19)	-22.204	-20.536	050
	•••	4.850	ĭ	_	5		-25.889	- 250		•	4.850	( 61 )	-26.388	-21, 203	050
	Ġ.	4.850	1	<b>~</b>	29		-118.953	250		6	4.850	( 19 )	-26.388	-66.838	25.1
	CTAM	5.000	⊼ ~	_	21	. 616	-38.366	050		JIAN	5.000	( 19 )	-35.544	-61.230	0.50
4	МАХ	2,500	( 12		61	. 021	000	-,288		HAX	2.500	( 20 )	13,215	11.432	050
μ	= MEMBER	2 (	+1	ا در	0	)) 11				: = MEMBER	2 (	·	3 ( 2		
,	ETAN	0.000	20	. 6	28 88	. 975	-8.224	-90.271		ITAN JAN	0.000	( 19 )	-28,343	8.123	-67.407
(1	: = HEHBER	Ð	~	4 <b>2</b>	ပ •	31 88			81	: = HEMBER	3	1 (2)	# D C 4		
-	HAR	0.000	- i	0.6	28. 29.	. 343	-8.123 8.224	-67.407 -96.057		I TAN NAL	0.000 7.056	( 19 )	-28.817 -28.975	8.224	-90.271 -73.193

SHEAR MINIMUM

TITLE. VIADUCT OF DOUBLE TRACK (3-10=30) C1

SHEAR HAXIHUM

		-CASE-		H	0	K	1		-CASE-	1144		X
= = HEMBER	(3), (4)	# #	2	ul uli uli			= = MEHBER	<del>-</del>	·	1 - 2) 6 = =		
ITAN	0.000	12	_	-1.534	144.512	288	201	. 60			*	
	.150	12	_	20.007	142,693	- 28.8				070977	20.250	35.0
~	.150	16	_	5. 337	36,154	1.250	ic	0 0 0	67	245-72	3/6-15	050
ריו	350	15		12, 352	33,986	25.	<b>U</b> P	176	2 2	245.72	-1.661	050
: : : : :	1.350	16		40.411	20,710	1.25.	7 4	2000		100.00	977-7-	050
ī	2.500	20	_	13.215	11.632		ru	1 * 6.70	67 1	270-67	->-839	050
ص	3.650	200	_	23. 128	9.4			2000		13.615	-11.432	050
~	4.650	2		26.054	100		0 +		. 14	40.411	-20.710	250
		ć			7,7,7	000		4.650	7.	12,352	-33,986	250
0 (	2000	2		75.7	1.001	050	æ	4.850	- 14	5.337	-36.154	-, 25E
r	ç	32	~	27.342	-37.974	0.50	ው	4.850	( 27 )	20.007	-142,543	288
JAN	5.000	20	_	21.616	-38,366	050	NATO	5.000	( 12 )	-1.534	-144,512	286
* = MEMBER	2 (	-	3	II II			= = MEMBER	> <	1 #	# . !! 		
JTAN	0.000	1 19	•	-28.343	8.123	-67.407	ITAN	0.000	( 20 )	26.817	-8.224	-90.271
= = MEMBER	n	2	3	# II			= HEMBER	E		= = 0 ( )	  -  -	
ITAN	0.000	( 19	•	-28.817	8.224	-90.271	ITAN	0.00.0	( 50 )	28,343	-8.123	-67.407

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ITAN	0.000	( 18 )	-5.648	64,953	.915	LTAN	0.000	( 11 )	-10.457	64.953	1.58
<b>-</b>	.150	18	1 4.057	64.442	•915	ਜ	. 150	( 17 )	752	54.447	
ا ہ	.150	18		12.744	.915	~	. 150	( 17 )	752	12.744	-458
	. 350		6.533	12,005	-915		.350	( 17.)	1.724	12.005	458
* Lf	2.00	91	16,324	7.296	- 915		1.350	17.1	11.514	7.296	854*-
ف ۱	3.650	2,5	16.32	7.296	4117	n u	386.2		15.862	300.	458
^	4.650	( 18 )		-12,005	.915		4.650	17.	1.724	12.005	004
<b>e</b> 0	4.850	- 18	4.057	-12.744	.915		4.850	( 17 )	752	-12.744	
ድ	4.850	( 18 )	1 4.057	-64.442	.915	ָּמַי	4.850	( 17 )	752	164.647	0 KG
JIAN	5.000	- 16	879.5-	-64,953	.915	JAN	5.000	( 11/ )	-10.457	-64.953	854*-
= = HEMBER	5 (	+	3) (2		•	= = MEMBER	2 (	1 - 3 )	H H	•	ı
TTAN	0.000 7.056	( 19 )	28.343	8.123	-67.407	ITAN	000°	( 12 )	1.354	288	-226.042
T = MEMBER	n	1 84	= = 0 < 7		-	= = MEMBER	3	2 - 4 1	# II	,	
LTAN	0.000	203	28.343	-8.123	-67.407	I TAN JIAN	0.000	(12)	-1.354	28 80 80 80 80 80 80 80 80 80 80 80 80 80	-226.042

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	ITAN	0.000	22	-	-25.744	72.011	.062	ITAN	0.000	2	_	-39.152	77,374	0.
٠.	+4 -4	.150	2	_	-14.986	71.424			. 150	2	2 .	-27.589	76.737	90 •
	۰.	.150	7	-	_	11.974		2	.150	2	_	-27.589	17.337	0.
	m	350	2	-	_	11.123		m	.350	~	_	-24.206	16,486	-
		1.350	7	_	-4.098	5,708		-	1.350	7	_	-10.266	11.071	0.
	r.	2.500	77	_	-2-182	-2.682		ın	2,500	<b>4</b>	~	-2.182	2.692	•
	•	3.650	25 -	_	860 - 1-	-5.708		•	3.650	2	_	-16.266	-11.D71	•
-	~	4.650	55	_	-12.675	-11.123		~	4.650	2	_	-24.266	-16.486	•
	€0	4.850	25	_	-14.985	-11.974	•	<b>40</b>	4.850	~	_	-27.589	-17,337	0.
	or	4.850	22	_	-14.986	-71:424		6	4.850	~	_	-27.589	-76.787	•
	MATO	5.000	1 22	_	-25.744	-72.011		NATO	2.000	2	_	-39.152	-77.374	0.
	PAX	2,778	22 )	1	-1.745	484	290*	XAH	2.500	( 22	<u> </u>	-2.182	2.682	•06
11	= MEMBER	2 (	. 1 ° . <del>∓1</del>	m	E = 3 (			= = MEMBER	ER 2 (	+	3.	n 11	:	
	ITAN JTAN	0.000	( 22 ( 21	~~	1.412	- 438	-126,754	ITAN	0.000	( 21	<b>~</b> ~	-1.996	-562	-121.39 -135.43
11	= MEHBER	2	2 +	4	= = 0 C			= = MEHBER	ER 3 (	2	4	11 11		
	NATI	0.000	22 )		1.996	562	-121.391	NATI	0.000	( 21	~~	-1.412	4.38	-126.75
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ITAH	0.000	( 22	_	-39.15	!	!	<b>:</b>	TITAN .	0.000	21.1	-25.74	73 044	
e4,6	150	2	-			1		44	. 150	77	14.98		
<b>\</b>	150	25						2	.150	( 22 )	-14.98		
)  -4	1.350	2,0	- -	100.00	ļ			m.	350	( 23 )	-12.67		
in,	2.500	22	. ~	-2.182	2.682	000	٠.	or W	1.350	( 27 )	860-1-	5.708	290*
ع	3.650	( 22	_	٠.			•	ص د	3,650	55	140.75	1	:
► 6	650	25		۸.			•	~	4.650	( 21 )	-24.20		
0 0	7.0	22	۰.			•		€0	4.850	( 21 )	-27.58	1	i -
		2 :	^	-14, 966	;	,		σ,	4.850	(21)	-27.58		
•	2-000	22 1	-	•				JIAN	5.000	( 51 )	-35.15		
= MEHBER	2 (	, <del>-1</del>	m	"   O			- 10	= = HEHBER	2		3 1 6		
I TAN JTAN	0.000	27 51 51		-1.996 1.970	562 0 .562	-121.391		ITAN	0.000°	( 22 )	1.412	864.1 864.1	-126.754
= HEMBER		2	3	3 (	ţi			= MEMBER	e m	2			
ITAN JIAN	0.000	12 2		-1.412	2 .438	-126.754	,	LIAN	0.000	( 22 )	1.996	- 562	-121,391

AXIAL MINIMUM

TITLE.. VIADUCT OF DOUBLE TRACK (3-10=30) C1

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ITAN	000-0	12 )	्रा <sup>र</sup> <del>-</del> -	25.744	72,011	:	ITAN	0.000	( 22 )	-39.152	77.374	
, <b>-</b> -1	.150	12	_	-14.986	71.424	. 1162	-1	.150	( 22 )	-27.589	76.787	
N	.150	72		14.986	11.974		~	.150	( 22 )	-27.589	17.337	:
,PO	.350	<b>₹</b>	-	12,675	11.123		<b>m</b>	.350	( 22 )	-24.206	16.486	
•	1,350	, , ,	_	860.4-	5.708		-	1.350	( 22 )	-10.266	11.071	:
ľ	2.500	77	_	-2.182	-2.682		, Lo	2.500	( 22 )	-2.182	2.682	
عب	3.650	- 21	. 1	10.266	-11.071		<b>م</b> ب	3.650	( 22 )	960.4-		
~	4.650	<u>.</u> 21	_	24.206	-16.486		^	4.650	( 22 )	-12,675		
, eco	4.850	12		27.589	-17,337		<b>60</b>	4.650	( 22 )	-14.936		
6	4.850	12	_	27.589	-76.787		o	4.850	( 25 )	-14.986		
JIAN	5.000	( 21	-	39, 152	-77-374		NATO	5.000	( 52 )	-25:744	-72,011	1062
= = MEMBER	5 , 2	. । . श्ल	m	, II II	e	•	≥ ≤ HEHBER	2	-	3 ) ( ) = =		
NATI	0.000	22		-1.996	562	-121.391	ITAN	0.000	( 22 )	1.412	82 85 87 87 87 87 87 87 87 87 87 87 87 87 87	-126,754
= = MEMBER			-	# # II			= = MEMBER		. 1	11 12 12 12 12 12 12 12 12 12 12 12 12 1	·· .	
ITAN	0.000	22 3		1.996	562	-121.391	NALL	0.000	( 21 )	-1.412	.438	-126.754

TITLE . VIADUCT OF DOUBLE TRACK (3\*10=30) C1

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= = MEMBER	Н	1 - 2	H 11 9 4			= = HEHBER	.R 1 (	- 1 	2)(2	r.	•
ITAN	0.000	( 35 )	. •	36,578	.041	Z	u fino	35 )			
<b>,</b>	.150	( 32 )	16.874	36,186	.041	+		36	000.59	070.00	
<b>~</b> 1	.150	( 32 )	16.	-3. 449	100.		150	, w	662.64	20.00	:
<b>m</b>	.350	( 28 )	•	21.389	131		350	36	1 -40.717	72.42	
<b>#</b> 1	1.350	( 28 )	. •	8.113	131	<b>3</b>	1:350	36	1000	7 - 8 -	,
٠,	005.2	12 )	٠	-2.682	150	S.	2.500	31	-3.271	7.33	•
۰ ۵	3.650	- BE	•	-8:113	131	•	3.650	32	16,001	-1 A H 3	,
٠,٠	4.650	20.		-21.389	131	_	4.650	35	-40.717	-22.423	
<b>3</b> 0 (	4.850	32 )	. •	3.449	. 041	eu	4.850	1 11	0.7.7.4	22 004	
D ;	4.850	( 36 )	•	-36.186	.041	0	4.850	35	165,250	162,625	7 7 7 7
NA L	5.000	( 36 )	.•	-36.578	.041	NATU	5.000	(35)	-54.663	-63.018	. 041
*A*	2.778	( 56 )	39.454	181	150	MAX	2.500	( 31 )	-3,271	-2,332	-,336
= = MEMBER	2 (		#  			A = MEMBER	27	1 +1	3 C		
ITAN	0.000	( 36 )	30.407	8.466	-95.939	NAT D	0.000 7.056	(35)	-29,911 -30,213	6.546 -6.466	-69.500
= = MEMBER	m	- 2	# 3 C			H = MEMBER	3 (	2	= = 0 ( +		
HTÄN JTAN	0.000	(36)	29,911 30,213	-8,548 8,466	-69.500 -101.725	NATI	0.000 7.056	(35)	-29,521	8,466 -5,548	-95.939

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ITAN	0.000	1.26	30.240		150	TAN 0.00	9	7 92	217 11	76 670	
<b>,-1</b>	.150	92	1 -8.297					, r	724 71	20.70	
~	.150	8	19.277	į	4 1			, <u>.</u>	10.01	001.00	
'n	.350	 8	1 -11, 795			i in	25	, u	+ C + C +		7.0
7	1,350	ි ස	18,596				-	, ,	77.0	7 626	
Į,	2.500	36	1.455					· ·	11.655	11 220	7.0
uo	3.650	 9.	10,415		•	(C)	, L	. ~	8 .000	073.52	7 7 7
	4.650	36	1 16.128		٠	1914	200		11.705	126.426	7070
•••	4.850	36	16.874				25	2 6	10.077		1.131
6	4.850	36	16.874			) ( ) ( ) ( ) ( ) ( ) ( ) ( ) ( ) ( ) (		200	- B 207	30.400	TC T *
KATO	5.000	36	11.416	-36,578	.041	JIAN 5.000	20	24 )	-36.240	-147,193	150
= = MEMBER	2	. । च	3 C M			= MEMBER 2	_	. #4	# U		
ITAN	0.000	35	1 -29.911	60 40 40 40 40 40 40 40 40 40 40 40 40 40	-69.500	ITAN C. DOO	- •	35	un a	-B • 455	626*56-
= = MEMBER	, n	}. I	# # D C #		007*	18E.R		- 4 - 1 - 1	-30,233	0 3 3 0	-101.725
ITAN	000.0	35	123.521		-95.939			, de	29,911	8.50	.69,500
JAN	7.056	32	30.213	8.466	-101.725	JTAN 7.056		( 36 )	-30.407	8.548	-75.286

PICK UP '4

			CAS	-CASE-	H	0	X			-CA SE-	H		X
n	: = MEMBER	<b>→</b>	-	~	11 19 ~		÷	= = MEMBER	7	· I	= = 9 ( 2		
	ITAN	0.00.0	35	-	-16.952	62.621	1.035	TAN	0.000	333	164.25	796 29	9 P
	-	.150	35	_	-9.597	62,110	1.035	· •	150	7 7 3	725 366	7074	9 0
	N	.150	35	~	-9.596	10.413	1.035		25.		345.35	120	0000
•	m	.350	35	-	-7.587	9.673	1.035	1 1-3	5		22.122	926 94	0000
		1.350	35	_	128	4.964	1.035		1.350	33	110	0 627	0 d
	ĸ	2.500	35	~	1.538	-2.332	1.035	'n	2.500	333	13.271	200	
	9	3.650	35	_	-5.492	-9.627	1.035	ع	3.650	333	1 0 . J	1 1 1 1	9 6
	<b>~</b>	4.650	35	~	-17.614	-14.336	1.035	~	4.650	334	-12.396	-0 673	1 1
	<b>6</b> 0	4.850	32	-	-20,556	-15.076	1.035	•	4.850	133	114.406	7 4 5	0000
	σ.	4.850	35	_	-20,556	-66.774	1.035	o	4.850	25	14.405	211	1
	JTAN	5.000	35	_	-30.611	-67.285	1.035	JTAN	5.000	( 33 )	-23.761	-62.621	38.6
u	= HEMBER	5 2	+	m	# II 3 C	•	:	= # MEMBER	5 (		3) ( = =		
	ITAN	0.000	32	-	-29.911	8.548	-69.500	ITAN	0000	( 26 )	2.410	650	-234,543
	UT A R	7.056	೫ -	_	30.407	8.548	-75.285	JAN	7.056	( 92 )	-2.177	650	-243.222
ti	: = MEMBER	E	2		== 0(			= = MEMBER	m.	ı N	= = 0 ( 5		
	ITAN	0.000	3	-	29.911	-6.548	-69,500	ITAN	0.000	( 54 )		.650	-734.543
	UTA₹	7.056	36	_	-30.407	-8.548	-75.286	ZYLT	7.056	24.7	2.177	.650	277.7.2

TITLE..VIABUCT OF DOUBLE TRACK (3.10=30) C1

		-	Š.	MOMENT MAXIMUM	£.					HOMENT MINIHUM	Ę	
		-CASE-	.	H	D	N			-CASE-	·k	2	N
= = MEMBER			~ 2	1 9 C			# = MEMBER	, ,	+	1-216==		
ITAN	0.000	( 19	•	21,616	35,366	050	ITAN	0.000	( 36 )	-54.683	63,018	770
<b></b> 1	150	, , ,	: . _}	29.460		250	-1	. 150	36 )	-45,259	62,626	
N	150	<b>.</b>		29.480		1	2	150	36	-45.259	22,991	
י	320	31 );	_	34.422	. ;	,	<b>.</b>	.350	(36)	-40.717	22,423	40
<b>.</b>	1.350	25	_	53,259		-288		1:350	( 36 )	-19.991	18.813	0
ın (	2,500	22	_	61-021	000.	288	ķ	2,500	(31)	-3.271	-2,332	338
ِ م	3.650	75	_	53,259		i	•	3,650	( .35 )	-19.991	-18.813	0.7
<b>~</b> ,	4.650	16.1	_	34.422		250	~	4.550	72	-40.717	-22.423	
æ	4.850	16.	_	29.460			eC)	4.850	35	145.250	-22,001	
S	4.850	15	_	29.460	-118,953	250	o or	4.850	32	652.54-	150.501	
STAN	5.000	6 20		21.616			JAN	5,000	( 35 )	-54.683	-63.018	.041
T A X	2.500	( 12 )	_	61.021	000	288	MAX	2.500	( 31 )	-3.271	-2,332	-,338
= = MEMBER	5 (	· .	~ ~	H		•	= = ME4BER	2	-	3) C = =		
ITAN	0.000	~	_	29,521		-95,939	ITAN	0.000	1 36 )	-29.911	10 10 10	000
JTAN	7.056	( 35 )		30.407	8.548	-75.286	JIAN	7.056	( 36 )	-36.213	-6.456	-101.725
# = MEMBER	3	CI	3	" "			= MEMBER	3	- 2	== 0 ( +		
ITAN	0.00.0	( 36 )	_	29.911	-8.548	-69.500		0.000	( 35 )	-29,521	6.465	-95,939
UTAN	7.056	35	_	30.213		-101-725	JTAN	7.056	(36)	-3E -407	-8.548	-75.286

	7	-CASE-	Н	0	N	i	]	-CASE-		0	3
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ITAN	0.000	( 56 )	-30.240	147,193	150		6	, •			
<b></b> 4	.150	( 52 )	-8.297	145,375		24-	0000	٠.	11.416	36.578	:
N	.150	30 >	-19.277	38.686		4	150	٠	16.874	36.186	
M	.350	( 30 )	-11, 795	36.318	1011	N P	. 150	. 35	16.874	-3.449	
3	1.350	( 30 )	18,596	23.042	12.1		0.55	35.)	16,128	-4.016	
r.	2.500	( 36 )	-1.455	13.220	170	7 (	1.350	35 )	10.415	-7.626	:
œ	3.650	(36)	10.415	7.626		n u	005-7	( 25 )	-1.455	-13.220	
_	4.650	( 36 )	16,128	4.016			0.00.0	( 82 )	18.596	-23.042	:
<b>6</b> 0	4.850	( 36 )	16.874	633	1 1 2		4.650	( 58 )	-11.795	-36.318	
or or	4.850	( 36 )	16.874	-36.186	1 2	<b>50 (</b>	4.850	( 28 )	-19.277	-38.486	
JTAN	2.000	( 36 )	11.416	-36.578	10.1	NAT.	4 - 85 B	25.	762-8-	-145.375	150
.								,	0+2-06-	14/-193	
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MTAN	0.000 7.056	( 35 )	30,407	8.548 8.548	-69.500	LIAN	0.000	(36)	29.521	-8.466 -8.466	-95.939
= MEMBER	٠ ٣	2 - 2	= D		:	= MEMBER	3 (	- 2	= = O		
ITAN	0.000 7.056	(35)	-29.521 30.213	8.466 8.466	-95.939 -101.725	N T T N N N N N N N N N N N N N N N N N	0.000	( 36 )	29.911	20 E	-69.500

PICK UP		1 - 1 - 1 - 1 - 1 - 1 - 1 - 1 - 1 - 1 -		*:						•	
		AXIAL	IAL HEXTHUM	r	•			AX	AXIAL HINIMUM	· :	
	1-1-	-CASE	H	0		1		-CASE-		0	N
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TTAN	0.000	( 35 )	-18.952	62,621	1.035	ITAN	0.000,0	( 52 )	-10.457	64.953	458
	.150	( 35 )	765.6-	62.110	1.035	•••	150	( 11 )	752	64.442	1.458
~	.150	( 35 )	-9.596	10.413	1.035	~	. 150	( 17 )	752	12.744	- 458
m	.350	( 35 )	-7.587	9.673	1.035	m	.350	( 17 )	1.724	12,005	- 458
	1.350	(35)	-128	4.964	1.035	3	1.350	( 11 )	11.514	7.296	4.58
ìn	2.500	( 35 )	1.538	-2,332	1.035	w.	2.500	( 17 )	15.852	250.	1.58
	3.650	( 35 )	-5. 492	-9.627	1.035	9	3.650	( 17 )	11.514	-7.296	1.458
	4.550	( 35 )	-17.614	-14.336	1.035	_	4.650	( 17 )	1.724	-12.005	458
. es	4.850	2	-20.556	-15.076	1.035	0	4.850	( 17 )	752	-12.744	458
o	A 50		-20.556	-66-774	1.035	σ	4.850	( 11 )	752	244.49-	1.458
JTAN	5.000	32 )	-30.611	-67.285	1.035	UTAN	9.000	( 17 )	-10-457	-64.953	154
= = MEMBER	2		II II		. !	= = MEMBER	2 (	1 1	H		
ITAN	0.000	(19)	-28.343	8.123 8.123	-67.407	ITAN	0.000 7.056	( 56 )	2.410	1.650	-234.543
= = MEMBER	n	1 2	in 11			= = MEMBER	3 (	2 - 4	= = 0		7
ITAN	0.000	( 20 )	28,343	-8.123	-67.407	ITEN	6.000 7.056	( 32 )	-2.410	.650 .650	-234.543

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HOMENY MAXIMUM		,	. · ·		•	***				/ 			
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TAN 0.000 ( 29) 14.783 -4.06 -115.693 -156	•		-CASE-		0	N			-CASE		2	N	
0.000 (13) 17.665 64.233156 11AN	= = MEMBER			2.16 ==			= = NENBER	7 (		# # O			
150 (14.) 29.460 118.953250 2 150 (29) -21.275 21.543 21.550 (14.) 29.460 25.889250 3 .150 (29) -21.275 21.543 21.550 (14.) 34.450 25.889250 4 1.350 (29) 1.550 16.003 16.509	ITAN	0.000	( 13 )	17.865	64,233	158	ITAN	0.00	( 30.)	-39.128	133.132	131	
-150 ( 14 ) 29,460 25,889250 3 .350 ( 29 ) -21,275 21,843	-4	.150	11.	29.460	118.953			150	( 59 )	-21:275	76.319	1 60	
.350 (14) 34,422 23,721250 3 .350 (29) -17,039 20,803 15,094 25,535 17,913288 5 .258 6 .1,350 (29) 1,550 16,094 25,550 (12) 53,259 17,913288 5 .3,650 (27) 2,217 -21,708 2,550 (15) 34,422 -23,721250 7 4,650 (27) 2,217 -21,708 1,650 (16) 29,460 -25,889 -2550 9 4,850 (27) -34,055 -127,138 1,7152 11,498 -120,535 -250 9 18,050 (27) -34,055 -127,138 1,7152 11,498 -120,535 -250 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0	~	.150	7.47	29.460	25.889	. • :	~	. 150	( 29 )	-21.275	21.543	- 038	
1.350 (12) 53.259 17.913288	m	.350	37	34.422	23.721		m	.350	( 52 )	-17.039	20.803	- 038	
2.500 (12) 61,021 .000288 5 2.50 (27) 16.616 -6.130 3.650 (12) 53.259 -17.913288 6 3.650 (27) 2.217 -21.708 1.650 (12) 2.217 -21.708 1.650 (12) 2.217 -21.708 1.650 (16) 24.422 -23.721250 8 4.650 (27) -24.055 -27.138 1.7152 1.650 (16) 24.460 -118.953250 9 4.650 (27) -34.055 -127.138 1.700 (16) 11.498 -120.535250 0.000 (27) -53.245 -128.138 1.600288 0.000 (27) -53.245 -128.138 1.600288 0.000 (28) -13.699 3.974 -19 0.000 (29) 15.143 -4.143 -123.075 1.000 (28) -14.955 -4.235 -21 0.000 (29) 14.783 -4.066 -115.693 0.000 (28) -14.955 -4.235 -20 0.000 (29) 14.783 -4.066 -115.693 0.000 (28) -14.957 -4.357 -20 0.000 (29) 14.783 -208.922 0.000 (28) -14.341 -3.474 -2.35 0.000 (28) -14.341 -3.474 -2.35 0.000 (28) -14.341 -3.474 -2.35 0.000 (28) -14.341 -3.474 -2.35 0.000 (28) -14.341 -3.474 -3.474 -2.35 0.000 (28) -14.341 -3.474 -3.474 -2.35 0.000 (28) -14.341 -3.474 -2.35 0.000 (28) -14.341 -3.474 -3.474 -2.35 0.000 (28) -14.341 -3.474 -2.35 0.000 (28) -14.341 -3.474 -3.474 -2.341 -3.474 -3.4	3	1.350	727	53.259	17.913	i	* ***	1.350	1 62 )	1.550	16.094	038	
3.650 ( 12 ) 53.259 -17.913286	Ţ,	2.500	( 75 )	61.021	000.		ī.	2.500	( 22 )	16.616	-6.131	B 17.	
4.650 ( 16 ) $34.422 -23.721250                                    $	٠	3.650	( 15 )	53.259	-17.913	ľ	<b>ع</b> .	3.650	( 22 )	2.217	-21.768	0.38	
4.850 (16) 29.460 -25.889250 8 4.850 (27) -34.055 -127.138 4.850 (16) 29.460 -118.953250 JIAN 5.000 (27) -34.055 -127.138 5.000 (16) 11.498 -120.535250 JIAN 2.500 (27) -53.245 -126.719 2.500 (12) 61.021 .000288 MAX 2.500 (27) 16.016 -6.130 2 ( 1 - 3 ) C = = HEMBER 2 ( 1 - 3 ) C = = 0.000 (29) 15.143 -4.143 -123.075 JIAN 7.056 (30) -14.955 -4.235 -21 3 ( 2 - 4 ) C = = HEMBER 3 ( 2 - 4 ) C = = 0.000 (29) 14.783 -4.066 -115.693 JIAN 7.056 (30) -14.957 4.235 -20 7.056 (27) 15.388 4.143 -208.922 JIAN 7.056 (36) -14.341 -3.974 -220	~	4.650	191	34.422	-23.721	250	<b>2</b>	4.650	( 27 )	-26.841	-34.984	038	
4.850 (16) 24.460 -118.953250 JIAN 5.000 (27) -34.055 -127.138 5.000 (16) 11.498 -120.535250 JIAN 5.000 (27) -53.245 -126.719 2.500 (12) 51.021 .000288 MAX 2.500 (27) 16.016 -6.130 2.500 (12) 15.143 -4.143 -123.075 JIAN 0.000 (28) -13.699 3.974 -19 7.056 (27) 15.208 4.143 -208.922 JIAN 0.000 (28) -14.927 4.235 -20 7.056 (27) 14.783 -4.066 -125.446 JIAN 0.000 (28) -14.927 4.235 -20 7.056 (27) 15.388 4.143 -208.922 JIAN 7.056 (30) -14.341 -3.974 -20 7.056 (27) 15.388 4.143 -208.922	<b>RO</b>	4.850	( 16 )	29.460	-25.885	250	æ	4.650	( 27. )	-34.055	-37.152	038	
5.000 ( 16 ) 11.496 -120.535250 JIAN 5.000 ( 27 ) -53.245 -128.719  2.500 ( 12 ) 61.021 .000288 MAX 2.500 ( 27 ) 16.016 -6.130  2 ( 1 - 3 ) C =	σ.	4.850	( 16 )	24.460	-118.953	250	<b>ው</b>	4,850	( 22 )	-34.055	-127.138	138	
2.500 (12) 61.021 .000286 MAX 2.500 (27) 16.016 -6.130 2 ( 1-3) C = =	LTAN	5.000	16 1	11.498	-120,535	250	NATO	5.000	( 22 )	-53.245	-126.719	038	
2 ( 1 - 3 ) C = = = = HEMBER 2 ( 1 - 3 ) C = =	HAK	2.500	. 12 1		000	288	МАХ	2.500	( 22 )	16.016	-6.130	038	
ITAN 0.000 (29) 15.143 -4.143 -123.075 ITAN 0.000 (28) -13.699 3.974  JAN 7.056 (27) 15.208 4.066 -115.693 JIAN 7.056 (30) -14.955 -4.235  = HEMBER 3 ( 2 - 4 ) C = = HEMBER 3 ( 2 - 4 ) C = =  ITAN 0.000 (29) 14.783 -4.066 -186.446 ITAN 0.000 (28) -14.927 4.235  JIAN 7.056 (27) 15.388 4.143 -208.922 JIAN 7.056 (36) -14.341 -3.974	= = HEHBER	. <del>.</del> 2	· •				= = HEMBER	2 (	1 - 4				
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0.000 (29) 14.783 -4.066 -186.446 ITAN 0.000 (28) -14.927 4.235 7.056 (27) 15.388 4.143 -208.922 JIAN 7.056 (36) -14.341 -3.974		٠ ٣	- 2	11		٠,	= = MEMBER	3 6	2 - 4	Ħ			
	TAN	0.000	( 29 )	14.783	-4.066	-185.446	ITAN	0.000	( 28 )	-14.927	4.235	-203.092	

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ITAN	0.000	( 56 )	-30.240	147,193	-, 150	ITAN	0.000	( 27 )	4.56.1	64 911	
• <b>-1</b> ∶	150	. 92	-8.297	145,375	-,150	+	150	( 22 )	408.5	100.	
N I	.150	30 >	-19.277	38.486	!	~	150	( 27.)	14.818	6.644	
, ,	350	000	-11.795	36,318	•	<b>M</b>	.350	( 27 )	15,058	12.8.2	
<b>*</b> 1	1.350	000	18.596	23.042		3	1,350	( 27 )	18.718	1.165	200
١ ٨	2,500	52	16.016	8.798		ı,	2.500	( 28 )	33,930	-7-454	
رع	3.650	62 )	19.385	-6.779		<b>.</b>	3.650	( 28.)	18,596	-23.042	124
_	4.650	1 59 1	5.256	-20.055	038		4.650	( 28 )	-11,795	45.45	1010
<b>=</b> 0	4.850	( 53 )	1.028	-22-23	038	. <b>cc</b> )	4.850	28.	-16.277	200	
σ,	4.850	( 53 )	1.027	-112-209	038	5	4.850	24.	-8.237	1165 275	
JTAN	5.000	( 62 )	-15,923	-113.791	- 038	UTAN	5.000	( 54 )	-30.240	-147,193	150
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TAN	0.000	1 22 1	-13.483	4.866	-108 146	Z Z	=,	(30)		4.235	-209,092
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JIAN	0.000	( 28 )	-14,927	4.235	-209.092	ITAN JAAN	7.056		14.783	-4.066	-186,446

TITLE..VIADUCT OF DOUBLE TRACK (3.10=30) C1
PICK UP 6
AXIAL HAXIHUM

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			-CAS	i i		0			J	-CASE-			X
10 11	HEMBER	<u>۔</u> 1	- -	~	11 12 2			= = HEMBER	7 1 (	1 - 2	# B 9 6		
11	AN	0.000	. 27	•	4.561	61,901	038	ITAN	0.000	( 12 )	-1.534	144,512	288
. 4	-	.150	. ( 27	<u>.</u>	13.808	61.390	038	<b>-</b>	150	( 12 )	20.007	142.693	2000
	.2	.150	12	~	13.808	6.614	038	2	150	( 12 )	20.007	35.673	288
	m	.350	2	_	15.058	5.874	038	m	.350	( 27 )	26.834	33,180	-258
	•	1.350	2	-	16.718	1.165	036	•	1.350	( 12 )	53.259	17,913	288
	'n	2.500	( 23	~	16.016	-6.130	038	2	2.506	7	61,621	909.	288
	9	3.650	<b>2</b>	•	2.217	-21.708	038	ۍ	3,650	7	53,259	-17.913	288
	_	4.650	2	_	-26.841	-34.984	038	~	4.650	( 15 )	26.894	-33,180	288
	••0	4.850	<u>ک</u>	_	-34,055	-37,152	038	•	4.850	~	20.002	-35.673	288
	6	4.850	2	<u>-</u>	-34.055	-127.138	038	σ.	4.850	7	26.607	-142,693	-288
5	אארט	5.000	≈ 	- -	-51.245	-128.719	038	UIAN	5.000	( 21 )	-1.534	-144.512	288
. H H	MEMBER	) 2	, H	'n	H 11 2) C			= = MEHBER	1 2 8	1 - 3	H 3	•	
<u>,</u>	ITAN	0.000	133		-11:439	3,512	-105-417	NATE	0000	( 26 )	2.410	650	-234,543
5	r *		-	-	740 07	376.00	*06.977	2		0	-2.17	# CQ*-	777.5.47-
II ( 11	MEHBER	m	2	<b>3</b>	# 	; ;		= = MEMBER	m m	3 - 2	# # <b>3</b> C		•
===	LTAN	0.000			12.738	-3.512	-183.717	NATI	0.000	( 57 )	-2.410	.650	-234.543

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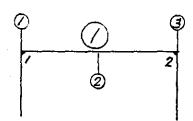
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		-CASE-	H	0				CASE-	4		
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ITAN	0.000	( 19 )	21.616	36,366		ITAN	0.000	( 36 )	-54.683	63.018	F 40.
-	.150	( 49 )	27.342	37.974	;	4	156	(36)	-45,259	62.626	170
~	.150	( 16.)	27.342	-1.661	1	2	150	( 36 )	-45.259	22,991	. 141
m	.350	( 19 )	26.954	-2.228	i	m	350	( 36.)	-40.717	22, 423	170.
<b>.</b>		( 13 )	23.028	-5.839	050	3	1.350	(36)	-19,991	18.813	1.40
<b>.</b>	2.500	( 19 )	13.215	-11:432	i	\$	2.500	(36)	-1.455	13.22£	.041
۵	3.650	( <b>2</b> 2 )	23.028	5.839		٠	3.650	( 32 )	-19.991	-18.813	0.41
~	4.650	( 20 )	26.954	2.228	050	~	4.650	( 35 )	-40.717	-22,423	1711
•	4.850	( 50 )	27.342	1.661		φ	4.650	( 35 )	-45.259	-22-991	. 140
\$	4.850	( 50.)	27.342	-37,974	•	္	4.850	35 )	-45,259	-62,626	170
UTAR	5.000	( 02 )	21.616	-38.366	- G -	JAN		(32)	-54.683	-63.018	.041
MAX	4.850	( 50 )	27.342	1,661	050	MAX	2.500	( 36 )	-1.455	13.220	.041
= = MEMBER	1 2 . 5	+ - 10	11 (t-			= = MEMBER	BER 2 (	1 - 3	= = 0 (		
ITAN	0.000	(36)	29,521	8,456	-95.939 -75.286	ITAN	0.000	( 35 )	-29,911	6,546	-69.500
= = MEMBER	3 (	- tr	# # D	. `	•	= = MEMBER	BER 3 (	2 - 4	= = 0 (		
LTAN	0.000	(36)	29.911	8,5,65	-69.500 -101.725	LIAN	0.000	(35)	-29,521	8,466	-95.939

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-,	= = MEMBER	) #	। स	2.7	0	ं भा ,स	:	· ·	• •	= = MEHBER		1 - 2	E II 9 (			
4 ;	NATI	0000	_		-54	. 683	63.018	.041		ITAN	0.000	( 35 )	11.416	36.578	.041	
	<b>.</b>	150	-		-15	259	62.626	.041	-	4	.150	( 32 )	16.874	36,186	140.	
	N	150	36	_	15.2	528	22,991	.041	•	N	.150	(32)	15.874	-3.449	140	
1	m	350	٠,		7	717	22-423	041	;	m	350	(32)	16-128	-4.016	.041	
	<b>.</b>	1.350		_	13	991	18.813	.041		<b>.</b>	1.350	(32)	10.415	-7.626	0.1	
	w,	2.500	<b>~</b>	_	Ţ	4.55	13.220	. 041		2	2.500	( 32 )	-1.455	-13.220	.041	
	ع	3.650	1 36	<b>-</b>	10.	415	7.626	. 041	٠	9	3.650	(32)	-19.991	-16.813	041	
		4.650	36	_	16	. 128	4.016	.041		~	4.650	( 32 )	-46.717	-22.423	140.	
	<b>KO</b>	4.850	36	_	15.	,874	3.449	. 041		80	4.850	(32)	-45.259	-22.991	10.	
	or	4.850	36		16.	428	-36.186	. 041		G*	4.850	( 35 )	-45.259	-62.626	.041	
	JIAN	5.000	36	_	=	416	-36.578	.041		UTAN	5.000	(32)	-54-683	-63.016	.041	
•	= HEMBER	2 .		ю.	O <sub>1</sub>	11				= = HEHBER	2 (	1 3	= = O C		٠	
	ITAN	0.000	35	:	-29	911	8.548	-69.500		ITAN	0.000	( 36 )	25,521	-8.466	-95,939	
	JAN	7.056	32		30	20.407	8.548	-75.286		JAN	7.056	( 36 )	-36-213	-8.465	-121.725	
	= = HEHBER	3 (	2	G	O	() ()				= = HEHBER	3 (	2 - 4	) E = 0			
•	ITAN	0.000	( 35		-29	29.521	334.8	-95.939	,	NATI	0.000	(36)	29.911	18.548	-69.500	

- (6) Calculation of upper beam
  - 1. Stress calculation of upper beam

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- (1) The case, without electric pale Bending stress
  - (a). Summary of stresses
    - (i) At the support point

Pick up No 2

			(		
		Ø	100.	3	100.
Combined	Top	-35.54	20	= 35.51	9
Stress	Botton	34.42	114	34.42	116
Dead &	load	-10.45	10	-/245	10

## (ii) Span momer

		()	
		@	CO.
Combined Stress	Bottom	67.02	12
Dead Load	Boccom	19.82	10

(Note 1) Dead load is of Pick-up No. 1 (Note 2) co. — combination

- (b) Allowable scress for upper beam, safe against cracking
  - (i) At the support point 2

Dead load Md = -10.45 tm (case 1)

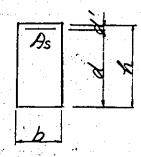
Train load + Impact MT-I = -7-32 (case 2)

IM = -17.77 tm

 $d = \frac{-7.32}{-17.77} = a41 > 0.25$ 

Hence Vsa= 1000 Mane

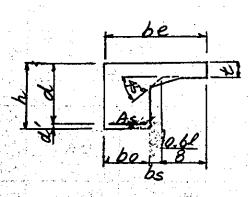
- (C) Crass section used for scress calculation
  - (i) Cross section at the support paint



Cross section at the spon center poince

(ii) Effective width of T-bean at

compression fibre



$$t = 28$$
 cm

	(0	d) Calcu	lation o	of benda	ing stress
	<u>.</u>				
		(	3)	2	
М	(tm)	-10.45	-35.54	61.02	
N	(1)	, V			
S	(1)				
b	(cm)		70	122.5	
h	(cm)	2.0	$\infty$	200	
d	(cm)	19	0.9	194.2	
ď	(cm)		9.1	5-8	·
4 -	/ •1			029-5	
As	( c <del>m²</del> )	029-5	= 32.12	= ,32.12	
p		200	25-13	a 00135	
				t=28	
As	(cm²)			t/d = a144	
p <sup>'</sup>				1/a - 00111	·
				<u> </u>	•
e = M	(Cm)				
e = M	/N+u				
e = M	/N-u				
e/h					
d/e		,			
ď /h	!	1			
d'/d					
	(kg/cm²)	0.41	1-40	1.32	
k				C.106	
ç					
j				a 9.3.2	
1/Lo		9.	.24		
1/Ls		451			
$\beta = \sigma s$					
<del></del>	{ kg/cm²}		12.9	15-8	
	(kg/cm²)	180	630	1040	
<del></del>	(kg/cm²)				
<del> </del>	(kg/cm²)	1000	/ 300	1800	
<u> </u>	(kg/cm²)		90	90	
	(kg/cm²)				
num	rer	M-1	<del>'</del>	M-47.48	
Combin	ation	0	D+S	D+T+I	

- (e) Required minimum cross section of re-bars
  - (i) At the top of support point

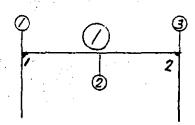
$$As = \frac{M}{\sqrt{sa \cdot j \cdot d}} \times \frac{4}{3}$$

$$= \frac{35.54 \times 10^{5}}{1800 \times 0.875 \times 190.9} \times \frac{4}{3} = 15.76^{cm^{2}}$$

Hence 
$$D29-5 = 32.12^{cm^2} > 15.76^{cm^2}$$

(ii) At the span center point

$$As = \frac{61.02 \times 10^5}{1800 \times 0.875 \times 194.2} \times \frac{4}{3} = 26.60^{\text{cm}^2}$$



(2) The case, electric pale is equipped allowable compression stress due to bending

(a) Summary of stresses

(i) At the support point

Pick up No 4

	$\bigcirc$				
		0	100.	3	l CO.
Combined	Top	-54.68	 	-5468	35
Stress	Botton	20.30	28	20.30	30
Dead &	oad	- 32.15	22	32.15	24

(11) Span momes

		<b>(</b> )	
		@	co.
Combined Stress	Bottom	39.02	34
Dead load	Bottom	- 2.18	2/

(Note 1) Dead load is of Pick-up No.3 (Note 2) co. — combination

- (b) Allowable scress for upper beam, safe against cracking
  - (i) At the support point 2

Dead load pole load Md. = -39.15 (cose 24)

Train load + Impact MI+I = -7.32" (case 2)

IM = -46-47 em

 $d = \frac{-7.32}{-46.47} = 0.16 < 0.25$ 

Hence Vsa= 1200 Mene

			·		PAGE 230
	200	1 2:00	+ bandin	g stress	
	accus	ation o	· percure	7 3,2 3,3 3	
	<del></del>	3)	(2)		
M (tm)		T -	39.02		
N (u)	-39.15	-5468	37.02		
S (1)	<u> </u>		-		-
b (cm)		0	122-5	,	
h (cm)	20		200		
d (cm)		0.9	1942		
d' (em)		9.1	5-8.		
			029-5		
As (cm²)	029-5	-= 32.12	· ·		
p		0240	0.00135		
4			t=28		
As' (cm²)	•		\$d=0.144		
p	,				
e= M/N (cm)					·
e = M/N + u					
e = M/N - u					
e/h					
d/e		·		<u> </u>	
d'/h					
d'/d		<u></u>			
Me/bd*(kg/cm²)	1.53	2-14	0.84		
k			0-186		<del></del>
С	· · · · · · · · · · · · · · · · · · ·				
j		····	0.443		
1/Lc		-24		·	
1/Ls β= σs/σc	9	k5/	-		
<u></u>					
$\sigma c = (kg/cm^2)$ $\sigma s = (kg/cm^2)$		19.8	10.1		
T (kg/cm <sup>2</sup> )	690	970	660		· · · · · · · · · · · · · · · · · ·
OSO (kg/cm²)				<u></u>	<del></del>
σca (kg/cm²)	1200	1800	90		·
Ta (kg/cm²)		90	10		
Nonogram		47	M-47 40	•	
number	M-1	<del></del>	M-47.48 D+T+I+E		
embination	D+E	JUTSTE	DETTITE	<del> </del>	JICA

- (d) Required minimum cross section of re-bans
  - (1) At the top of support point

$$As = \frac{M}{\text{Rsa} \cdot d} \times \frac{4}{3}$$

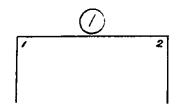
$$= \frac{39.15 \times 10^{5}}{1200 \times 0.875 \times 190.9} \times \frac{4}{3} = 26.04^{cm^{2}}$$

Hence 
$$D29 - 5 = 32.12^{cm^2} > 26.04^{cm^2}$$

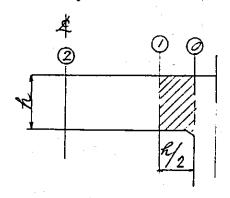
(ii) At the span center point

$$As = \frac{39.02 \times 10^5}{1800 \times 0.075 \times 194.2} \times \frac{4}{3} = 17.01^{\text{cm}^2}$$

## 2. Shearing stress of upper beam



(1) Summary of shearing stress

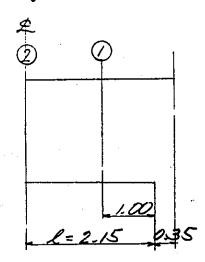


For the design of cross section of beam end, the value of shearing stress at 1/2 point is applied.

	Left support	<i>co.</i>	Right support	20.
0	36.32		-36,32	
0	23.04	30	- 23.04	28
2	7.46		7.46	

(Note) CO. - combination

- (2) Shearing stress
  - (a) Shearing stress caused by bending



(i). Shearing scress of the member of unifornheight

$$\gamma_1 = \frac{23.04 \times 10^3}{70 \times 190.9} = 1.72^{\frac{1}{9} \text{cm}^2} < 3.9^{\frac{1}{9} \text{cm}^2}$$

$$T_2 = \frac{7.46 \times 10^3}{70 \times 1900} = 0.56^{\circ}$$

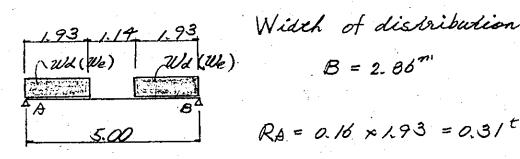
- (b) Torsional moment
  - a) Torsional momes caused by the loads on Rahmen (rigid frame)
  - (i) Due to dead load

    Derived from slab calculation

    Uniformly discribeted load

    Equivalent uniform load of crack weight

    Wd = 0.45 4m × 1/2.86 = 0.16 4m²

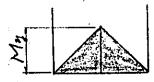


At the l/4 paints  $M44 = 0.31 \times 1.25 - 0.16 \times 1.25^{2} \times l/2 = 0.26 \text{ tm}$   $We/4 = \frac{32 \cdot M'/4}{3 \cdot l^{2}} = \frac{32 \times 0.26}{3 \times 5.00^{2}} = 0.11 \text{ tm}^{2}$ 

Total uniform load

5Wd= 1.78 + 0.11 = 1.89 tm2

Fixed end moment, at negative side  $M_7 = -\frac{1}{24} \text{ Wd } \ell z^2 = -\frac{1}{24} \times 1.89 \times 5.00^2 = -1.97^{tm}$ 





 $MTA = 1.97 \times 2.50 \times 1/2 = 2.46 \text{ tm}$   $MTI = 2.46 \times \frac{2.15}{2.50} = 2.12 \text{ tm}$   $MT2 = 2.46 \times \frac{1.15}{2.50} = 1.13 \text{ tm}$ 

(ii) Caused by crain load

From the slab calculation

Equivalent uniform load

We = \frac{16}{1.5 \times 2.86} = 3.73 \frac{4}{10^2}

Refer the dead load diagram  $RA = 3.73 \times 1.93 = 7.20^{\circ}$ 

Ac the  $\frac{2}{4}$  point  $M = 7.20 \times 1.25 - 3.73 \times 1.25^2 \times 1/2 = 6.09^{tm}$   $W = 1/4 = \frac{32 \times 6.09}{3 \times 5.00^2} = 2.60^{tm}$ 

Hence, We = 2.60 4me

Fixed end moment, at negative size  $M_{\eta} = -\frac{1}{24} \times 2.60 \times 5.00^2 = 2.71^{\text{tm}}$ 

 $M_{TA} = 2.7/ \times 2.50 \times 1/2 = 3.39^{tm}$   $M_{TI} = 3.39 \times \frac{2.15}{2.50} = 2.92^{tm}$   $M_{T2} = 3.39 \times \frac{1.15}{2.50} = 1.56^{tm}$ 

Train load + Impact

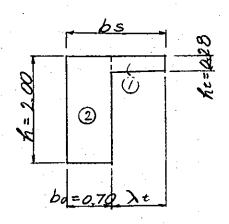
$$\lambda = 0.480 \times (1 - \frac{5.00}{200}) = 0.468$$

$$M_{TI} = 2.92 \times 5 = 4.29$$

(iii) Dead load + Train load + Impact

$$\Sigma M_2 = 1.13 + 2.29 = 3.42$$

b) Torsional moment, beared by cross beam



Effective width

$$bs = bo + \lambda t$$
 $\lambda t = 3 \cdot ht$ 
 $= 3 \times 0.28 = 0.84^{tm}$ 
 $bs = 0.70 + 0.84 = 1.54^{m}$ 
 $h = 2.00^{m}$ 

(i) Calculation of distribution ratio

	a	b	a/b	R	$It = R \cdot a \cdot b^3 \qquad (mt)$
<b>(</b> )	0.840	0.250	<i>3.000</i>	0.263	0.263x0.840x0.283 = 0.00485
2	2.00	0.700	2.857	a260	0.260 × 2.00 × 0.703 = 0.17836
Total				ZIt=0.18321	

(ii) Torsional moment beared by the beam (for re-bar arrangement in axial direction)

Dead load + Train load + Impact

Front face of calumn  $Mt1 = 6.41 \times \frac{0.17836}{0.18321} = 6.24$ At the  $\frac{1}{2}$  point  $Mt2 = 3.42 \times 7 = 3.33$ 

C) Shearing stress caused by earsian

Shearing stress caused by torsion is

calculated followed the equation.

$$Tt = \frac{Me}{It} \cdot b \cdot q$$

b: Sharther side length

a: longer side length

le: Table - 40.2

(i) Front face of column

$$\frac{a}{b} = \frac{200}{70} = 2.857$$
  $4 = 0.980$ 

 $Tt_1 = \frac{6.24 \times 10^5}{18.321 \times 10^6} \times 70 \times 0.980 = 2.34^{-100}$ 

(ii) At the 1/2 point

$$\frac{a}{b} = \frac{200}{70} = 2.857$$
  $\gamma = 0.980$ 

## (c) Combined shearing stress

Combined allowable shearing stress  $Ta = 17 \times 1.3 = 22.1^{E0/m^2}$ 

Combined shearing stress

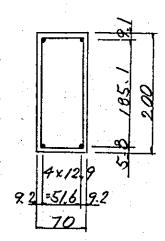
Stirrup 0/3-2 sets are arranged in 25.0 cm ctc.

(d) Calculation of axial re-bar arrangement, resisting corsional moment.

Required re-bar arrangement is calculated followed the equation.

$$As = \frac{Mt(b_i + k_i)}{0.8 \cdot \sqrt{sa \cdot b_i \cdot k_i}}$$

a) Front face of calumn



$$Mt = 6.24^{tm}$$

$$Sa = 1800^{t0/cm^2}$$

$$b = 516 + 2.9 + 1.3 = 55.8^{cm}$$

$$h_1 = 185.1 + 2.9 + 1.3 = 189.3$$

Required cross section of re-bars arranged at sharter side

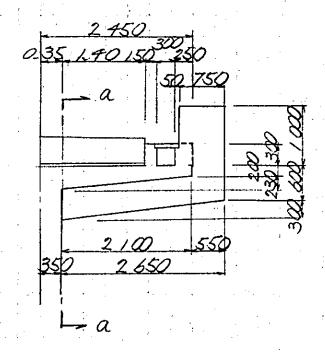
$$Asb_1 = 10.05 \times \frac{55.8}{2x(55.8 + 189.3)} = 1.14 cm^2$$

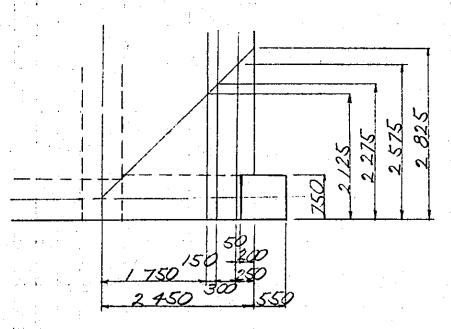
- 3. Required cross section of re-bars arranged at longer side  $Ash_1 = 10.05 \times \frac{189.3}{2(55.8 + 139.3)} = 3.88^{cm^2}$
- (i) Top and Batton

  Minimum section of re-bare

  As = D29-5 = 32.12 cm² > 0.14 cm²
- (ii) Side (one side)  $As = 0/6 4 = 7.94^{cm^2} > 3.88^{cm^2}$  8% of main bars section (bath sides)  $As = 32.12 \times 0.08 \times 1/2 = 1.28^{cm^2} < 7.94^{cm^2}$

## 4. Calculation of electric pale beam





electric pale Road  $M = 5.00^{tm} \quad N = 2.00^{t}$ 

Moment of @-(a) section
a) Dead load

				<del>,</del>
	Calculation	/ (t)	X (-em)	Mica
Electric pale.				5.00
		2.00	2.275	4.55
hand rail	0.20 × 1.975	0.40	2.000	0.80
Curb	2.5 × 0.30 × 0.25 × 1.95	0.37	1.975	0.73
bellasi stopper	2.5 ×0.15 × a 30 × 2.20	0.25	1.475	0.37
Duct lid	25 x 0.05 x 0.30 x 2.425	0.09	1-700	0.15
Cable	0.06 x 2.425	0-15	1.700	0.26
ballast	1.9×0.481× (0.75+2.125)×/2×1.40	1.84	a 812	1.59
Sloping concrete	2.35×0.07×(0.75+2.125)×/2×1.46	) a 33	0.812	0.27
Slab	2.5"×0.20×(0.75+2.825)×/2×210	1.88	1.253	2.36
• <del>•</del>	2.5 × 0.23 × (0.75 + 2.325) ×/4 ×2.10	1.08	0.977	1.06
Support of	2.5 × {0.60 × 2.65 - (0.20 + 0.43)			
electric pole	× 1/2 × 2.10} × 0.75	1.74	1. 388	3.29
<b>4</b>	2.5" × 0.30 × 2.65 × 1/2 × 0.75	a 75	0.883	0.66
•	2.5 × 0.75 × 0.75 × 100	1.41	2.275	3.2/
total		12.29		2 <i>4.30</i>

- b) Train load and Impact load
  - (1) Train load

- (2) Impace coefficient

  l = a58" --- i = a587
- (3) Train load + Impace load  $Ma-a = 0.94 \times (1+0.587) = 1.49^{tm}$   $S = 5.59 \times 0.58 \times ($ , ) = 5.14
- () Combination of stress

  (Dead + Train + Impacts) Road

  M = 24.30 + 1.49 = 25.79 to

  S = 12.29 + 5.14 = 17.43 to

Theretore, Isad = 1200 +9/am2

(4) 0		1 0	•
	ress ca	consaci	in .
	@-@ section	9	
M (tm)	2430	25.79	
N (i)			
S (t)		<u> </u>	
b (cm)	75	4	
h (cm)	90	5	
d (cm)	80.9	<u>,,                                    </u>	
d' (cm)	9.1	. ,	<u> </u>
As (cm²)	029-5 = 32./2	*	
p	0.00529	<b>5</b>	
1	<u> </u>		
As' (cm²)			·
p'			
e= M/N (cm)			
e = M/N + u			
e = M/N - u			
e/h			
d/e			
d /h			,
d/d			
He/bd2(kg/cm2)	4.95	5.25	
<u>k</u>			
c			
j			
1/Lc	•	<u> 686</u>	
1/Ls	2/2	2/2	
$\beta = \sigma s / \sigma c$	<u> </u>		
σc (kg/cm²)		36.0	
OS (kg/cm²)	1050	1110	
T (kg/cm²)			
osa (kg/cm²)	1 200	1 800	
σcα (kg/cm²)		90	
ta (kg/cm²)		<del></del>	
number	M-1	<del></del>	
combination	0	0+T+I	LICA

#### (d) Calculation of shearing stress

$$7 = \frac{17.43 \times 10^3}{75 \times 80.9} = 2.87 \times 3.9^{150/m^2}$$

Therefore

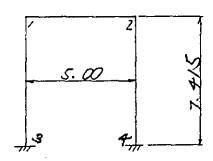
Stirrup

Use 0/3-2 set 25 cm ctc.

Bar arrangement in ordial direction Use D16-2 bars (one side)

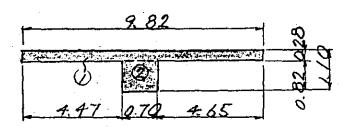
- § 7. Rigid frame analysis on transversal section (2-2) of elevated structure
  - (1) Elements for vigid frame analysis

    1. Conligulation and dinension of rigid frame



2. Cross-Sectional area and moment of Inentia of the member

(1) Member (1~2)



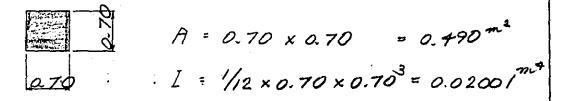
	b (m)	h (m)	A (m²)	y (m)	A. y (m)
$\bigcirc$	9.820	0.280	2-750	0.140	0.38500
2	0.700	0.820	0.574	0.690	0.39000
Σ.			3.324		0.78106

$$y = \frac{0.78106}{3.327} = 0.235^m$$

<u></u>							
	b (m)	h (m)	A (mt)	y (m)	I. (m <sup>4</sup> )	A.y. 2 (m+)	Io + A. yo 2 (m²)
$\bigcirc$						0.02482	
2	0.700	0.820	0574	0.455	003216	0-11883	0-15099
Σ			<i>3</i> .324		005012	0-14365	0.19377

CYOSS-Sectional Area  $A = 3.324^{m^2}$ Moment of Inertia  $I = 0.19377^{m4}$ 

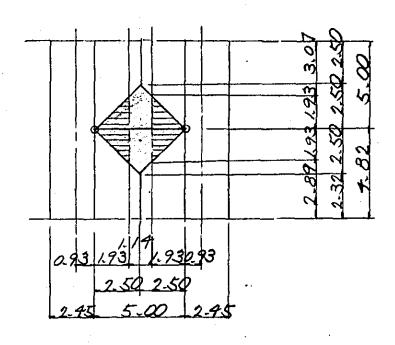
#### (2) Member (123.224)



(3)  $axial\ height$  $h_1 = 7.650 - 0.235 = 7.415^m$  (2) Calculation of loads

1. Dead Load

(1) Member (1-2)



#### (a) Disthibuted Load (A)

Weight of slab grading Concrete ant Ballast

Slab  $2.5 \frac{1}{100} \times 0.28 = 0.700 \frac{1}{100}$ Grading Concrete  $2.35^{\circ} \times 0.07 = 0.165^{\circ}$ Ballast  $1.9^{\circ} \times 0.481 = 0.919^{\circ}$ Wd =  $1.78^{\circ}$ 

Wd1 = 1.78 /m x 2.50 x 2 = 8.90 /m

(b) Distributed Load (B) Caused by Track weight
Distributed width

b = 2.00 + 1.5 × (0.3/6 + 0.07) + 0.28 = 2.86 m

W = 0.45 1/m × - 2.86 = 0.16 1/m

Wd = 0.16 /m x 1.93 x 2 = 0.62 4/m

(c) Distributed load (c) of CROSS beam and haunche of Slab

Beam 2.5 1/2 x 0.70 x 0.82 = 1.44 1/m

Haunch 2.5" x 0.95 x 0.15 x /2 x 2 = 0.17"

Wd = 1.614m

- (d) concentrated load of elements acting at joint P1. P2 as shown beam joint P1. P2
- 1) Weight of elements on the slub

  except track Weight

 $(2.99 - 0.95 \%) \times (4.82 + 5.00) = 24.45$ 

2) Distribut load (A)

178 x (232+482) × 1/2 × 250 = 15.89

3) Distributed load (B)

178 x (2.50 +5.00) x /2 x 2.50 = 16-69

t) Distributed load (C)

0.16 x (2.89+4.82) x /2 x 1.93 = 1.19

5) Distributed load (D)

0.16'x (3.07 + 5.00) x 1/2 x 1.93 = 1.25

6) Distributed load (E)

a 16 x (4.82 + 5.00) x a 93 = 1.46

7) Cantilever Slab

25th (0.20+0.43) x /2 x 2-10 x 9.82 = 18.24

B) Haunch of Slab

25 × 0.45 × 0.15 × 1/2 × 8.22 = 0.69

9) Longitudinal beam

 $2.5^{\frac{4}{10}} \times 0.35 \times (1.20 + 0.92) \times 9.12 = 16.92^{t}$ 

10) Haunch of longitudinal beam

25 x 1,20 x 0,40 x 1/2 x 0,70 x 2 = 0.84

11) Subtraction from Distributed load

-1.52 t/mx 0.35

= -a53

12) Deficit of column Weight

- 2.5 x a 70 x 0.70 x (0.28 - 0.235) = -0.06

P1 = P2 = 95.03

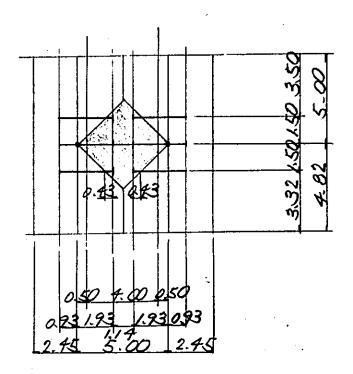
Column Welght

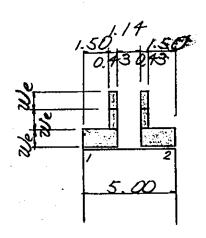
g = 2.5 1/2 x 270 x 0.70 = 1.23 1/2

- 2. Train Load and Impact Load
  - (1) Train Load (Single track)

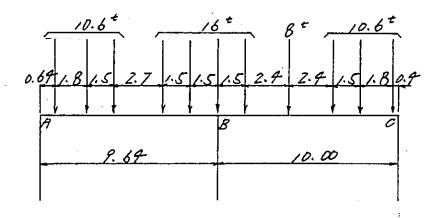
KS -16

(a) Distributed load acting on rigid - frame





(i) Concentrated load Caused by axial load acting on longitudinal beam



$$78 = \frac{1}{9.64} \times \{10.6 \times (0.65 + 2.55 + 3.85)\}$$

$$+ 16 \times (6.64 + 8.14 + 9.65)\}$$

$$+ \frac{1}{10.00} \times \{10.6 \times (0.40 + 2.20 + 3.70) + 8 \times 6.10\}$$

$$+ 16 \times 8.50\} = 73.41^{t}$$

$$P = 73.41^{t} - 5.59^{t/m} \times (0.43 \times 3 + 1.50) = 57.81^{t}$$

- (2) Impact coefficient
  - (a) Within Migid frame Section

    l1 = 5.00 m i01 = 0.480

    l2 = 9.73 " i02 = 0.433

For the case of double track reduction is made followed the formula

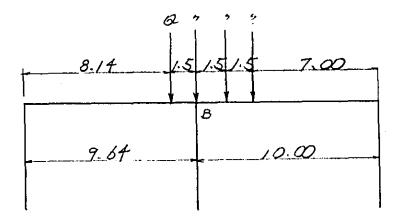
$$\dot{N}_{1} = 0.480 \times (1 - \frac{5.00}{2.00}) = 0.468$$

$$\dot{N}2 = 0.433 \times (1 - \frac{9.73}{200}) = 0.412$$

#### (3) Train Load + Impact

· Concentrated load Caused by Single track loading

#### 3. Train lateral load



$$H = \frac{1}{9.64} \times 2.40 \times (8.14 + 9.64) + \frac{1}{10.00} \times 2.40$$

$$\times (7.00 + 8.50)$$

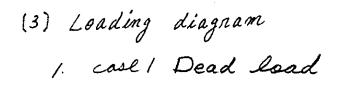
$$= 8.15^{+}$$

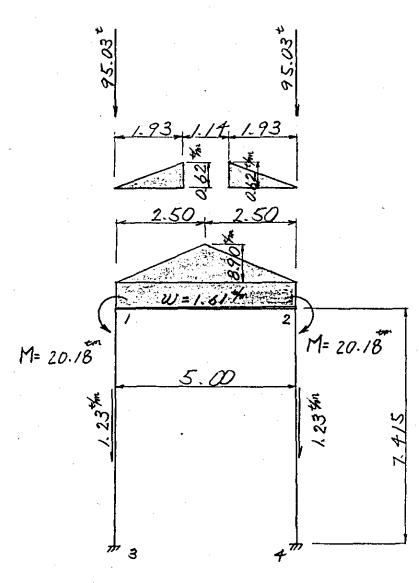
4. Forces of temperature change and/or Drying

- · Temperature Vise
- · Temperature drop + Drying Contraction
  -15°C

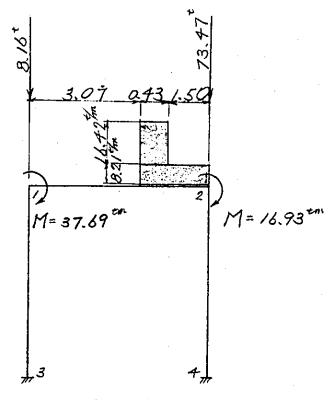
5. Dead load + Seismic force Kh = 0.1

 $H = \begin{cases} 1.6 / \times 5.00 + 5.00 \times 8.90 \times 1/2 \\ + 0.62 \times 1.93 \times 1/2 \times 2 + 95.03 \times 2 \\ + 1.23 \times 7.415 \times 1/2 \times 2 \times 0.10 \end{cases}$   $= 23.07^{t}$ 

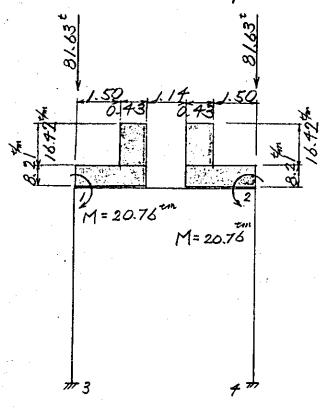




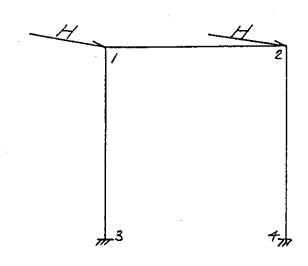




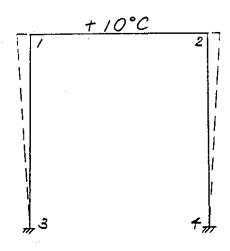
3. cue 3 Train load + Inpact



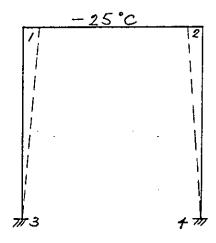
4. case 4 Train lateral load
$$H = \frac{8.15}{2} = 4.08^{\pm}$$



5. case 5 Temperature

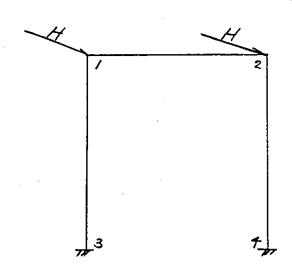


#### 6. case 6 Temperature + Contraction



#### 7. case 7 Seismic load

$$H = \frac{23.07}{2} = 11.54^{t}$$



#### (4) Combination of loads

#### 1. Basic load

Case No.	Kind of load	Loading Pattern
/	Dead load	
2	Train load + Impace	
3	Train load + Impact	
4	Train lateral load	
5	Temperature	+ 10°C
6	Temperature + Contration	- 25°C
7	Seismic Load	11
		•

#### 2. Combined load

Case No.	Combination of loads	7
8	/	1. 0000
9	1 + 2	"
10	1 + 3	′,
	1 + 2 + 4	0.8696
12	1 + 3 + 4	. "
13	1 + 2 - 4	65
14	1 + 3 - 4	<del>,</del>
15	1 + 5	4-3
16	1 + 6	4,
17	1 + 7	0-6667
18		6

#### 3. Critical case

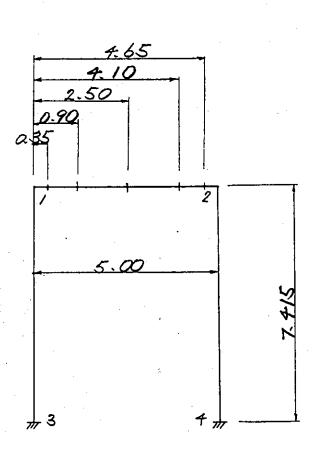
No.1 case 8 Crack

No. 2 case 9 ~ 18 Synthetic

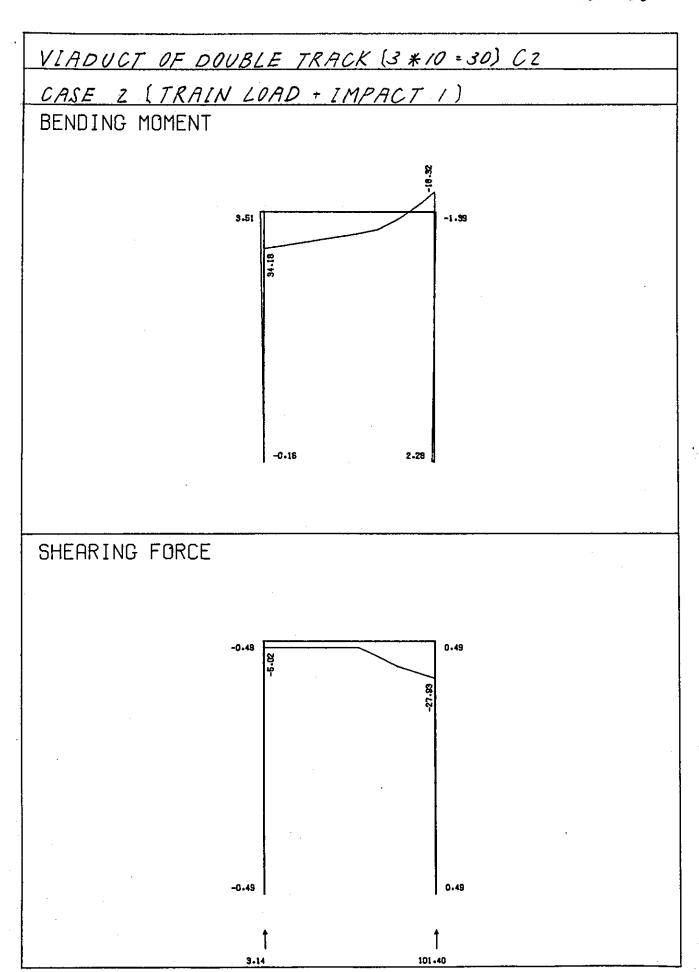
No. 3 case 9 ~ 14 footing

No. 4 case 17 ~ 18. "

9. Points of computing Stresses

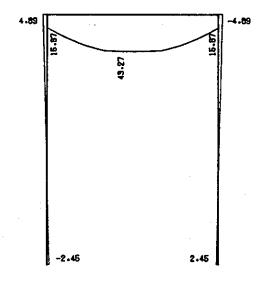


(5) stress diagnam VIADUCT OF DOUBLE TRACK (3 \* 10 = 30) CZ CASE / (PEAD LOAD) BENDING MOMENT SHEARING FORCE -0.12

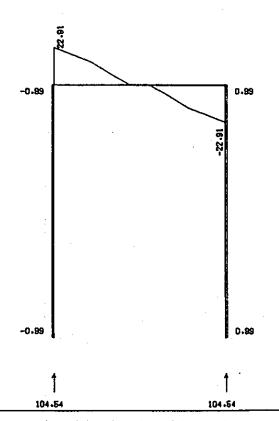


#### VIADUCT OF DOUBLE TRACK (3 \* 10 = 30) CZ CASE 3 (TRAIN LOAD + IMPACT Z)

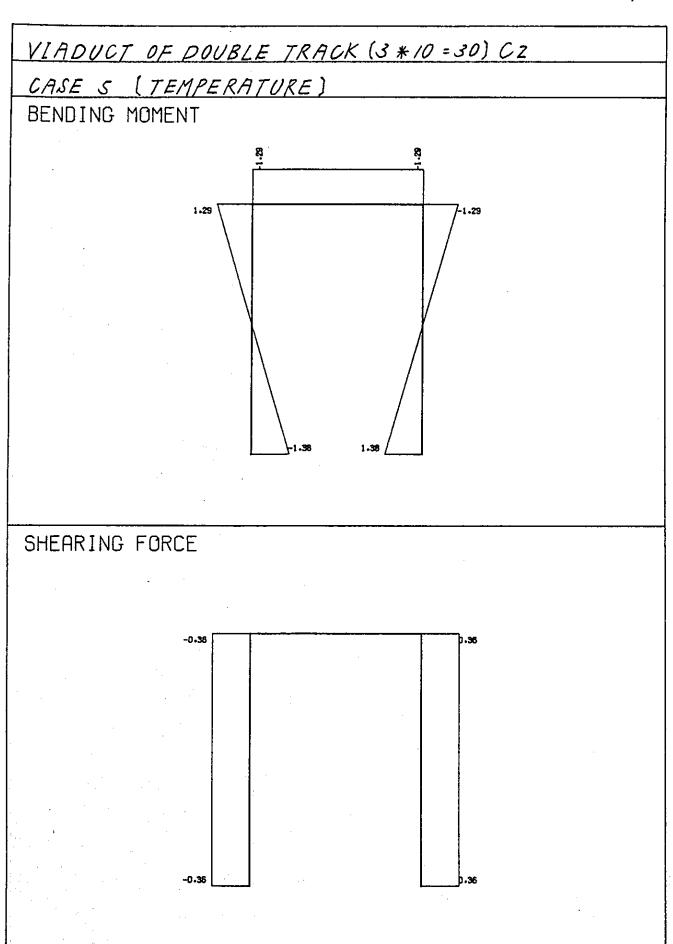
BENDING MOMENT



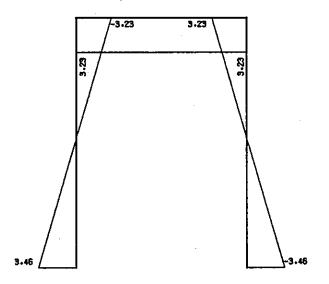
SHEARING FORCE



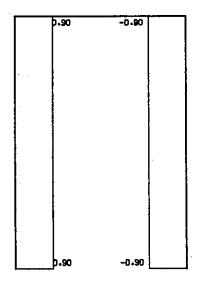
# VIADUCT OF DOUBLE TRACK (3 \* 10 = 30) CZ CASE & (TRAIN LATERAL LOAD) BENDING MOMENT SHEARING FORCE



### VIADUCT OF DOUBLE TRACK (3 \* 10 = 30) CZ CASE 6 (TEMPERATURE + CONTRACTION) BENDING MOMENT



SHEARING FORCE



## VIADUCT OF DOUBLE TRACK (3 \* 10 = 30) CZ CASE 7 (SEISMIC LOAD) BENDING MOMENT SHEARING FORCE 11-51

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= = MEMBER 2 (	5 2	1				= HEMBER	2	ir H	1-3)C==		
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ITAN 0.	000	8	. 596	121	-110.553	ITAN	0.000	ر ع	965*	121	-110.553
-	7.415	( 8	298	-121	-119,674	NATO	7.415	9	- 298	-,121	-119.674

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2 5900 (12) 31.275 19.266 -756 1 520 (18) -33.546 21.614 3 2.500 (10) 47.743 .000 -869 3 2.500 (15) -22.351 19.567 5 4.500 (14) 19.032 -25.082 -756 5 4.650 (17) -22.351 -19.567  JAN 5.000 (18) 14.887 .000 -869 HAX 2.500 (15) 2.769 -000  HAX 2.500 (10) 47.743 .000 -869 HAX 2.500 (15) 2.769 -000  HAX 2.500 (10) 47.743 .000 -869 HAX 2.500 (15) 2.769 -000  HAX 2.500 (10) 27.546 -7.593 -84.883 ITAN 0.000 (17) -26.341 7.754   JIAN 0.000 (18) 28.341 -7.754 -68.609 JIAN 7.415 (18) -22.7546 7.553   JIAN 0.000 (18) 28.341 -7.754 -68.529 JIAN 7.415 (18) -27.546 7.754   JIAN 7.415 (17) 28.341 -7.754 -68.529 JIAN 7.415 (18) -27.546 7.754   JIAN 7.415 (17) 28.341 -7.754 -68.529 JIAN 7.415 (18) -27.546 7.754   JIAN 7.415 (17) 28.341 -7.754 -68.529 JIAN 7.415 (18) -27.546 7.754   JIAN 7.415 (17) 28.341 -7.754 -68.529 JIAN 7.415 (18) -27.546 7.754   JIAN 7.415 (17) 28.341 -7.754 -68.529 JIAN 7.415 (18) -27.546 7.754   JIAN 7.415 (17) 28.341 -7.754 -68.529 JIAN 7.415 (18) -27.546 7.754   JIAN 7.415 (17) 28.341 -7.754 -68.529 JIAN 7.415 (18) -27.546 7.754   JIAN 7.415 (17) 28.341 -7.754 -68.529 JIAN 7.415 (18) -27.546 7.754   JIAN 7.415 (17) 28.341 -7.754 -68.529 JIAN 7.415 (18) -27.546 7.754   JIAN 7.415 (17) 28.341 -7.754 -68.529 JIAN 7.415 (18) -29.156 -7.754   JIAN 7.415 (17) 28.341 -7.754 -68.529 JIAN 7.415 (18) -29.156 -7.754   JIAN 7.415 (17) 28.341 -7.754 -68.529 JIAN 7.415 (18) -29.156 -7.754   JIAN 7.415 (17) 28.341 -7.754 -68.529 JIAN 7.415 (18) -29.156 -7.754   JIAN 7.415 (17) 28.341 -7.754 -68.529 JIAN 7.415 (18) -29.156 -7.754   JIAN 7.415 (17) 28.341 -7.754 -68.529 JIAN 7.415 (18) -29.156 -7.754   JIAN 7.415 (17) 28.341 -7.754   JIAN 7.415 (17) 28.341 -7.754   JIAN 7.415 (17) 27.546   JIAN 7.415 (17) 27.546   JIAN 7.415 (17) 27.754   JIAN 7.415 (17) 27.754   JIAN 7.415 (18) 28.341   JIAN 7.415	ITAN	0.000	1111	25.616	3.965	326	ITAL		_	-41.000	21.527	.080	i
3 2.500 ( 10 ) $47.743$ . 000869 3 2.500 ( 15 ) 2.769000 4.100 ( 17 ) -22.351 -19.567000 5 4.650 ( 17 ) -22.351 -19.567000 1 4.100 ( 18 ) 14.887826756 4 4.100 ( 17 ) -22.351 -19.567000 1 4.000 ( 18 ) 14.887826089	2	900	( 21 )	31.275	19,268	756			18	-33,546	21.614	080	
$\begin{array}{cccccccccccccccccccccccccccccccccccc$	М	2.500	( 10 )	47.743	000	869		2	<i>,</i> –	2.769	790-61	200	
JIAN 5.000 ( 18 ) 14.087 -25.082756 5 6 6.07	er u			31.275	-19,268	756			1	-22.351	-19.567	- 040	
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# MEMBER 3 ( 2 - 4 ) C = =	ITAN	0.000	( 18 )	27.546	7.593	-84.883	JIAN	;	<u>'</u>	-26,341	7,754	-62,529	:
0.000 ( 18 ) 28.341 -7.754 -62.529 ITAN 0.000 ( 17 ) -27.546 7.593 7.415 ( 17 ) -27.546 7.593 7.415 ( 17 ) -29.156 -7.754		1	į	ű			п	BER	2	_			; ;
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		•	:		:	:	:				;	<b>!</b>	

GRO-FANSY V6.3

	X		080	080	.08C 326	374		-84.483 -90.954		-62.529 -58.509					
<b>3</b> C	0		-828	-1.541	-11.177	-43.450	1	-7.593 -7.593	,	-7.754		i 			f
SHEAR HINIMUM	<del> </del>	= = 9 (	14.887	13.417	-11.060	-37.902	= = 0 (	27.546	# # # # # # # # # # # # # # # # # # #	3,5	:	!	:		
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1		ER 1 (	0.00	900	4.100	5.000	ER 2 (	6.000	ER 3 (	0.000		:			
•	: I	= HEMBER	ITAN	757	ט ב ר	JIAN	A HEMBER	YTAN JTAN	= = MEHBER	ITAN	ì				
:			756	1756	080	.080		-62,529 -68,639	:	-84.883		:			:
			38.586	29.604	2.788	828	! !	7.754		7.593		-			•
SHEAR HAXIMUM	H	2 2	-16.153	14.737	13.417	14.887	# # D	-28.341 29.156	# 11 20 0	-27.546		:			
SHE!	CASE	1 - 2 1	1 24 7	31.	( 18 )	( 18 )	1 - 33	( 11 )	1 2	(-17)		:			
		R 1 (	0.000	006	4.100	5.000	R 2 (	7.415	3 (	0.000			i		
		= = HEHBER	ITAN	2	) <del>-2</del> (5	JTAN	= # HEHBER	LTAN	= HEMBER	TTAN					:
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				•		: .	:	:	CRC - FANSY	V6•3	1	
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		-CASE-	-	0	N			-CASE-	H	0	N	
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ITAN	0.000	( 16 )	-14.219	13.499	. 890	ITAN	0.000	( 10 )	-3.718	38.429	698	
ر د د د	006.	( 16 )	-3.015	10.943	068*		980	101	954-92	28, 100	698	i
7 .3	4.100	16 1	-3.015	-10.943	.890	7 3	4.100	100	26.456	-28.100	-,869	
JIAN	5.000	( 16 ) ( 16 )	-9.599	-12.830	068*	JTAN	5.000	97 )	9.108	-34.786	- 869	
T HEMBER	5 (	# 1	= 9 (	:		= HEMBER	ER 2 (	£ + 1	= 0.			
ITAN	0.000	( 17 )	-28.341 29.156	7.754	-62,529	LTAN	0.000	( 10 )	4.296	998 989	-215.089	
= = MEMBER		7 + 2	II.			= HEMBER		2 .	= 0		. !	. :
TTAN	7.415	( 18 )	28.341 -29.156	-7.754	-62.529	LIAN	0.000	( 10 )	-4.298 2.148	. 869 . 659	-215.089	
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I TAN 0	0.000	( 11 )	25.616	3.965	-326	ITAN	0.000	1 71 )	-15.45.2	707.02	714	:
	.350	( 11 )	26.898	3,296	326	+1	350	14.	-3.190	35.00 E 1.3	7.670	
N I	006	15 )	31.275	19,268	756	-2	006	(13)	11.702	747	700	
2	2.500	( 10 )	47.743	000	869	כיו	2,500	( 11 )	22,705	45.534	305	
	001.	7.	31.275	-19.268	756	3	4.100	(11)	-11.050	-33.970	326	
	000		19,032	-25.082	756	īv	4.650	(11)	-31.388	-39.783	326	
2	7•0 au		939.6	-28.250	756	LTAN	5,000	(11)	-45.879	-42.952	-326	-
HAX 2	2.500	( 10 )	47.743	.000	869	MAX	2.500	( 11 )	22.705	-3.534	-,326	
= = HEHBER	2 -	1 - 3	= = 0 (			= = MEMBER	R 2 (	1 - 3	= - 0 -			
TTAN O	0.000	( 11 )	16.658 13.505	-4.304 3.222	-192,209 -101,631	1TAN JIAN	0.000	(11)	-10.389	3.222	-93.700	
= MEMBER	3 (	4 - 2	1 5 = =			= = MEMBER	3 (	7 - 2	# J		•    -  -	! !
STAN D.	0.000	[ 13 ]	12.231	-3.222	-179.143	JIAN	0.000	( 12 )	-15.058	4.304	-192.209 -187.675	
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v6.3	-			3.965	1.409	-33.970 -39.866	044.24-	-4.304		-3.222								
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		nun		38.586	29.604 5.168	-19.268 -25.082		3.222		4.304								
		H-H	E E 5	-16.153 -3.190	14.737	31.275 19.032 9.686	= 0 (	-10.389 13.505	11 0	15.257	1			•		:		
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1:	£		ER 1 (	0.000	2.500	4.650 5.000		0.000	ER 3 (	7.415	:			;		:	. ;	1
	PICK UP		= = MEMBER	ITAN	ผท	JTAN	į H	LTAN	= = HEMBER	LTAN	; ; ;		. 1				:	
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			. 69.8	869	- 869	8 6 G	869	869	:	-215.089		-215.689	-224.210	***	•								•	
· · · · · · · · · · · · · · · · · · ·	0		38.429	34.785	28.100	-28,160	-34,786	134.85-	:	869		•	690.						•	· · · · · · · · · · · · · · · · · · ·		*		
AXIAL HINIMUH		= = 5 (	-3,718	9.108	26.456	26.456	9.108	-3.716	= D	4.296	11 11	-4.298	2.148		1				,					
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\$ 1		HEHBER 1 (	0.00	350	2.500	4.100	4.650		MEMBER 2 (	0.000	MEMBER 3 (	0.00	7.415										•	i i
•		H H H	ITAN	<b>~</b> ■	o m	3	5	,	FOR III	LTAN	#3. # II	ITAN	JTAN	:			:		,					
:	X		326	326	326	326	-,326	925.		-93.700 -101.631	•	-179.143	-187,075				:			;				
<b>r</b>	0		3,965	3.296	1.409	-33.970	-39.783	766.34		3.222		3.222		:		1	!			i		٠		: : :
AXIAL HAXIMUH		= = 9 (	25.616	20.07	22.705	-11.060	-31.388	2.0 .0 .	# # D	13.505	# 0	12.231	-11.663				!		•					; ; ;
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CRC-FANSY V6.3

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1		-CASE-	-CASEH0	0	X	•		-CASE-	-CASEH		N	:
= = HEMBER	7	1 - 216	== 91			= = HEHBER	2 1 (	1 - 1	2 1 6 = =			
LTAN	0.000	7.21.1	14.887	-,828	.080	ITAN	0.000	( 18 )	-41,000	21.527	080	
-	.350	( 17 )	14.516	-1.341	.080	<b>-</b>	.350	(.18)	-33.546	21.614	080	
2	900	( 11 )	13.417	-2.788	.080	2	900	(18)	-22.351	19.567	080	
n	2.500	( 18 )	2,985	11.177	080	m	2.500	( 17 )	2.985	-11.177	.080	
<b>.</b> 7	4.100	( 18 )	13.417	2,788	.080	-	4.100	(1)	-22,351	-19.567	080	
ŝ	4.650	( 18 )	14.516	1.341	• 080	5	4.650	1 17 )	-33.546	-21. D14	080	
JAN	5.000	( 18 )	14.887	.628	080	UTAN	5.000	( 11 )	-41.000	-21.527	.080	:
HAX	2.500	2.500 (18)	2.985	11-177	080	HAX	2,500	2.500 ( 17 )	2.985	2.985 -11.177	.060	
= = MEMBER	2 (	1 - 3 ) C	= = 0 ( )			# = MEMBER	1 2 8	1 -	3 ) C = =			
ITAN	0.000	( 18 )	27.546	7.754	-84.883	LTAN	0.000	( 17 )	-28,341	7,754	-62,529	r
= HEHBER	3 (	3 ( 2 - 4	4 ) C = #			= HE48ER		. 1	# U #			
LTAN	0.00	18 )	28.341	-7.754	-62,529	NATH	0.00.0	( 17 )	-27.546	7,593	-84.863	
JIAM	7,415		28.759	7.593	+96°06-	NATO	7.415	( 18 )	-29.156	-7.754	-68,689	

CRC-FANSY V6.3

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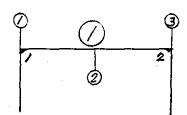
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	1112		0.80	080	180.	080°		-64.663		-84.863							
T.			21.527	19.567	2.788	1.341		-7.593		7,593				:			
AL MINIHUM		= = 9 (	-41-000	-22.351	13.417	14.887	== 0.	27.546	= = 0	-27.546	1 !	!	:				
AXIAL	-CASE-	1 - 2	( 18 )	81.	-	<u>-'-</u>	P = 4	( 18 )	7 - 2	(17)			1				
•	]	BER 1 (	0.000	900	4-100	4.650	BER Z'(	7.415	GER 3 (	7.415							
	***************************************	= = MEMBER	ITAN	2	7 3	JIAN	= HEMBER	TIAN	= = MEMBER	ITAN	:		1		·		
	N		.080	. 080	080	080		-62.529		-62.529 -68.609		:					
•			-1.341	-2.788	-19.567	-21.014	***************************************	7.754		-7.754	: •	*			:		
AXIAL MAXIHUM	The Market	" " "	14.887	13.417	-22.351	-33.546	= = 0 (	-28.341 29.156	E 3	28.341 -29.156	· .i						
AXI	-CASE-	1 - 2	( 17 )	( 17 )	( 17 )	( 17 )	ਲ 	(17)	2 - 4	(18)						1	
		MEMBER 1 (	-	006		5.000	MBER 2 (	0.000	= MEMBER 3 (	7.415	**************************************						
		HE HE	ITAN	٧ ٢	9	JTAN	= = HEMBER	1TAN JTAN	発用を	LTAN				:			

- (6). Calculation of upper beam
  - 1. Stress calculation of upper beam



(a) Summary of stresses
(i) At the support point

Pick up No 2

•		0	100.	3	<i>CO.</i>	
Combined	Top	-41.00	18	= 25.88		
Stress	Boccon	26.90	111	19.03	14	
Dead Load		-19.58	8	-19.5B	8	

(ii) Span momex

		<b>(</b> )	
		2	co.
Combined Stress	Bottom	4274	10,
Dead load	Boccom	4.48	18

(Note 1) Dead load is of Pick-up No. 1 (Note 2) co. — combination

- (b) Allowable seress for upper beam, safe against cracking
  - (i) At the support paint 2

Dead load Md = -19.58 tm (cose 1)

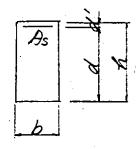
Train load + Impact MT+I = -18.32 \*m (case 2)

IM = -37.90 tm

 $d = \frac{-18.32}{-37.90} = 0.48 > 0.25$ 

Hence Isa = 1000 Mome

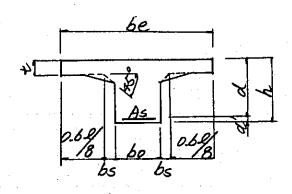
- (C) Crass section used for stress calculation
  - (i) Cross section at the support point



Cross section at the span center point

(ii) Effective width of T-bean at

compression fibre



(4	d) Calcul	lation c	+ bendi	ng stress
		•		
		(3)		
M (tm	19.58	45.88	47.74	
N (t	· -			
S (1	)			
b (cm	7	0	175	
h (cm	1/0		110	
d (cm	100	9	104.2	
d' (cm	, i	?~/	5-8	
			029-7	
As (cm²	029-6	= 38.54	44.97	
p	0-00		0.002+7	
4 -			t = 28	
As' (cm²	'  ·		1/d=a 269	•
p <sup>'</sup>				
e = M/N (cm	)		,	
e = M/N + u			·	
e = M/N - u		<del>,</del> , , , , , , , , , , , , , , , , , ,		
e/h				
d/e				
d' /h				
d /d				
He/bd*(kg/cm²	2-74	6.44	251	
k				
С				
j		· · · · · · · · · · · · · · · · · · ·	<b>N</b>	
I/Lc		79	<b>4/4</b>	
1/Ls		06	440	
β= σs/ σc				
σc (kg/cm²)		437	23.0	
OS (kg/cm²)	570	1330	1110	
T (kg/cm²				
OSO (kg/cm²	1000	1800	1 800	
σca (kg/cm²	1	90	90	
Ta (kg/cm²				
Nonogram	M-1	M-1	M-47-48	
Combination		O+7+I	O+T+ I	

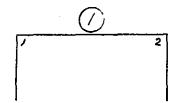
- (e) Required minimum cross section of re-bans
  - (i) At the top of support point

$$As = \frac{M}{0 \cdot s_{0} \cdot t \cdot d} \times \frac{4}{3}$$

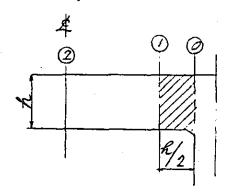
$$= \frac{45.88 \times 10^{5}}{1800 \times 0.875 \times 100.9} \times \frac{4}{3} = 38.49^{cm^{2}}$$

(ii) At the span center point

## 2. Shearing stress of upper beam



(1) Summary of shearing stress

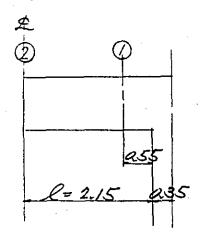


For the design of crass section of beam end, the value of shearing stress at 42 point is applied.

	Left support	co.	Right support	co.				
0	35-42		-39.7 <i>8</i>					
0	29-60	14	-33.97	//				
2	5-17		9.33					
		   . 						

(Note) CO. - combination

- (2) Shearing stress
  - (a) Shearing stress caused by bending



(i) Shearing scress of the member of uniform height

$$\gamma_1 = \frac{33.97 \times 10^3}{70 \times 100.9} = 4.81 \text{ fant} > 3.9 \text{ for}^2$$

$$C_2 = \frac{9.53 \times 10^3}{70 \times 100.9} = 1.35$$

Torsional moment

Torsional monent is disregarded, because that is acting symmetrically

Therefore, stirrup calculation is made.

# (b) Calculation of diagonal tension bar i) Calculation of total shear Referr R.C. standard 39, (2).(2)

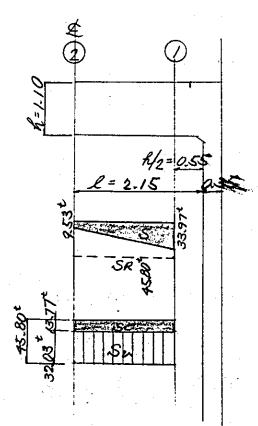
#### where

Sc: Shearing stress beared by concrete(t)

Sv: Shearing stress beared by stirry (t)

Sb: Shearing stress beared by turned up bars (t)

Assumed Sv ≥ Sb



Shearing force diagram

Resisting shearing force diagram

(i) Shearing stress beared by concrete

Sc= 1/2. Tc. b.d

Where, To: 3.9 Fg/cm2

b. : Widsh of member (cm)

d: Effective reight of member

at the examining section.

Sc1 = 1/2 × 3.9 × 70 × 100.9 ×10-3= 13.77t

(ii) Shearing force beared by stirrup

Su = Av. Psa. d

1.15. D

where, Av: Total cross section (cm²) of stirrup with the section S.

Psa: Allowable censile stress of re-bar

Psa= 1800 x8/cm²

8: Interval of stirrups measured along the member onis (cm)

Arranged stirrups D/3-2 sets 25 cm ctc,  $Av = 1.267 \times 4 = 5.07$  cm<sup>2</sup>

 $S_{2} = \frac{5.07 \times 1800 \times 100.9}{1.15 \times 25 \times 10^{3}} = 32.03 t$ 

- (iii) Shearing stress beared by turned up bars

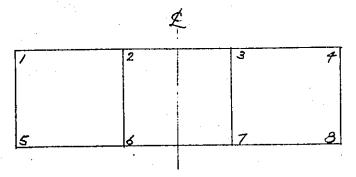
  Disregarded the turned up bars

  for calculation.
- (iV) Total shear  $\Sigma SR = Sc + Su + Sb$   $\Sigma SR = 13.77 + 32.03 + 0$   $= 45.80^{t} > S_{i} = 33.97^{t}$

Re-bars D16-2 sets (one side) in axial direction

#### & B. Calculation of column

1) Rahmen (Rigid Frame) calculation in railway protile

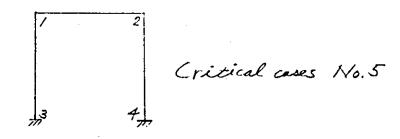


#### Critical ases No.2

Member	M (tm)	N (E)	case number
1-5	36-75	83.76	2/
1-5	26.49	72.54	20
2 - 6	38.88	79.14	20
6-2	34.81	85.64	20

(Refer the result of computor analysis on stress Calculation)

# 2) D-WRalmen (Rigid frame) calculation in the direction of railway cross section



Member	M (em)	N <sup>(t)</sup>	case number
1-3	28.34	67.41	19
<b>3</b> - /	30.41	75.29	35
4-2	30.21	101.73	35

# 3) 2-2 Ralmen (Rigid frame) calculation in the direction of railway cross section

Member	M (em)	<b></b>	case
/ - 3	28.34	62.53	17
3-/	29.16	68.61	17
4-2	28.76	90.96	17

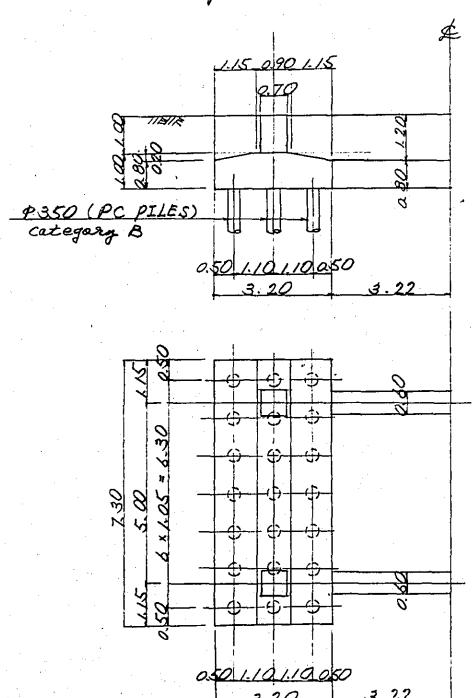
Critical cases No 2

,	Stress	calcula	tion			······································
	Rehme	n. in to	ailway	profile		•
	1-5	1-5	2-6	6-2		
M (tm)	£6.75	26.49	38.88	34.81		
N (i)			79.14			
Š (1)						
b (rm)			70		na say sa a a a a a a a a a a a a a a a a	
h (cm)		7	0	p. 444		
d (cm)	· · · · · · · · · · · · · · · · · · ·	6	3.9			
d' (cm)			6./			
As (cm²)		D25 -	5 = 25	.34 cme		
p		0.0	0517			
As' (cm²)		D25 -	5 = 25	.34 cm²	: :	
р		0.0	20517	4		+
e = M/N (cm)	43.88		49.13	40.65		
e = M/N + u		·				·
e = M/N - u	van er a sundanskannen er en steden.	/ <b></b>				
e/h	0.627	0.522	0.702.	0.581		
d/e						
d'/h d'/d	0.087	0.087	0.087	0.087		
Ne/bd*(kg/cm²)	17.09	14.80	16.15	17.48		# # * * * * * * * * * * * * * * * * * *
k	0.463	0.513	0.437	a482		
c	0219	0.261	0.197	4 235		
j		e de la companya de la companya de la companya de la companya de la companya de la companya de la companya de				
1/Lc						
1/Ls						
β= σs/ σc					<del></del>	
σc (kg/cm²)	78\/_	_56.8_	82.2	743		
σs (kg/cm²)	1140	660	1340	1000	· · · · · · · · · · · · · · · · · · ·	
τ (kg/cm²)						<u> </u>
σsα (kg/cm²)		18	<i>∞</i>			···············
$\sigma ca = (k_B/cm^2)$	e e e e e e e e e e e e e e e e e e e		20			
$Ta = (kg/cm^2)$	<del></del>			<del>                                     </del>		<del></del>
number	MN-5.6.7	,	<b>*</b>	<u>,</u>	, ,	
combination	D+S	5	,	5		<u> </u>

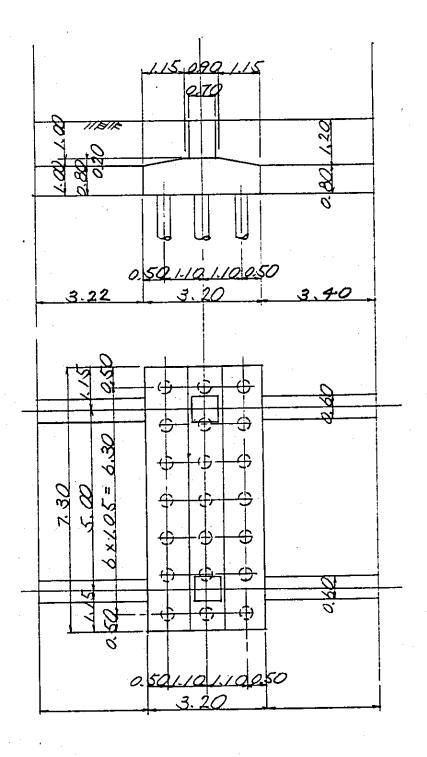
	()-()P.5		dinaction	2-2 Rah		lin a -ti-
				of railure		
		3 — 1			<i>उ /</i>	[ ·
M (tm)				28.34		48.76
N (i)				62,53		
S (1)	<u> </u>					
b (ст)		70			70	
h (cm)		70			70	**************************************
d (cm)	and the state of t	63.9		-	63.9	
d' (cm)		6.1			6./	······································
As (cm²)		- سد	31			× 24
		-5=	•	1	-5 = 2	
<i>p</i>		0.00517			0.00517	
As' (cm²)	D 25	-5_=;	25.34	025	-5 =	25.34
p'		0.0051			2 00517	
e = M/N (cm)		40.39	<b>{</b>	45.32	42.50	31.62
e = M/N + u	•					
e = M/N - u	,					
e/h	0.601	0.577	0.438	0.647	0.607	0.452
d/e	· · · · · · · · · · · · · · · · · · ·					
<u>d'/h</u>	0.087	0.087	0.087	0.087	U.087	0.087
d' /d	,	<b>.</b>				· · · · · · · · · · · · · · · · · · ·
Ne/bd*(kg/cm²)	/ 9.7.0	1		12.76	1	
. k	0.473	0.484	057	0.455	0.471	0.563
· C	0.228	0-237	0307	0.2/2	0.226	0.299
$\frac{J}{1/Lc}$		e name process of the second s			· - · · · · · · · · · · ·	
1/Ls			** ****		,	
$\beta = \sigma s / \sigma c$	·	,				,
$\sigma c = (k_B/cm^3)$	101	64.9	67.6	50.1	62.1	62.2
$\sigma s = (k_R/cm^2)$	60.4	[ · · ·	600	910	880	580
$\tau = (k_R/cm^2)$	840	860	<u></u>			
osa (kg/cm²)		1 800	-		1 800	
$\sigma ca = (k_R/cm^2)$		90			90	· · · · · · · · · · · · · · · · · · ·
$\tau a = (k_R/cm^2)$		<u></u>			- Vacana and American Services	
Nonogram	M.	N-5.6.7		MI	V-5.6.7	
combination		0+S+E	7	D+5	**	

#### §9. Basic criteria for calculation

- 1. Analysis in the direction of railway profile
  - 1) Configulation and dimension
    - i) End part



### ii) Intermediate part



- 2) Own weight footing and buried beam, and weight of earth (column part)
  - 1) End part

= 26.49

ii) Intermediate part

= 30.57'

- 3). Supporting power of piles and calculation of footing
  - i) End part
    - Ordinary case (Pead load) 2.10 case 1

      Axial load at the batton of calumn = 126.95<sup>th</sup>

      Weight of topting, buried beam

      and earth = 55.76

IN = 182.7/t

Reaction beared by one pile  $P = \frac{\Sigma N}{n} = \frac{182.71}{10.5} = 17.40 \text{ Pile} \left\langle 24 \text{ Pile} \right\rangle$ 

• Ordinary case + temporary case (Dead lood

+ Train load and Impact) d=1.0 case 12

Ariel load at the botton of column = 271.90

Weight of footing, buried beam

and earth = 55.7%

ZN = M + Mc

Reaction beared by one pile  $P = \frac{5N}{n} = \frac{327.66}{10.5} = 31.21 \frac{1}{pile} < 36 \frac{1}{pile}$ 

Earthquake case (Dead Load)
 + Seismic load) L=1.5 case 21
 Axial load at the bottom of calumn
 90.25 × 1.5 = 135.38<sup>7</sup>

Weight of footing, buried beam and earth = 55.76"

IN = 191.14+

Reaction beared by one pile

P = IN = 191.14 = 18.20 The < 48 Pile

Horizontal resisting force beared by one pile Horizontal force at the bottom of calumn

8.51 × 1.5

= 12.77 t

Footing and buried beam

(3.74 + 24.49) × = 25 × 0.1 = 0.85"

Horizoncal force of half partion of column

25 x 0.70 x 0.70 × 7.918 x 1/2 x 0.10 = 0.48

ZH = 1410+

H = 14.10 = 1.34 Pple

#### ii) Intermediate part

• Ordinary case (Dead load) d=1.0 case 1

Axial load at the botton of calumn = 124.71<sup>t</sup>

Weight of tooting, buried beam

and earth = 64.54

IN = 189.25t

Reaction beared by one pile  $P = \frac{\Sigma N}{n} = \frac{18925}{10.5} = 18.02 \text{ pile} \left( \frac{24 \text{ pile}}{n} \right)$ 

• Ordinary case + temporary case (Dead lod + Train load and Impact) &= 1.0 case 15 Arial load at the botton of column = 25%. 44 Weight of footing, buried beam

and earth

= 84 54"

IN = 323.78+

Reaction beared by one pile  $P = \frac{\Sigma N}{\pi} = \frac{323.98}{10.5} = 30.86 \text{ pile} < 36 \text{ pile}$ 

Earthquake case (Dead load
 + Seismic load) L=1.5 case 21
 Axial load at the bottom of calumn
 = 128.46<sup>t</sup>

Weight of footing, buried beam

and earth

= 64.54"

IN = 193,00°

Reaction beared by one pile

P = \frac{\int N}{n} = \frac{193.00}{10.5} = 1838 \frac{7}{Ale} < 48 \frac{7}{10.6}

Horizontal resisting force beared by one pile Horizontal force at the bottom of calumn  $9.31 \times 1.5 = 13.97^{t}$ 

Footing and buried beam

(3.74 + 30.57) × 27 ×0.10 = 0.96"

Horizontal force of half partion of calumn

2.5 x0.70 x0.70 x Z 918 x/2 xa10= a+8 4

EH = 15.41 +

H = 15.41 = 147 Pile

## 4). Bending moment calculation analysis is made calculated bending moment at the calumn from.

Bending moment caused by own weight of footing 1.25

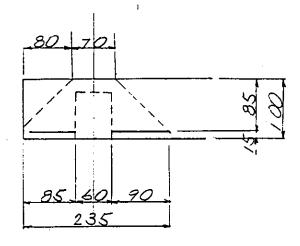
N, = 2,5 × 1.25 × 0.80 × 3.65 = 9.13 N2= 2.5 x 1.15 x 0.20 x /2 x 3.85 = 1.05 N3 = 25 x 0.10 x 0.20 x 3.65 = 0.18 N4 = 1.8 x 1.25 x 1.00 x 3.65 = 8.21 NS = 1.8 x 1.15 x 0.20 x /2 x 3.65 = 0.76

Ma= 9.13 x 0.625 + LOS x a+83 + a/8 x 0.05 + 8.2/x a 625 + 0.76 x a.867 = 12-01 em

IMa = P.n. X - Ma

			P	Æ		· · · · · · · · · · · · · · · · · · ·		(tm)
		2	Reaction of Pile	Number of piles	arm <u>loverage</u>	p.n.x	-Ma	ΣMa
	Ordinary	-	17.40		0.85		-12-01	39-7
part	Ordinary + Temporary	1.00	3/.21	4	· 7	92.85	4	80.8
	Earthquake	1.50	18.20	4	4,	54.15	,	42.1
Inter- mediate	Ordinary	100	18-02	<b>4</b> /	4	53.61	5	10
	+ Temporary	100	30.86	и	5	91.01	5	1.73
ert	Earthquake	1.50	1838	ל	5	54.68	5	ART S

#### 5) Stress calculation



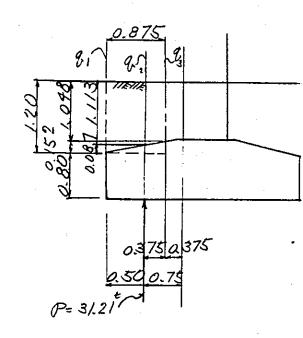
$$\rho = \frac{74.95}{175 \times 85} = 0.00504$$

From the Nomogram M-1,

$$\hat{V}_{C} = \frac{80.84 \times 10^{5}}{175 \times 85^{2}} \times 6.98 = 44.6 \text{m}^{2} \angle 90^{\frac{1}{2} \text{cm}^{2}}$$

Check, safe against cracking

# 6) Calculation of shearing fore caused by beading



Imposed load

$$f_0 = \frac{2}{a_0/d} = \frac{2 \times d}{a_0}$$

$$a_0 = \frac{h}{2} = \frac{h \cdot \infty}{2} = 0.50^m$$

$$V_0 = \frac{2 \times 0.802}{0.50} = 3.208 \angle 4$$

Therefore, to= 3,208

$$q_3' = \frac{q_3}{r_0} = \frac{4.27}{3.208} = 1.33$$

$$w = (4.16 + 4.22) \times /2 \times 0.50 + (4.22 + 1.33)$$
$$\times /2 \times 0.375 = 3.14$$

Considered the full with,

$$rac{2}{a/d} = \frac{2 \times 0.802}{0.75} = 2.139 < 4$$

$$P' = \frac{P}{T} = \frac{109.24}{2.139} = 51.07^{+}$$

- 2. Analysis in the direction of railway cross section
  - 1) Weight of longitudinal buried beam and earth (Per one calumn)
  - i) End part

Buried beam  $2.5^{1/3} \times 0.60 \times 0.80 \times 3.22 = 3.86^{t}$ Earth  $1.8^{\circ} \times 0.60 \times 1.20 \times 3.22 = 4.17^{\circ}$  $\Sigma N = 8.03^{t}$ 

Horizonxal force of helf parties of calumn  $2.5^{4m_{\times}^{3}} 0.70 \times 0.70 \times 3.528 \times 0.10 = 0.43^{t}$ Euried beam  $3.86 \times \frac{0.7}{2.5} \times 0.10 = 0.11^{\circ}$  $\Sigma H = 0.54^{t}$ 

Intermediate part

Buried beam  $2.5^{+3} \times 0.60 \times 0.80 \times (3.22 + 3.40) = 7.94^{t}$ Earth  $1.8 \times 0.60 \times 1.20 \times (3.22 + 3.40) = 8.58^{t}$  $\Sigma N = 16.52^{t}$ 

Horizontal force of half partion of column  $2.5^{\frac{4}{10}} \times 0.70 \times 0.70 \times 3.708 \times 0.10 = 0.45^{t}$ Buried beam  $7.94 \times \frac{0.7}{2.5} \times 0.40 = 0.22^{t}$   $\Sigma H = 0.57^{t}$ 

## Stress at the bottom of column

End part

Ordinary case (Dead Lood + Electrification load) case 21

$$N_3 = 130.07 + 203 = 132.10^{t}$$

$$H_3 = 0.56^{t}$$

$$\begin{cases} M_4 = 1.68 \text{ tm} \\ N_4 = 1.35.43 + 8.03 = 143.46^{t} \\ H_4 = 0.44^{t} \end{cases}$$

Ordinary case + Temporary case ( Pead load + Train load

and Impact + Electrification load) d=1.00 case 24

$$\begin{cases} M_3 = 1.47^{4m} \\ N_3 = 237.86 + 8.03 = 245.89^{t} \end{cases}$$

$$M_{4} = 2.18^{tm}$$

$$N_{4} = 243.22 + 8.03 = 251.25^{t}$$

$$H_{4} = 0.65^{t}$$

Ordinary case + Temporary case (Pead load + Train load) and Impact 1 + lateral load + Electritication load) case 27

$$\begin{cases} M_3 = 15.21 \times 1.15 &= 17.49^{tm} \\ N_3 = 115.89 \times 1.15 + 8.03 &= 141.07^t \\ H_3 = 4.07 \times 1.15 &= 4.68^t \end{cases}$$

$$\begin{cases} M4 = 15.39 \times 1.15 &= 17.70^{-10}, \\ N4 = 208.92 \times 1.15 + 8.03 &= 248.29^{t}, \\ H4 = 4.14 \times 1.15 &= 4.76^{t} \end{cases}$$

Ordinary case + Temporary case (Dead load + Train load and Impact 2 + lateral load + Electrification load) Case 28

$$\begin{cases} M_3 = 14.34 \times 1.15 &= 16.49^{\pm m} \\ N_3 = 201.71 \times 1.15 + 8.03 = 240.00^{\pm} \\ H_3 = 3.97 \times 1.15 &= 4.57^{\pm} \end{cases}$$

$$\begin{cases}
 M_4 = 14.96 \times 1.15 \\
 N_7 = 216.69 \times 1.15 \\
 H_7 = 4.24 \times 1.15
 \end{cases}
 = 17.20 \text{ tm}$$

$$\begin{cases}
 N_7 = 216.69 \times 1.15 \\
 H_7 = 4.24 \times 1.15
 \end{cases}
 = 4.88 \times 1.15$$

Earthquake case (Dead load + Seismic load + Electrification load) d=1.50 case 35  $M3 = 30.41 \times 1.50 = 45.62^{tot}$   $M3 = 75.29 \times 1.50 + 8.03 = 120.97^{t}$   $H3 = 8.55 \times 1.50 + 0.54 = 13.37^{t}$   $M4 = 30.21 \times 1.50 = 45.32^{tot}$   $M4 = 101.73 \times 1.50 + 8.03 = 160.63^{t}$   $M4 = 8.47 \times 1.50 + 0.54 = 13.25^{t}$ 

#### ii) Intermediate part

Ordinary case (Dead lood) &= 1.00 case /

$$M_3 = 0.30^{tm}$$

$$N_3 = 119.67 + 16.52 = 136.19^{t}$$

$$H_3 = 0.12^{t}$$

$$\begin{cases} M4 = -0.30^{tm} \\ N4 = 119.67 + 16.52 = 136.19^{t} \\ H4 = -0.12^{t} \end{cases}$$

Ordinary use + Temporary case (Pead load + Train load

$$\begin{cases} M3 = -2.15^{4m} \\ N3 = 224.21 + 16.52 = 240.73^{t} \\ H3 = -0.87^{t} \end{cases}$$

$$\begin{cases} M4 = 2.15^{tm} \\ N4 = 2.24.21 + 16.52 = 240.73^{t} \\ H4 = 0.87^{t} \end{cases}$$

Ordinary case + Temporary case (Pead load + Train load and Impact + lateral load) 
$$d = 1.15$$
 (ase 1)

$$M3 = 13.51 \times 1.15 = 15.54^{tm}$$

$$N3 = 101.63 \times 1.15 + 16.52 = 133.39^{t}$$

$$H3 = 3.22 \times 1.15 = 3.70^{t}$$

$$M4 = 15.11 \times 1.15 = 17.38^{tm}$$

$$N4 = 197.41 \times 1.15 + 16.52 = 243.54^{t}$$

$$H4 = 3.87 \times 1.15 = 4.45^{t}$$

Ordinary case + Temporary case (Dead load + Train load and Impact 2 + lateral lead) d = 1.15 case 12  $M3 = 11.52 \times 1.15 = 13.25^{tm}$   $N3 = 189.81 \times 1.15 + 16.52 = 234.80^{t}$   $H3 = 2.79 \times 1.15 = 3.21^{t}$   $M4 = 15.26 \times 1.15 = 17.55^{tm}$   $N4 = 200.14 \times 1.15 + 16.52 = 246.68^{t}$   $H4 = 4.30 \times 1.15 = 4.95^{t}$ 

# Earthquake case (Dead load + Seismic load)

$$M_{3} = 29.16 \times 1.50 = 43.74^{em}$$

$$M_{3} = 68.61 \times 1.50 + 16.52 = 119.44^{t}$$

$$H_{3} = 7.75 \times 1.50 + 0.67 = 12.30^{t}$$

$$H3 = 7.75 \times 1.50 + 0.67 = 12.30^{t}$$

$$\begin{cases} M = 28.76 \times 1.50 = 43.14 \\ M = 90.96 \times 1.50 + 16.52 = 152.96 \\ H = 7.59 \times 1.50 + 0.67 = 12.06 \end{cases}$$

### 2) Stability calculation

		Allowable Supporting Power	Į.	Pmax	Pmin	Hmax
	Ordinary	24 t	21	18.67	17.44	0.05
Eand Part	Ordinary + Temperary	36 °	1	33.82		1
	Earthquake	48 t	!	25.45		
	Ordinary	24 t	1	17.61	17.61	0
Intermediate Park	Ordinary Ordinary + Temporary	36 *	11		11.83	' ·
	Earthquake	48 t	17	2 <del>4</del> .27		

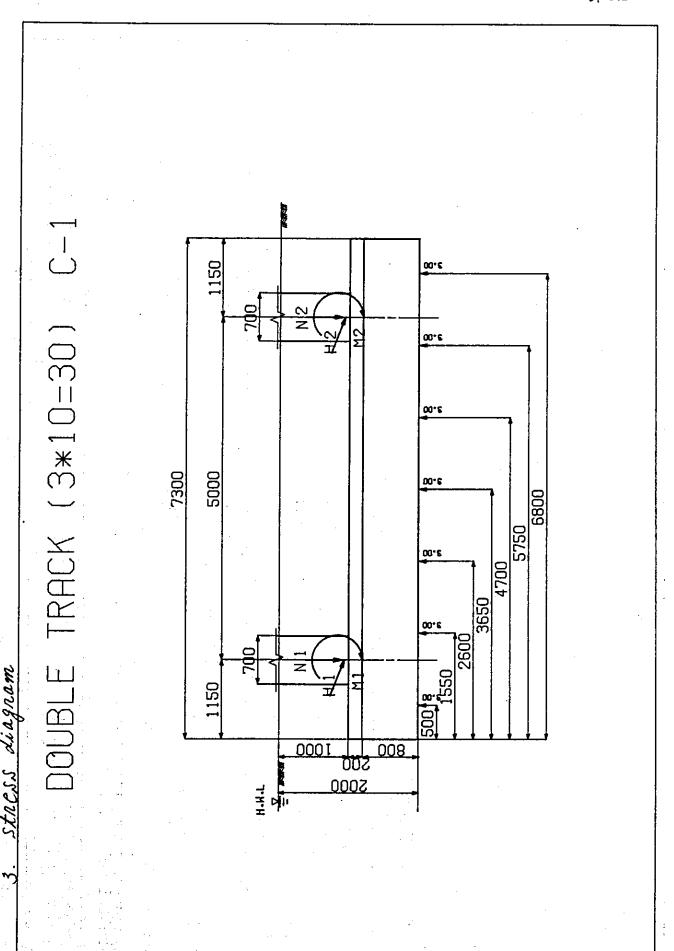
Analysis on the body of pile  $\beta = 0.298^{m-1}$ 

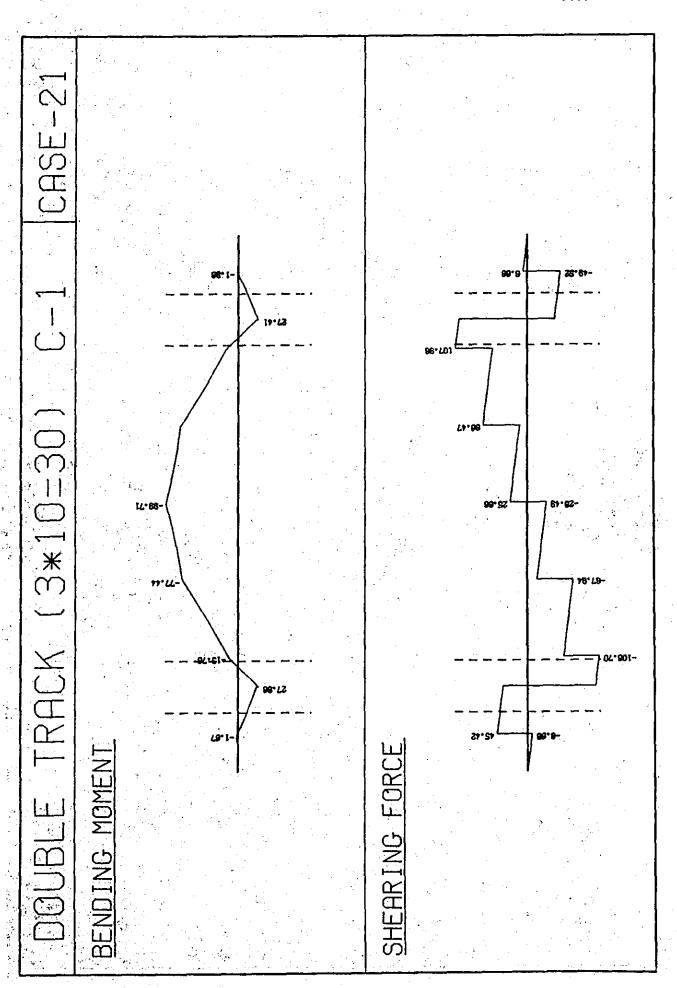
In direction of railway profile  $H = 1.47^{t}$   $M = 0.322 \times \frac{H}{B} = 0.322 \times \frac{1.47}{0.298} = 1.59^{tm}$   $N = 18.38^{t}$ 

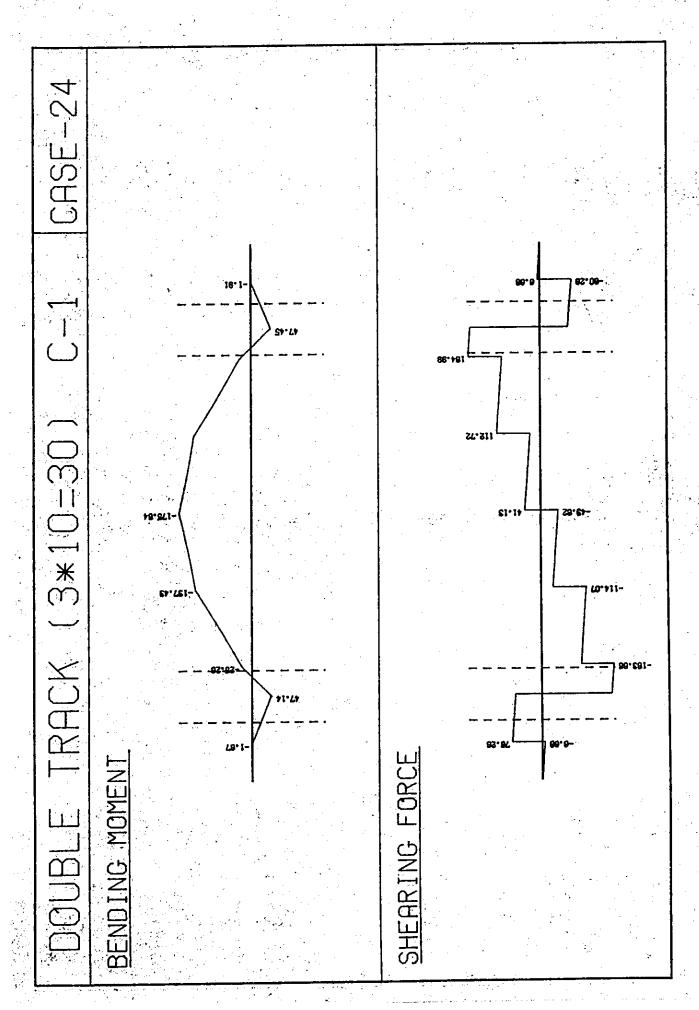
In direction of railway cross section  $H = 1.33^{t}$   $M = 0.322 \times \frac{1.33}{0.298} = 1.44^{tm}$   $Nmax = 2706^{t} (sN considered)$ 

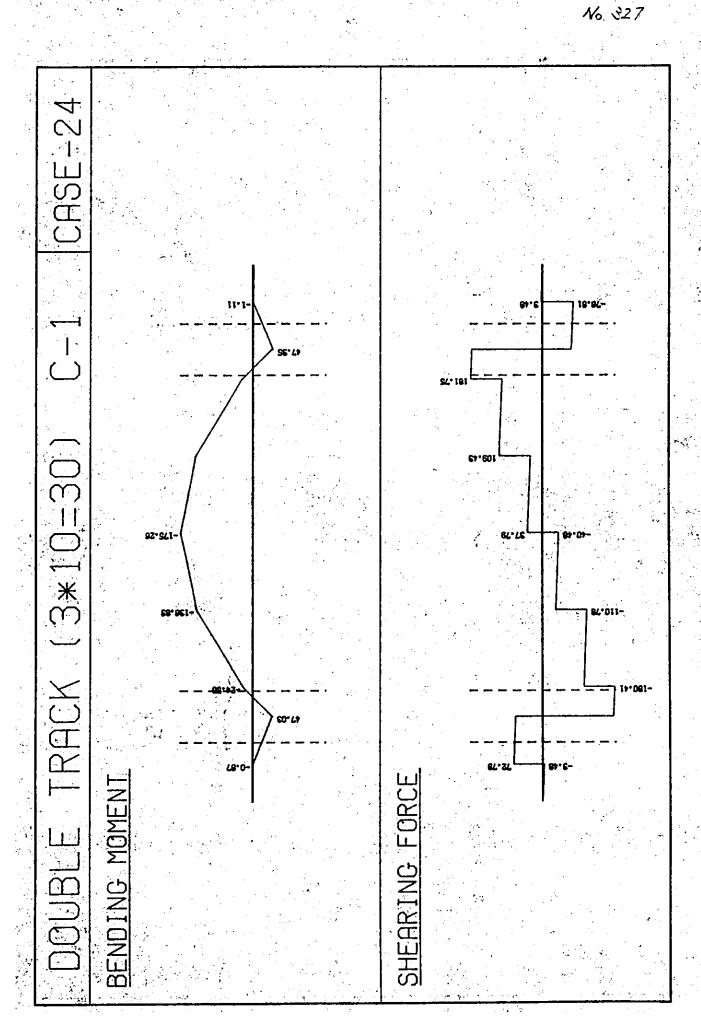
Nmin = 9.05 t ( & N considered)

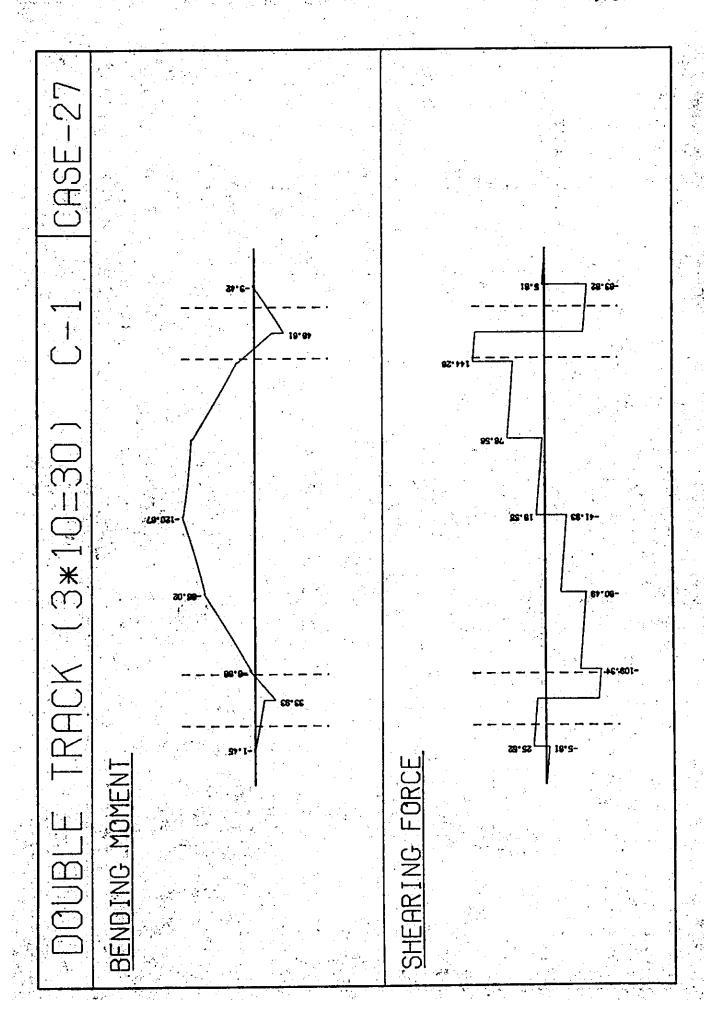
According to the "Interaction curve", Kind B is employed.

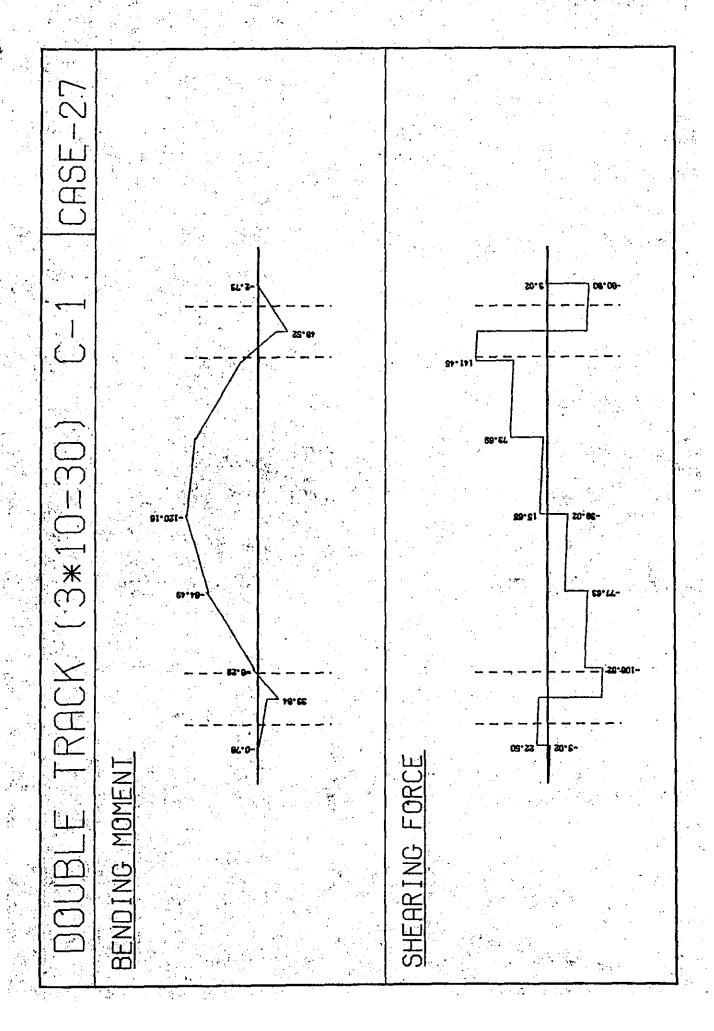


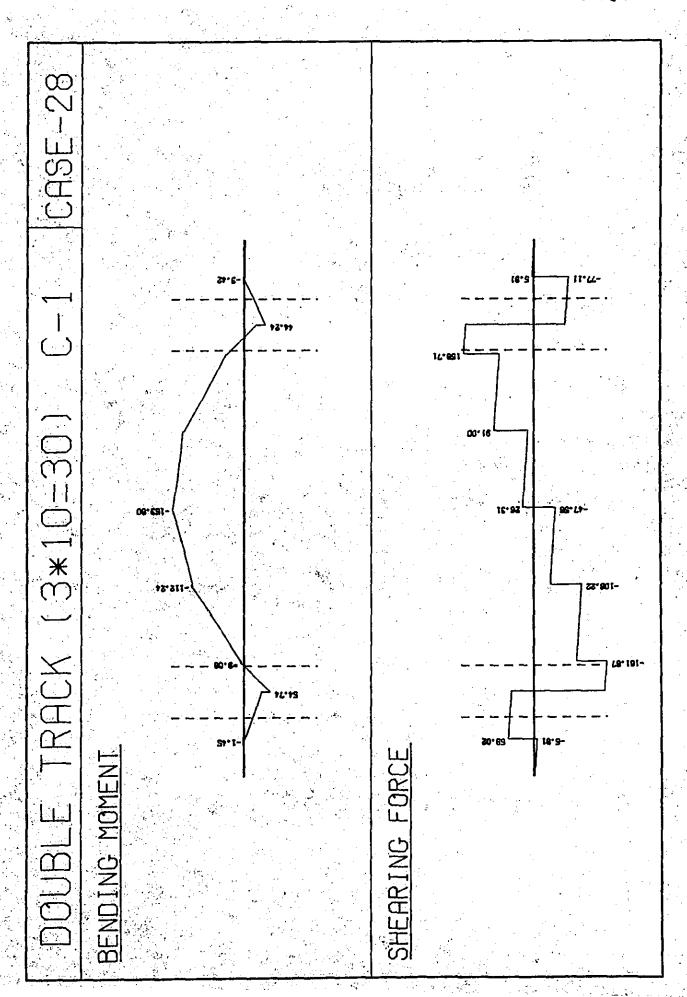


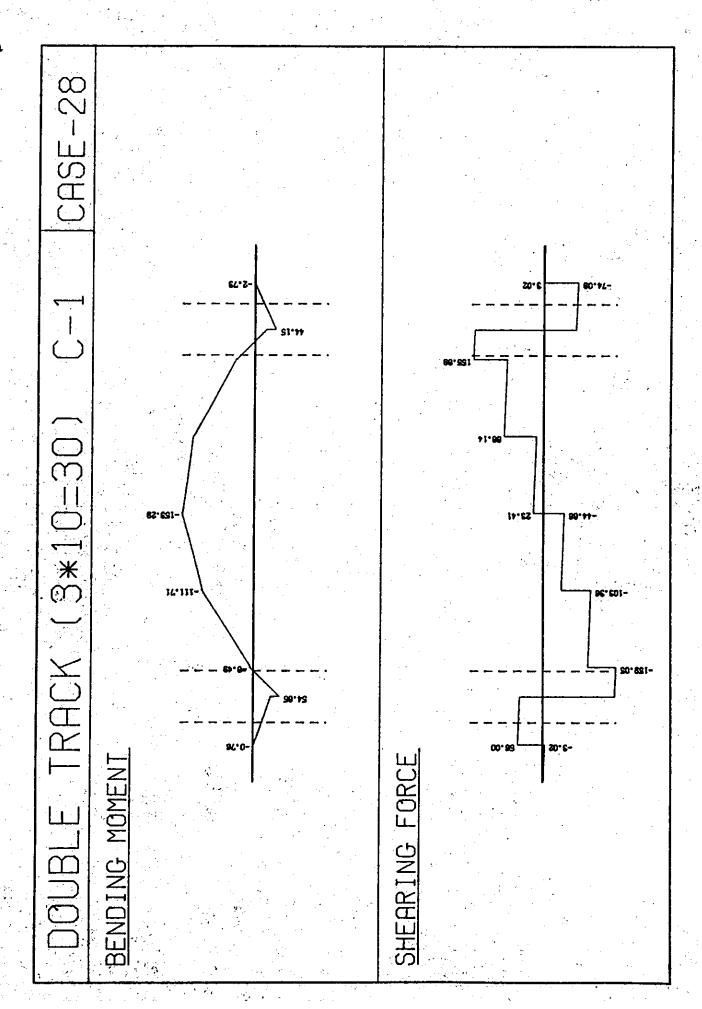


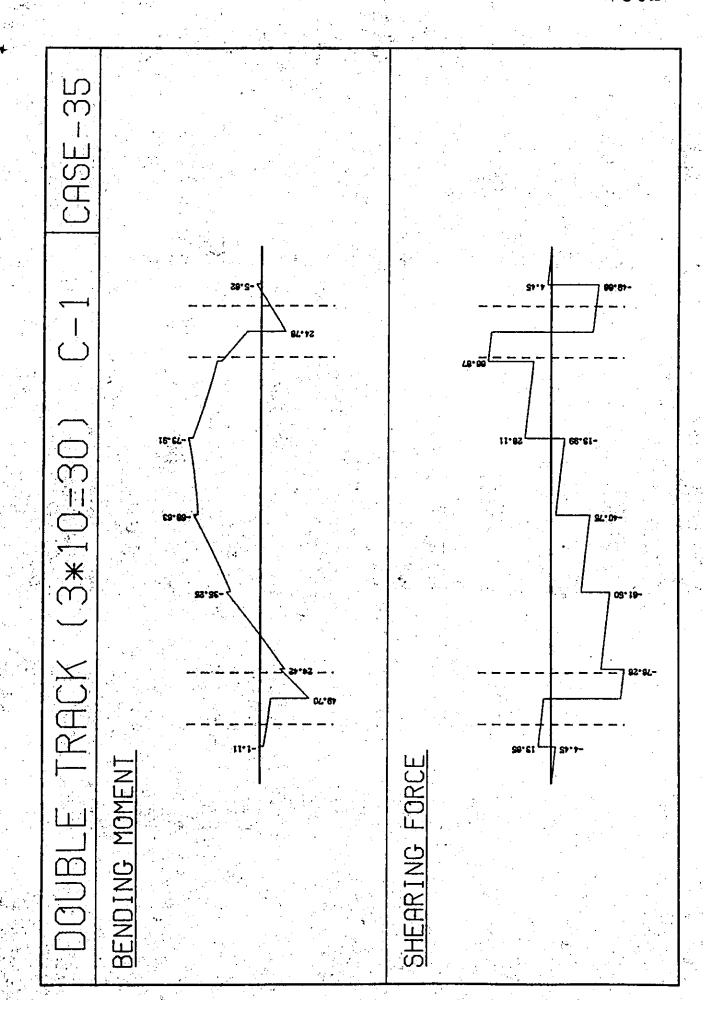












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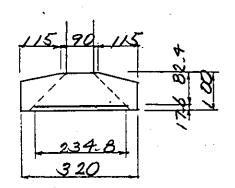
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- 1) Stress calculation
- a) End park
  - i) Botton side

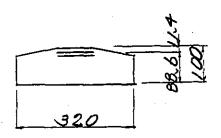


$$A_s = 0/9 - 15^{cm} ctc$$

$$= 2.865 \times \frac{234.8}{15} = 44.85^{cm}$$

$$P = \frac{44.85}{234.8 \times 82.4} = 0.00232$$

## ii) Tep side



$$p = \frac{127.07}{320 \times 88.6}$$

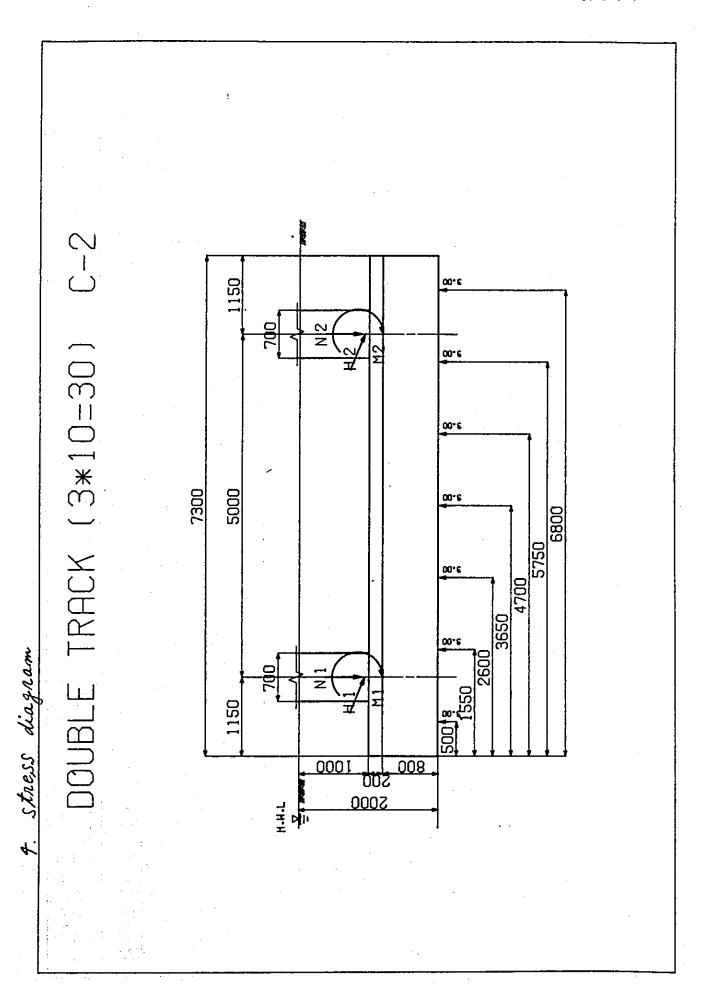
From the Nomagram M-1

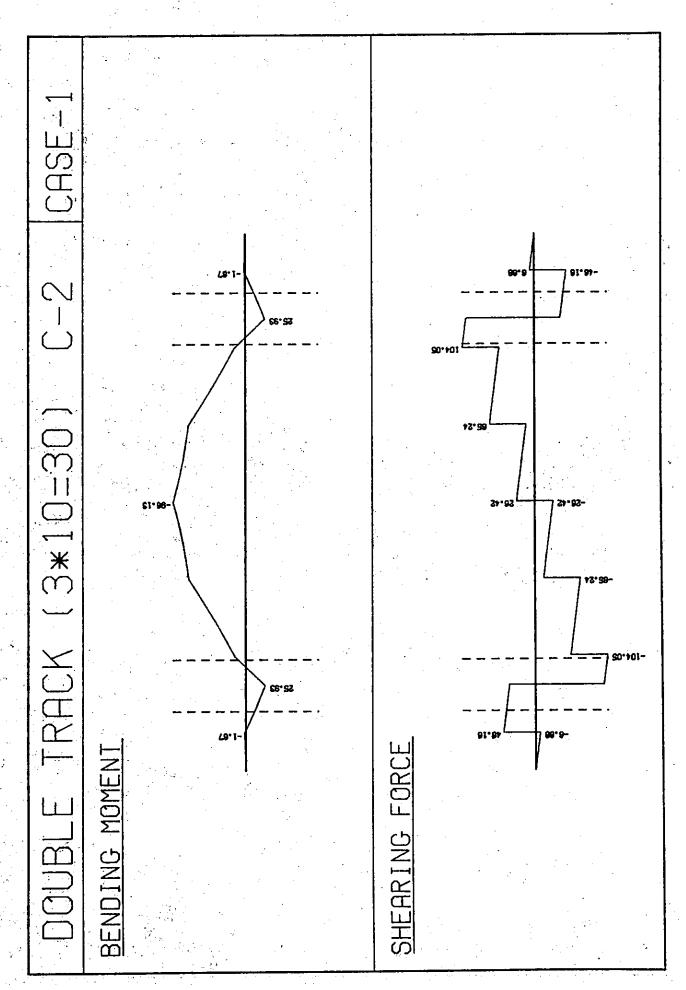
$$\widehat{V}_{c} = \frac{175.84 \times 10^{5}}{320 \times 88-6^{2}} \times 7.29 = 51.0^{\frac{kg}{00^{2}}} \langle 90^{\frac{kg}{00^{2}}} \langle 90^{\frac{kg}{00^{2}}} \rangle$$

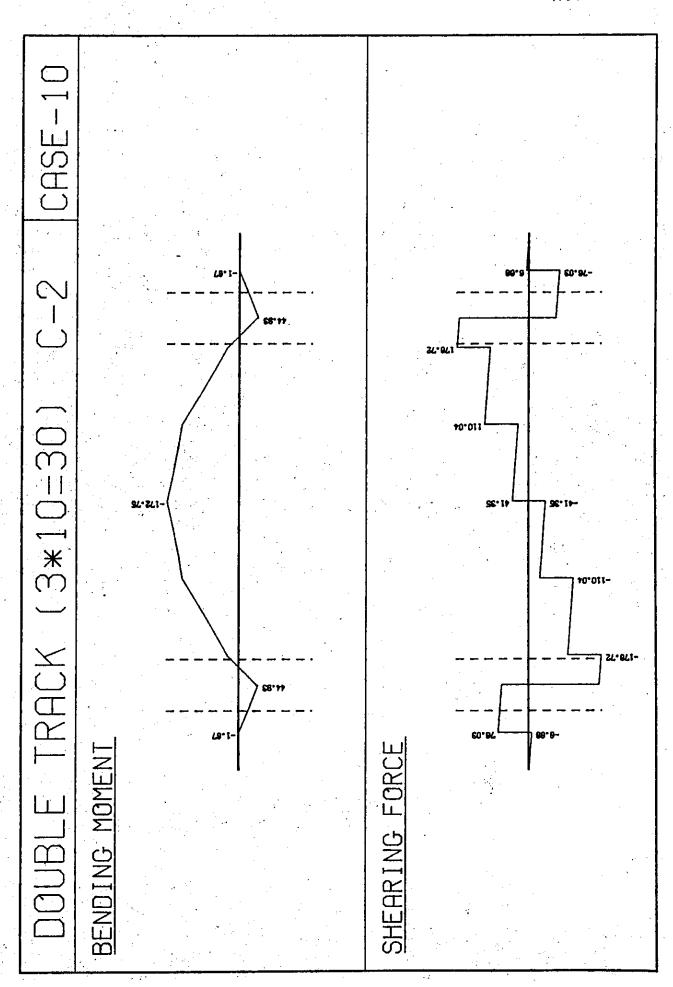
$$\widehat{V}_{5} = \frac{175.84 \times 10^{5}}{320 \times 88-6^{2}} \times 7.29 = 51.0^{\frac{kg}{00^{2}}} \langle 90^{\frac{kg}{00^{2}}} \rangle$$

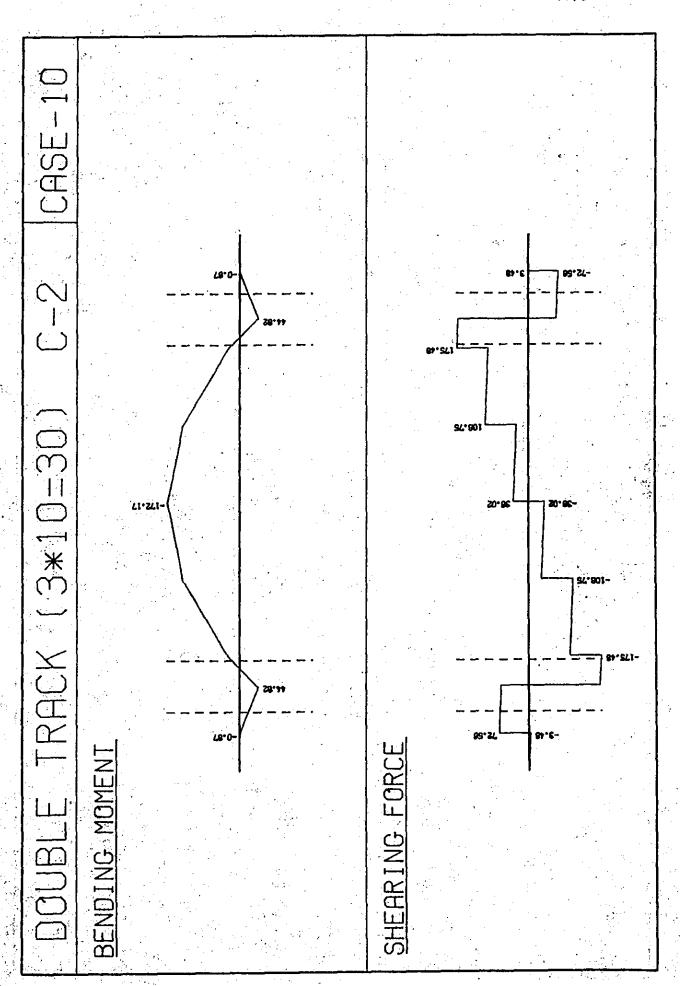
Check, safe against cracking
$$M = -99.71^{\circ m}$$

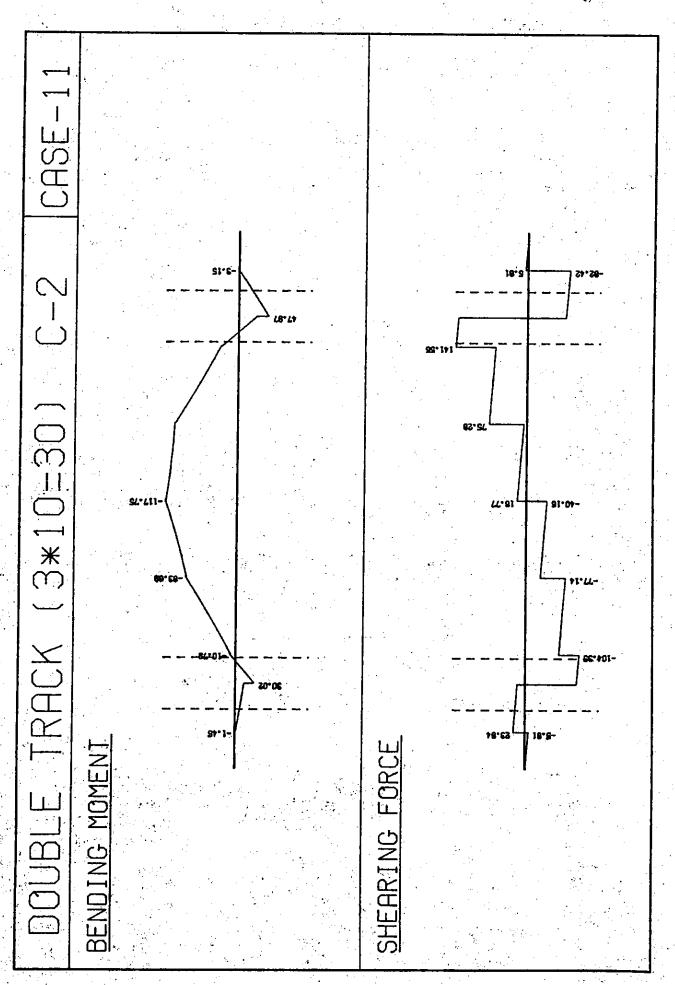
2) Shearing stress 1/2 paint from column front

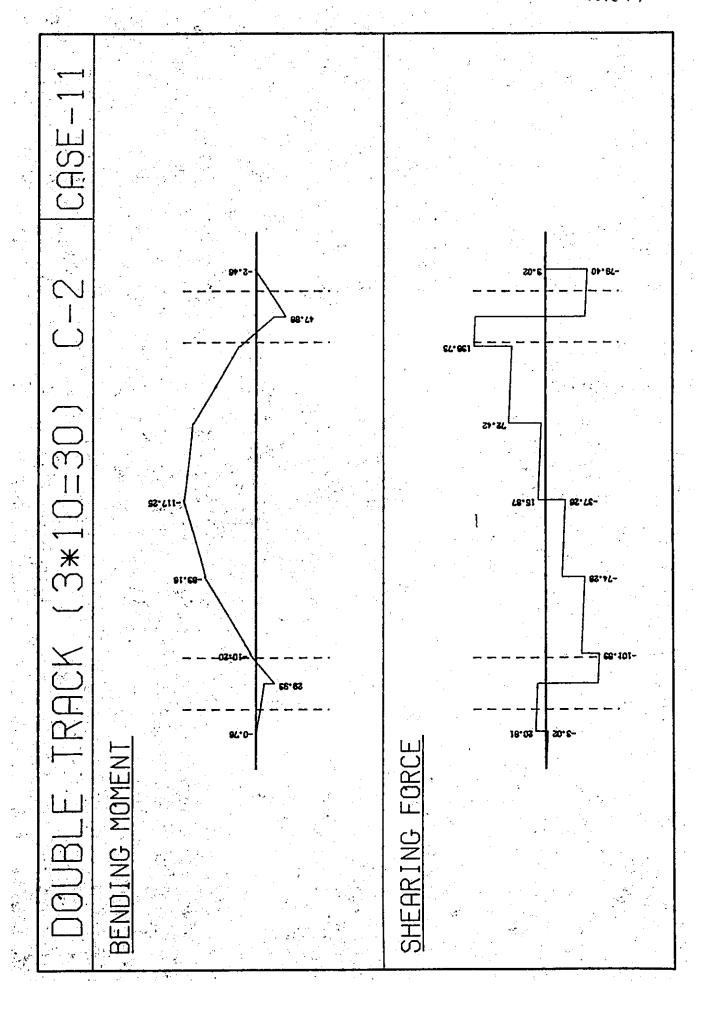


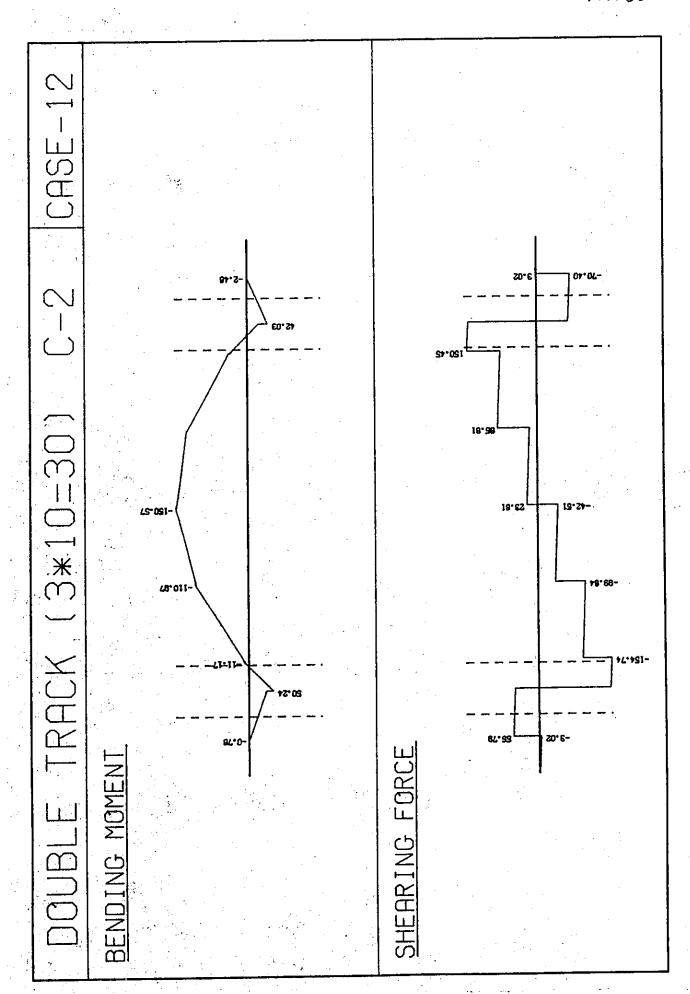


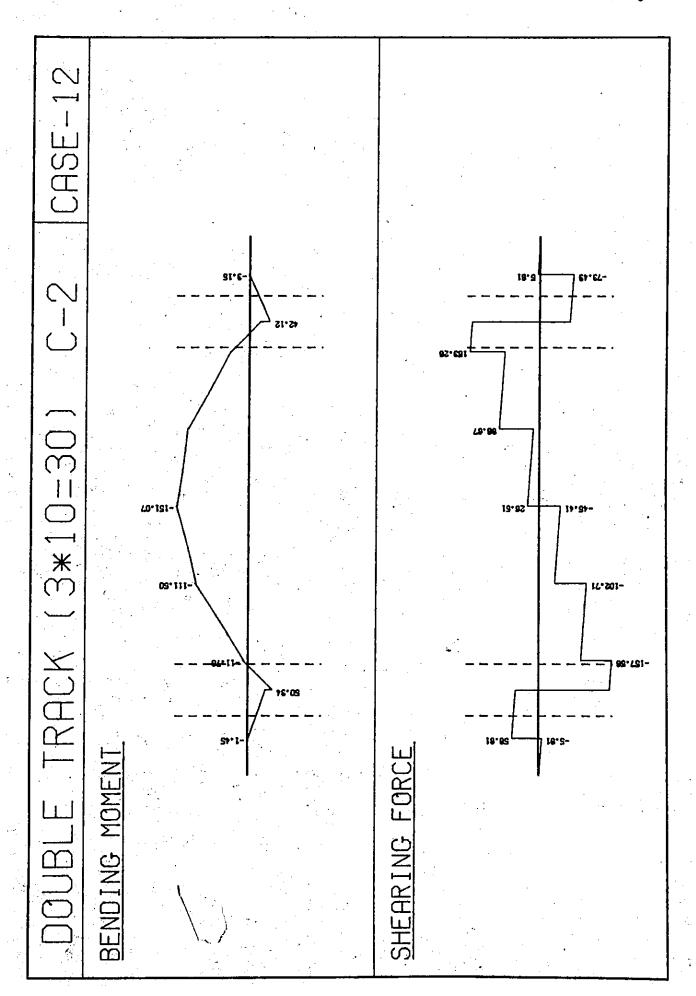


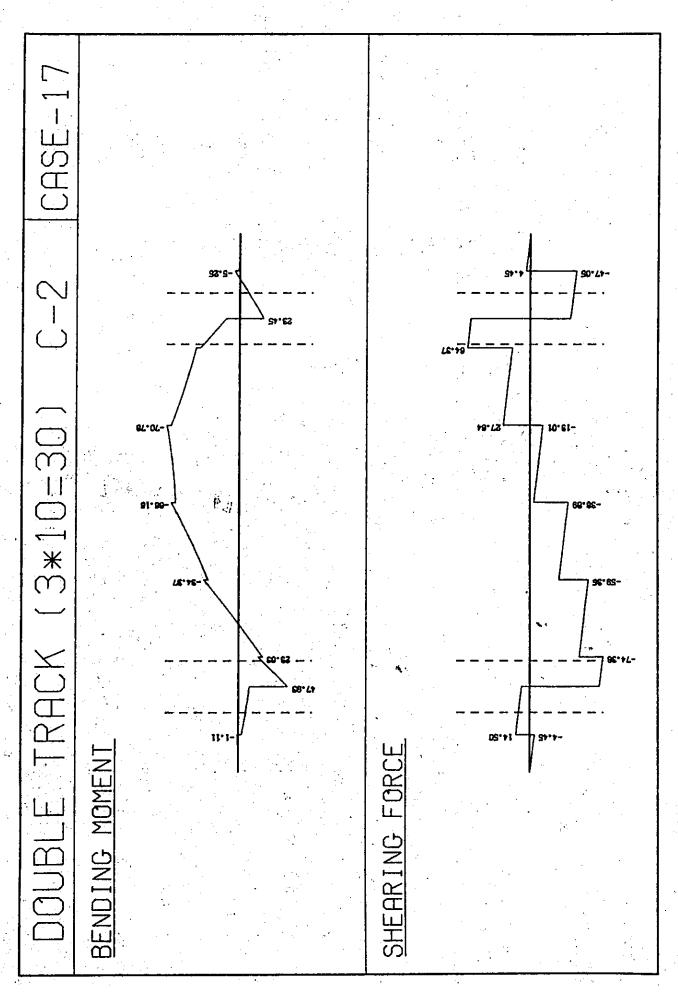












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ATA CARD 1	OUBLE TRAC	2 2	9.0	0.8	2	1.15	2.0	0.7	21.0	5.0	٣	2.0	0.30	0.12	136,19			240.73	-2.15	-0.87	240.73	15.54	3.70	133.39	15.54	3.70	133,39	13.25	3.21	234.60	13.25	3.21	234.60	43.74	12,30	119.44	i	a1 ~	
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INPUT DATA

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21.000 ( H ) KUIKEI = 0.350 ( H )	0.000621 (H**4) E = 0.350E+07 (T/H*#2)	91.000 (174*•3) HENIRYDU≈ 0.000 ( H )	•	 NSW HONSU	3.000	3.000 3.000 3.000	3.000			H = 1.52 (T) Y = 0.466 (H)		H = 1.21 (1) Y = 1.460 (M)		
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DOUBLE\_TRACK (3+10=30) G-2

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PICK UP DOUBLE TRACK (3010=30) G-2

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\*\*\* PICKUP - 1 \*\*\*

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1.1	1.150 (	7	25.934	-98.710 (	1)	25.934	-96.710		
	. 500 (	=	-9.432	-103.385 [	1	-9.432	-103,385		
5	.550 (	<b>:</b>	-14.618	-104.053 (	7	-14-618	-104.053	•	
1.5	550 (	13	-14, 618	-51.211 (	=	-14.618	-51.211		
2.0	100.	=	-39-016	-57,222 (	13	-39,016	-57.222		
2.6	. 600 t	=	-75.753	-65.237 (	7	-75.753	-65,237		
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3	700 (	1	-75.753	65.237 (	1	-75.753	65.237		
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5.5	5.750 (	=	-14.618	51-211 (	3	-14.618	51.211		
Į,	5.750	7	-14.618	104-053 (	2	-14.618	104-053		
5.4	900 (	=	-9.432	103.385 (	13	-9-432	103.385		
9	150 (	=	25.934	98.710 (	13	25.934	98.710		
6.1	150 (	13	25.514	-37.480 (	7	25.514	-37.480		
6.500	500 (		11.578	-12.155 (		11.578	-42,155		
6.80	900	1	-1.670	-46.163 (	=	-1.670	-46.163		
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<b>E</b>	0.000	-1.670	-1.670	11.578	514	25.934	:	-14-510	-75.753	-75.753	-96-132	-96,132		-75.753	-39-016	-14.618	-14.618	-9.435	25.934	25, 514	11.578	-1-670
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DOUBLE TRACK (3+10=38) C-2

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DOUBLE TRACK (3\*10=30) C-2

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. !	005*0	113	-0.756	-3.025	( 10)	-1.670	-6.679		
	0.500	10)	-1.670	76.031	( 17)	3.027	14.503	The community of the co	
•	0.800	(07)	20.539	72,024	( 17)	6.978	11.832		
•	1-150	10)	44.823	68.035	( 171	10.573	8.715		
•	1-150	173	47.933	-70.817	(10)	61.90	-173.381		
	1.500	173	22.602	-73.934	( 10)	-19.593	-178.056	The state of the s	· · · · · · · · · · · · · · · · · · ·
	1.550	173	16.894	-74 379	_	-28.512	-178.724		
	1.550	(11)	23.034	666.64-	(01)	-27.832	-99.443	The control of the co	:
	2.000	171	-0.367	-54.006	( 1	-73.286	-102.574		
	2.600	( 17)	-34.373	-59,349	( 10)	-136.690	-110.039		
	2.600	(10)	-136.690	-27.338	(21)	-109.264	-36.160		
	3.650	113	-117.246	-37.262	(112)	-151-075	-45.414	A CONTRACTOR OF A STATE OF A STAT	
	3.650	( 10)	-172,750	41.355	(17)	-62,022	-3.665		
	4.700	121	-136.082	30.713	( 17)	-70.779	-13.015		:
	4.700	(10)	-136.690	110.039	( 17)	-66, 639	27.639		
	5.300	101	-73.286	102.574	( 17)	-51.658	22.296		
	5.750	(10)	-27.632	99.443	(17)	-42.527	18,289		
	5.750	(113)	-28.512	178.724	(71)	-38.386	64.367		; ; ;
	5.600	(01)	-19.593	178-056	( 17)	-35.179	63,922		
*	6.150	(10)	41.909	173,381	( 17)	-13.352	60.805		
•	6.150	( 17)	23.448	-41.262	(11)	47.876	-75.469		
	6-500	(71)	8.461	-44-379	(11)	21.051	-78.938		
	6.800	( 17)	-5.253	-47.051	(11)	-3.153	-82-422		
	6.800	101	-1.670	6.679	(11)	-0-756	3.025	A CONTRACT OF A CONTRACT OF THE CONTRACT OF TH	
	4 450		6	666			666		

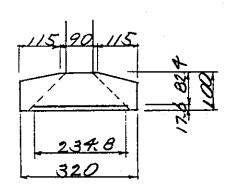
DOUBLE TRACK (3\*10=30) C-2

No.360

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11 31.821 1.1.557 1.2.3	11 33,821 11 31,597 12 30,373 12 28,140 17 25,751			
12 26.16 26.476 1.227 2.478 1.227 2.478	12 30,373 12 28,148 17 25,751			
28.72	17 25.751			
		!		:
		:		<u> </u>
	•	:		
			-	

## 1) Intermediate part

#### i) Bottom side



$$A_{s} = 0/9 - 15^{cm} c c$$

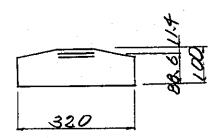
$$= 2.865 \times \frac{234.8}{15} = 44.85^{cm}$$

$$P = \frac{44.85}{234.0 \times 02.4} = 0.00232$$

From the Nomogram M-1, 1/Lc = 9.36 1/Ls = 467

$$\widehat{V}_{C} = \frac{50.34 \times 10^{5}}{2348 \times 82.4^{2}} \times 9.36 = 29.6^{\frac{100}{100^{2}}} \langle 90^{\frac{100}{100^{2}}} \langle 90^{\frac{100}{100^{2}}} \rangle$$

## ii) Tep side



From the Nomagram M-1

$$\hat{V}_{C} = \frac{172.75 \times 10^{5}}{320 \times 88.6^{2}} \times 7.29 = 50.1^{\frac{Kg}{lcm^{2}}} < 90^{\frac{Kg}{lcm^{2}}}$$

Check, safe against cracking

(set = 96.13 × 105 320 × 88.62 × 248 = 950 Kg/cm² < 1600 Kg/cm²

# 2) Shearing stress 1/2 point from column front

$$S = 102.57^{t}(case 10)$$

$$F = \frac{2 \times 82.4}{100} = 1.498 < 4$$

$$P = 25.35 \times 3 = 76.02^{t}$$

$$P' = \frac{76.02}{1.498} = 50.75^{t}$$

$$S = 102.57 - (76.02 - 50.75) = 77.30^{t}$$

$$7 = \frac{77.30 \times 10^{3}}{320 \times 82.4} = 2.93 \% \times (3.9 \% )$$

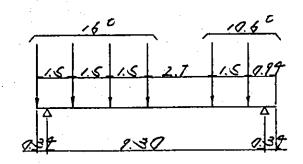
- 5.10. Calculation of shoes and beam suggesting parts
  - (1) Calculation of shoes
    - 1. Load Calculation
    - (1) Dead lead

Reaction of T section simple beam  $R = 118.90^{-t}$ 

Reaction per one Shol

Rd = 118.90 × 1/2 = 59.95

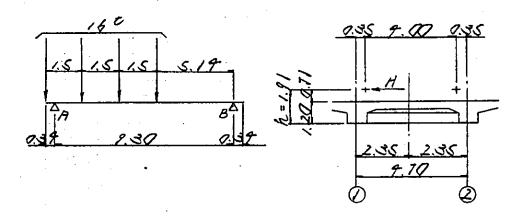
(2) Train load + Impact



Inpact coefficient i= 0.917

Reaction per one shoe

# (3) Lateral load caused by rolling stocks



$$RA = \frac{1}{1.30} \times 16 \times 9 \times 7.39 = 50.86^{t}$$

$$H = 50.86 \times 0.15 = 7.63^{t}$$

$$N = \frac{1}{1.60} + \frac{1.60}{1.60} \times \frac{1.91}{1.00} = \frac{1}{1.00} \times \frac{1.91}{1.00} = \frac{1}{$$

// // //	(4)	Sunmary	of	reactions	of	deam
----------	-----	---------	----	-----------	----	------

		Ø	2
	Dead Load	59.95	59.95
ad	Train load + Impact	77.52	77.52
20	Lateral load of Rolling Stocks	3.10	-3.10
tical	Dead load + Train load + Impact	136.97	136.97
7	Dead load + Train Load + Impact +.	190.07	/33.87
	Latral load (d=1.15)	(121.80)	(116.91)
Hori Eontal Farce	Seismic Load	1.13	7.13

- · Values in () are the converted figures of nomal condition
- · Seisnic load is regented the Explanation in Page 277 Explanation

- 2. Calculation of rusber shoes
  - (1) Required area for supporting load

$$15 \leq \frac{R}{10} \leq 80$$

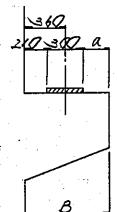
Hence,

$$\frac{Rmox}{80} \leq A \leq \frac{Rmin}{15}$$

$$\frac{Rmac}{80} = \frac{136.97 \times 10^3}{80} = 1710^{-cm^2}$$

Assumed the size of rubber shoe as

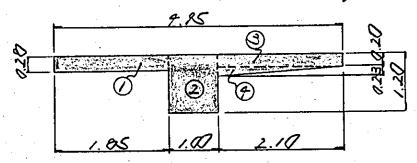
Width of deam supporting part



$$a \ge 150$$
 (l=10 mor less)

:. Say, 700 nm

- (2) Relative di Splacement between beam and substructure
  - (a) Displacement of seam Caused by the deflection of beam: Dla
    - i) Calculation of beam deflection



	b (m)	h (n)	A (2)	g (m)	A. y (n3)
$\bigcirc$	1.850	0.280	0.518	0.190	0.07252
2	1.00	1.200	1.200	9.600	0.72000
3	2.100	0.200	0.920	0.100	0.09200
Ŧ	2.100	0.230/2	0.292	0.277	0.06703
Σ	<u> </u>		2380		0.90155

	b (m)	h (m)	A (n2)	g (M)	Io (MT)	A. y 2 (M+)	I.+ A.g. (R+)
$\bigcirc$	1.050	0.280	0.518	0.239	0.00338	0.02859	0.03297
2						0.05861	e.
3	2.100	0.200	0.720	0.279	0.00190	0.03269	0.03909
<b>(4)</b>	2.100	0.230/2	0.292	0.102	0.0071	0.00252	0.00323
Σ			2.380			0.12391	0.27290
		-	<i>I</i> =	0.2728	O nt		

Magnitude of vertical deflection at the Beam Center Point

$$E = 2.7 \times 10^{6} \text{ m}^{2}$$

$$I = 0.27290^{m^{2}}$$

(a) Deflection Coused by dead load

Uniformly distributed load (From reaction

Calculation of T section simple heam) Ra = 118.90 - 5.96 = 112.99  $Wd = \frac{112.99}{10.00} = 11.29 \frac{5}{n}$ 

$$S_{1} = \frac{SWL^{q}}{369EI} = \frac{S \times 11.29 \times 1.30^{q}}{369 \times 2.7 \times 10^{6} \times 0.27290}$$
$$= 0.00199^{m}$$

(b) Deflection Caused by train load

Maximum bending moment at the Center Point

of Span

$$(KS-16)Ml = 116.89 \times \frac{16}{10} = 103.90^{CR}$$

$$S_{2} = \frac{SM_{1} l^{2}}{90 EI} = \frac{S \times 103.90 \times 9.30^{2}}{90 \times 2.7 \times 10^{6} \times 0.27290}$$

$$= 0.00/27^{M}$$

Deflection Caused by (Train load + Impact)

Solution 1917 = 0.00180 n

- ii) Displacement of seam Caused by bending deflection: alz
  - a) Dead load

$$d = 3.2 \times \frac{5}{l}$$

b) Train load

$$d = \frac{(0.00180)}{9.30} = 0.00056$$

Value in ( ) is the case considered impact

$$\Delta lal = 02.1 \times 0.00056 = 0.05^{cm}$$
 $\Delta lal + i = 02.1 \times 0.00079 = 0.06^{cm}$ 

(b) Displacement of beam Caused by temperature

Change: Alt

 $\Delta lt = \Delta t \cdot d \cdot l$ 

st: Temperature change ± 20°C

d: Coefficient of linear expansion

of beam 1 × 10 /c

l: Span l= (30.00 + 9.30) × 1/2 = 19.65 th

Hence,

Alt = ± 20 × 10 × 19.65 = ± 0.0039 = ± 0.39 cm

(C) Displacement of beam Caused by drying Contraction: Dls

als = Ecs · l

Ecs: Ratio of drying Contraction of concrete 20 × 10<sup>-5</sup>

Hence,

als = 20 × 10 - 8 × 19.65 = 0.0039 = 0.39 cm

(d) Displacement in Norigontal direction in Case of earthouake

Displacement Caused by korizontal force in Cace of earthquake

Relative displacement between beam and substructure

(e) Required thickness Ete

a) Normal Cose

$$\Delta m = \Delta la + \Delta lc + \Delta ls + \Delta le$$

$$= 0.09 - 0.39 - 0.39 = -0.79^{cm}$$

$$\Sigma Ce_1 = \frac{\Delta m}{0.7} = \frac{0.79}{0.7} = 1.06^{cm}$$

b) Temporary Case

an = an + alge = -0.79 + 0.05 = -0.69 "

$$\Sigma te2 = \frac{\Delta n'}{0.7} = \frac{0.69}{0.7} = 0.99$$
 cm

# c) Earthquake Case

$$\Delta E = \Delta la + \Delta lc + \Delta ls + (\Delta e_1 + \Delta e_2)$$

$$= 0.09 - 0.39 - 0.39 - (0.59 + 0.50)$$

$$= -1.03$$

$$\Sigma t_{\text{elg}} = \frac{\Delta E}{2.0} = \frac{1.88}{2.0} = 0.92$$
 cm

Therefore, use te = 12 nm of one layer.

- (3) Restricted tortional strain Corresponding to deflection ange at the support Point
  - (a)  $E\Delta te > \frac{a}{2} tand$ 
    - Sate: Average deformation of rubber shoe in vertical direction (cm)
      - a: Side length of rubber shoe in direction of bridge axis (cn)
      - d: Angle between beam bottom face and support face at the support point
      - $\Delta te = Ct \cdot \frac{f}{G} \cdot \frac{te^2}{a_0 2}$
      - ct: Factor determined by the ratio of both side lengths (Refer explanation Chart 179-1)

f: Bearing stress of rubber shoe in vertical direction (%n²)

(Dead load) 
$$f_d = \frac{Rd}{A} = \frac{51.45 \times 10^8}{30 \times 60} = 33.03 \frac{10}{100}$$

9: Elastic Modulus of rubber shoe in terms of shear ( $\frac{7}{3}$ cn<sup>2</sup>)

Subjected dead load 9 = 6.2  $\frac{7}{3}$ cn<sup>2</sup>

Subjected live load 9 = 0.0"

te: Thickness of rubber shoe, one layer te=12 mm

 $\Delta ted = 1.955 \times \frac{33.03}{6.2} \times \frac{1.2^{3}}{30^{2}} = 0.015^{cm}$ 

 $\Delta teli = 1.955 \times \frac{93.07}{9.0} \times \frac{1.2^2}{30^2} = 0.015$ 

Hence,  $\Delta te = 0.015 + 0.015 = 0.030^{cm}$ 

d = 0.00051 + 0.00079 = 0.00130

Hence,  $\frac{a \cdot tond}{2} = \frac{30 \times 0.00130}{2} = 0.020^{cR}$   $< \sum \Delta te = 0.030^{cR}$ 

(b) Maximum deformation in vertical direction: Estenox

 $\sum \Delta temax = \sum \Delta te + a \cdot tand/2$ = 0.030 + 0.020 = 0.050 cm

 $0.15 \cdot \Sigma te = 0.15 \times 1.2 = 0.18^{ch} > \Sigma \Delta te max$ 

(7) Sagety analysis in terms of buckling when subjected vertical load

 $a, b \ge 5 \cdot 5te$   $a = 30^{cn}, b = 60^{cn} > 5 \times 1.2 = 6.0^{cn}$ 

Analyzes as above, dimensions of rubber shoe are determined as fallows.

Bridge axis direction  $a = 30^{cm}$ Cross Sectional direction  $b = 60^{cm}$ Thickness of the layer  $t = 1.2^{cm}$  Use one layer

Gross thickness 1.9 cm (Including stainless

Steel cover plates)

- (2) Calculation of Stopper made of steel rod
  - 1. Horizontal seismic load applied for the Stopper desigh

    Ksh =  $\Delta f$  · Kh

△+: Extra factar

In direction of bridge axis  $\triangle f = 1.2$ In direction of cross section  $\triangle f = 1.4$ 

Kh: Horizontal seismic load for desigh Kh = 0.10

Horizontal seismic load applied for the

Stopper desigh will be,

Bridge axis Ksh = 1.2 × 0.10 = 0.12 Cross sextion Ksh = 1.4 × 0.10 = 0.14

- 2. Horizontal force acting the stopper
  - (1) Bridge axis

    One unit stopper per one main girder is equipped,

    With semi-rigid Construction.
  - (a) Seismic fonce due to dead weight

    Hsd = Ksh · Rd

    = 0.12 × 59.45 = 7.13 t

- (2) Cross sectional direction
  - (a) Seismic load due to dead weight

    Hsd = Ksh · Rd

    = 0.19 × 59.95 = 8.32 t

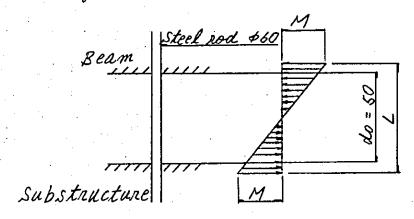
Analysed as above, horizontal fonce Calculation is made based on the case in Cross sectional direction

- 3. Stress calculation of the stopper made of steel rod
  - (1) Shearing Stress

    Steel rod  $\phi = 60^{mm} (SS41)$  As = 28.27 cm<sup>2</sup>

    H = 8.32 t  $T = \frac{H}{AS}$   $= \frac{8.32 \times 10^{3}}{28.27} = 290 \% (850 \times 1.5)$ = 1275 \frac{\kappa\_{2}}{2} \frac{\kappa\_{2}}{2} \frac{\kappa\_{3}}{2} \fr

## (2) Bending stress



$$L = do + \frac{1}{2} \cdot \phi = 50 + 60 \times \frac{1}{2} = 80^{mn}$$

$$H = 8.32^{t}$$

$$M = \frac{1}{2} \cdot H \cdot L$$

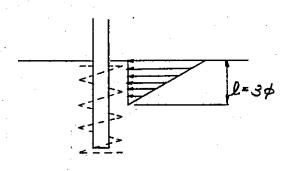
$$= \frac{1}{2} \times 8.32 \times 0.08 = 0.33^{tm}$$

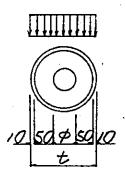
#### (3) Combined stress

$$\sqrt{\left(\frac{\delta s}{\delta sa}\right)^2 + \left(\frac{7}{7a}\right)^2} = \sqrt{\left(\frac{560}{2250}\right)^2 + \left(\frac{290}{275}\right)^2}$$

$$= 0.73 < 1.7$$

### 4. Calculation of bearing stress of concrete





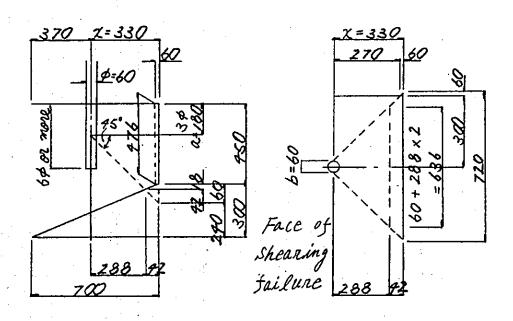
$$\delta_c = \frac{2 \cdot H}{\ell t}$$

$$t = 6.0 + (5.0 + 1.0) \times 2 = 18.0$$
 cm

$$\delta_{c} = \frac{2 \times 8.32 \times 10^{3}}{18.0 \times 18.0} = 51.4 \% < \delta_{ca} = 240 \times 0.8$$
$$= 192 \%$$

(3) Calculation of related of the Stopper installation

1. Shearing Strees Calculation



Shearing stress

$$T = \frac{H}{AZ}$$

$$AZ = \sqrt{2}\chi(2\chi + 2\alpha + b)$$

T: Shearing Stress

Az: Area of the face of Shearing failure

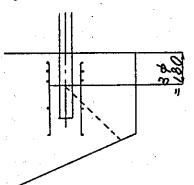
$$H = 7.13^{t}$$
 (In direction of nailway profile)
$$Z = \frac{7.13 \times 10^{3}}{4209.0} = 1.69^{100} < 3.9^{100}$$

2. Reinforcing ban arrangement around the part of stopper installation

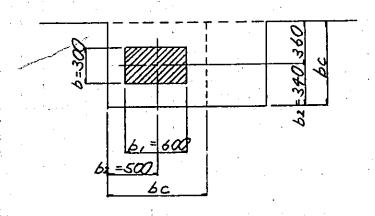
$$As' = \frac{H}{5sa}$$
=\frac{7.13 \times 10^3}{2.000 \times 1.5} = 2.38 \text{om}^3

As = D13 - 3 bans = 3.80 one > As = 2.38 cm²

Therefor, D13-3 bans are arranged within the range of 39.



(4) Calculation of shoe bed part subjected vertical load



1. Reinforcing bar arrangement for ventical load

$$As = \frac{1}{4} \left( 1 - \frac{b_1}{b_c} \right) \frac{R}{\delta sa}$$

Whene, As: Sectional onea of bans (cm)

bsa: Allowable tensile stress of ban ("yem")

5sa = 2 000 /m (SD35)

R: Ventical force acting on Shoe bed (Kg)

R = 59.45 + 77.52 = 136.97 Km2

bi : Acting width of bearing force (cm)

be: Distributing width of bearing power (om)

bc = 2.bz

$$b_1 = 30.0^{cm} \qquad b_2 = 34.0^{cm}$$

$$b_3 = 2 \cdot b_2 = 2 \times 34.0 = 68.0^{cm}$$

$$A_3 = \frac{1}{4} \times \left(1 - \frac{30.0}{68.0}\right) \times \frac{136.97 \times 10^3}{2.000} = 9.57^{cm^2}$$

(2) In direction of bridge cross section

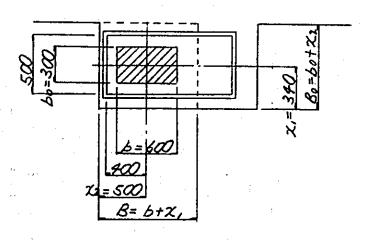
$$b_{1} = 6.00^{m} \qquad b_{2} = 50.0^{m}$$

$$b_{c} = 2 \cdot b_{2} = 2 \times 50.0 = 100.0^{m}$$

$$A_{s} = \frac{1}{4} \times \left(1 - \frac{60.0}{100}\right) \times \frac{136.97 \times 10^{3}}{2000} = 6.85^{cm^{2}}$$

2. Ban annangement

Range of ban annangement



B.B.: Range of ban arrangement

(1) In direction of bridge axis

$$B' = b + \chi_{1}$$

$$= 60.0 + 34.0 = 94.0^{m} > 2 \times 34.0 = 68^{m}$$

Therefore, the range of bar arrangement will be,  $B = 37.0 + \frac{97.0}{2} = 81.0 \text{ cm}$ 

Use D16 - 150 the (n = 6 bars,) then  $AS = 1.986 \times 6 = 11.92 \text{ cm}^2 > 9.57 \text{ cm}^2$ 

(2) In direction of bridge cross section

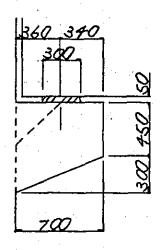
$$Bo' = bo + \chi_2$$
  
= 30.0 + 50.0 = 80.0 ° > 50.0 °

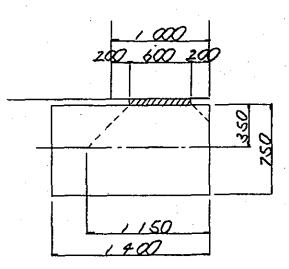
Therefore, the range of bar arrangement will be,  $Bo = 50^{cm}$ 

Use D16 - 150 ctc (4 bars) then  $AS = 1.986 \times 9 = 7.99 \text{ cm}^{2} > 6.85 \text{ cm}^{2}$ 

### (5) Calculation of beam supporting part

# 1. Bending stress calculation





Effective width B=0.60 + 0.20 + 0.35 = 1.15

(1) Load calculation

(a) Reaction of Tsection simple beam

Referred the summany table of shoe calculation and beam reaction,

Rd = 59.45 t

Re+2 = 77.52 t

(b) Own weight of beam support part

Wz = 2.5 " x 0.70 x 0.30 x 1/2 = 0.26"

Wd = 1.05 7m

- (2) Bending moment calculation

  Stress per  $B = 1.00^m$  of effective width
  - (a) Dead Load

    Md = 59.45 × 0.36 + 0.79 × 1.00 × 0.70 × 1/2

    + 0.26 × 1.00 × 0.70 × 1/3

    = 21.74 tm
  - (b) Train load + Impact

    Ml+i = 77.52 × 0.36 = 27.91<sup>tm</sup>
  - (c) Dead load + Train load + Impact EM = 21.74 + 27.91 = 49.65 tm

(3) Allowable stress, safe against cracking

$$\mathcal{L} = \frac{ML}{\Sigma M} = \frac{27.91}{49.65} = 0.56 > 0.25$$

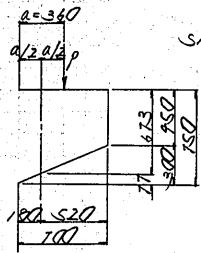
Hence, determined (sa = 1 000 "/om"

(4) Allowable stress for members that may be affected by fatigue of the material

Load Sharing ratio is assumed, as 1.0

M (tm)	21.74	49.65	•
N (i)			
S (1)			
b (cm)	100	IW	
h (cm)	75	75	:
d (cm)	69.4	89.4	
d' (cm)	5.6	5.6	
	D32-1250c		
As (cm²)	- 63.54	= 63.54	
p	0.00914		
As (cm²)	•	•	
p			
e=M/N (cm)			
e = M/N + u			
e = M/N - u			
e/h		es agrico	
.d/e			/
ď /h		The state of the s	
d' / d			
Nelbd* kg/cm*)			•
k			
c			
j			
1/Lc		6.12	
1/Ls	154	154	
β= σs/ σc			
σc (kg/cm²)	27.6	63.1	
OS (kg/cm²)	700	1590	
T (kg/cm²)			
osa (kg/cm²)	1 000	2 000	
σca (kg/cm²)	90	90	
Ta (kg/cm²)			
Nonogram	M-1	5	
combination	7. O.	D+ T+ I	
			JICA

# 2. Calculation of Shearing Stress Referred R.C. Standard 39 (1). (a). (b),



Spearing stress is calculated at the

a/2 point.

Effective width is assumed as the full width.

d= 67.3-5.6 = 61.7 cm

(1) Shearing force

P = 59.95 + 17.52 = 136.97 t

a = 0.36 m d = 0.617 m

 $\gamma = \frac{2}{a/d} = \frac{2 \times 0.617}{0.36} = 3.920 < 9$ 

Trerefors, 8 = 3.728

IS = 39.96 + 2.5 × 0.82 × 0.95 × 1.90

+ 2.5 × 0.52 × 0.223 × 1/2 × 1.90 = 90.98

(2) Spearing Stress

L= 5 d 190 × 10 = 9.14 1/cm² < La = 3.9 1/2 /cm²

(3) Bar arrangement for resisting against diagonal tension

Shearing force per unit one meter

$$S = \Sigma S / Effective with  $B = 1.90^{m}$   
=  $\frac{136.97}{1.90} = 97.89^{t}$$$

$$As' = \frac{s \cdot \sqrt{2}}{6sa} \qquad 6sa = 1000 \frac{18}{10m^2}$$

$$= \frac{97.89 \times 10 \times \sqrt{2}}{1000} = 76.87 \text{ cm}^2$$

Use diagonal tension Bars D29-125 ctc (8 Bars)

$$As2 = 63.59 \times \frac{1800 - 1810}{1800 \times \sqrt{2}} = 12.23$$

AS = 75.77 cm2 (AS = 76.87 cm2

3. Extra bars arranged at the Side face of beam support

Extra Bars are arranged at the Side face, with use of Bars of 90% cross Section of bending stress and installed in three steps.

As' = 63.59 × 0.90 = 25.92 cm2

Use extra bars of 019-250 ctc (9 bars)- in

three steps then,

As = 2.865 × 9 × 3 = 39.30 cm² > As' = 25.92

# 9. Bar arrangement chart

