REPUBLIC OF INDONESIA

MINISTRY OF COMMUNICATIONS

DIRECTORATE GENERAL OF BAND TRANSPORT

AND INLAND WATERWAYS

TENDER DOCUMENTS

NEW RAILWAY LINE FOR CENGKARENG AIRPORT CONSTRUCTION PROJECT

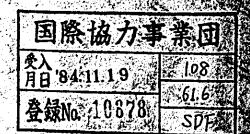
STRUCTURAL CALCULATION SHEETS

PACKAGE I CIVIL AND ARCHITECTURAL WORK

(6 of 11

AUGUST-1984





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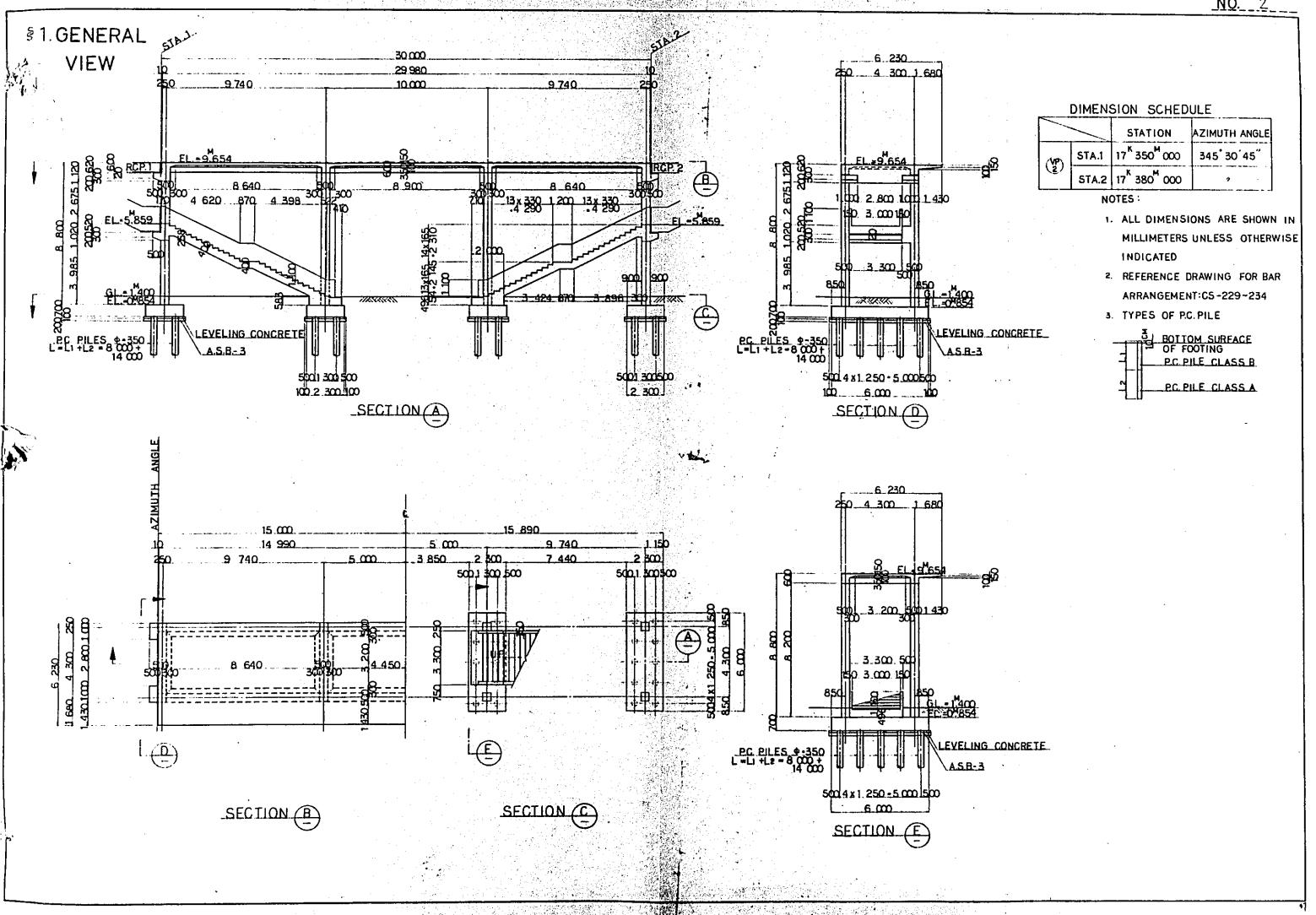
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§§ 9. VIADUCT OF PLATFORM VP2

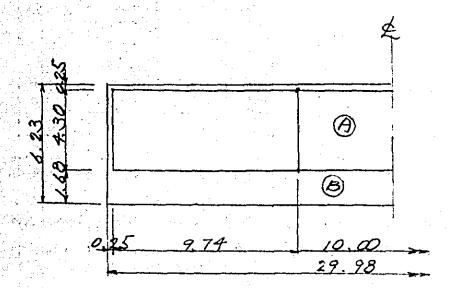
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\$2. Calculation of Slab



1. Slab for calculation

Slab @ --- Two - Way slab

Slab 8 ---- Cartilever slab

2. Calculation of slab 1

Four sides fixed span

lax = 4.30 - 0.50 = 3:80°

lay=100-050 = 9.50"

Four sides semi-tixed span

led = 3.80 + 0.15 = 3.95"

ley = 9.50 + 0.15 = 9.65

Span ratio

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

- 3. load calculations
 - a) Dead Load

b) pedestrian load w = 0.50 m²

4. Bending moment

1) Dead load

(Sharing of Load

ldx = 3.80" ldy = 9.50"

Coefficient of load sharing in the

direction X or Y

 $CX = \frac{ld\eta^{4}}{ldx^{4} + ld\eta^{4}} = \frac{9.50^{4}}{3.80^{4} + 9.50^{4}} = 0.975$

 $C\eta = \frac{ldx^4}{ldx^4 + ldy^4} = \frac{3.80^4}{3.30^4 + 9.50^4} = 0.025$

Sharing load

wdx = wd. Cx = 0.43 x 0.975 = 0.42 5/22

Wdy = Wd · Cy = " x 0.025 = 001"

(ii) Bending moment

 $mex_{1} = \frac{1}{24} wdx \cdot ldx^{2} = \frac{1}{24} \times 0.42 \times 3.80^{2} = 0.25^{**}$ $mex_{1} = \frac{1}{24} wdx \cdot ldx^{2} = \frac{1}{24} \times 0.01 \times 9.50^{2} = 0.04^{**}$ $mex_{2} = -\frac{1}{12} wdx \cdot ldx^{2} = -\frac{1}{12} \times 0.42 \times 3.80^{2} = -0.51^{**}$ $mex_{2} = -\frac{1}{24} \cdot w \cdot x^{2} = -\frac{1}{24} \times 0.43 \times 3.80^{2} = -0.26^{**}$

Torsional coefficient

$$Px = P_{\pi y} = \frac{5}{18} \times \frac{ldx^2 \times ldy^4}{ldx^4 + ldy^4}$$

$$= \frac{5}{18} \times \frac{3.80^2 \times 9.50^2}{3.80^4 + 9.50^4} = 0.0435$$

at the span center point

direction X

direction Y

at the support point

direction X

direction Y

2) Pedestian Load (people load)

We = 0.50 402

i) Sharing of Load

lex = 3.95 "

ley = 9.65 m

direction X

direction Y

Shared load in the direction of X or Y

direction X

direction

ii) Bending moment

Four sides simple

Four sides timed

Four sides semi - tixed

Torsional caefficient

$$= \frac{25}{36} \times \frac{3.95^2 \times 9.65^2}{3.95^4 + 9.65^4} = 0.1/32$$

At the span center point

direction X

direction

. At the support point

direction X

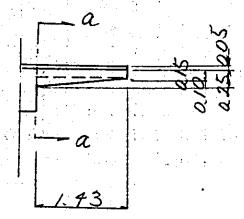
direction Y

5. Combined moment

(Dead Soud + Pedestian Soud)

	(1.75 p. 1.5		· · · · · · · · · · · · · · · · · · ·	(tm)
		Dead Load	Pedection load	total
At the support	ス	-0.51	-065	-1.16
paint	y	-0.26	-0.33	-0.59
At the span center	X	0.24	0.57	0.81
paint	y	0.04	007	011

- 6. Calculation of slab (8)
- (1) Calculation of cantilever slab
- (a) Dead load



 $M_{a-a} = 2.5 \times 1.43 \times 0.15 \times 0.715$ $+ 2.5 \times 1.43 \times 0.10 \times 1/2 \times a+17$ $+ 1.1 \times a05 \times 1.43 \times a715 = a51$

- (b) Pedestrian load

 Ma-q = 0.50² × 1.43 × 0.715 = 0.51²
- 2) Combined stress
 (Pead Load + Pedestrian Load)

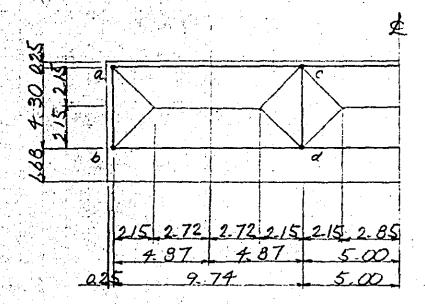
Ma-a = 051 + 051 = 1.02

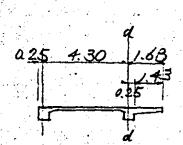
٠.	S		6			S - 100 -					٠.	•
						1	-//	Same :	$^{\circ}$			
v.	· •	,	1	1.1	100	100 mg		• //	<i>U</i> -	-	-	d .
_	~	~		يحو		CA	~~	~	<u>~ 0</u>		•	_

N 31 1 1 1 1 1 1 1 1 1 1 1 1 1 1 1 1 1 1	الريد فريد المراجعة	section	railway	Protile	(B) Salb		
	ac support	ax spari	at support	ax span	Front face	of column	
M (im)		1	14	1		- 1 20	
N (1)	-1.16	081	-0.59	a11	1.02	<u> </u>	
S (1)						<u> </u>	
b (cm)	100	100	100	100			
h (cm)	25	15	25	15			
d (cm)	22	12	23	, /	22		
d' (cm)	3	्रव	4	4	3	;	
	D10-15°	010-1500		D10-2000	D10-15 cm		
As (cm²)	= 4.76	= 4.76	}	= 3.57	·+.76	:	
<i>p</i>	0.00216	1	1	0.00325			
					7, 50, 50		
As' (cm³)			***	•		, · · · · · · · · · · · · · · · · · · ·	
p'						2 - 2	
e= M/N (cm)							
e = M/N + u						-	
e=M/N-u							
e/h							
d/e							
d' /h					,	·	
d'/d							
Helbd"(kg/cm²)	2-40	5,43	1.34	0.91	2-11		
k					· •		
<i>c</i>							
							
1/Lc	9,63	7.62	10.63	8 22	9.63		
1/Ls	500	279	630	338	500		
β= σs/ σc							
σc (kg/cm²)	23./	42.9	14.2	7.5	203		
OS (kg/cm²)	1200	1570	840	3/0	1050	·	
T (kg/cm²)						·.	
OSa (kg/cm²)	1800	.	•		<u> </u>		
OCA (kg/cm²)	90		4	' 4	*		
TO (kg/cm²)							
Number	M-1:	9	• • • • • • • • • • • • • • • • • • • •	,			
embination	Dead Load + Relitrian Load	,	,	5	5		

§3. Calculation of Torsional moment

1) Dead load





For the calculation, the dimension is assumed as 10.0m

a) Moment of supporting paint at the intermediate span for ement $1.1 \text{ th}^3 \times 0.05 = 0.055 \text{ th}^2$

Slah 25' x 0.15 = 0.375'

20do = 0.43 the

lx = 1000" ln = 4.30"

Distribution of load

$$C_{3} = \frac{l_{3}^{+}}{l_{3}^{+} + l_{3}^{+}} = \frac{1000^{+}}{4.30^{+} + 1000^{+}} = 0.967$$

Wy = 0.43 × 0.967 = 0.416 %

Moment of supporting Jains

M = 1/2 2/2 = 1/2 × 0.416 × 4.302 = 0.64 tm

b) Moment of cantilever slab

Pavement 1.1 this x 0.05 = 0.055 this

Slab 25 × 0.15 = 0375

2.5' × 0.10 × 1/2 = 0.125"

 $M_d = 0.055 \times 1.68^2 \times 1/2 + 0.375 \times 1.43 \times 0.965 + 0.125 \times 1.43 \times 0.727 = 0.73^{20}$

c) Tortional moment caused by Dead load

i) U- O Rohmen (Rigid Frame)

MTa = 0.64 x (2.15 x1/2 + 2.85) = 2.51 TO

MT0 = - 2.51 + 0.73 x 5.00 = 1.14"

Mr=2.51

Mr=1.14

ii) 2-2 Rahmen (Rigid grame)

$$M_{TC} = 2.51 \times 2 = 5.02^{tm}$$
 $M_{Td} = 1.14 \times 2 = 2.28^{\circ}$

$$M_T = 5.02^{cm}$$
 $M_T = 5.02^{cm}$
 $M_T = 2.28^{cm}$

- 2) Pedestrian lood (People lood)
 - a) Moment of supporting point at the intermoliate apar W = 0.35 the

load carrying factor $C_7 = 0.967$ Distribution of load $W_7 = 0.35 \times 0.967 = 0.34$ Moment of supporting paint $M = \frac{1}{12} \times 0.34 \times 4.30^2 = 0.52$

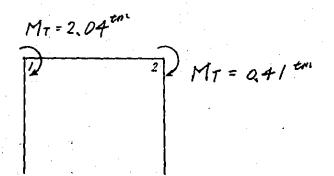
b) Cantilever slab $w = a.35^{4/2}$ $Md = 0.35^{4/2} \times 1.68^2 \times 1/2 = 0.49^{4/4}$ d) Torkional moment caused by

Pedestrian lood (people load)

i) O-O Rohmen (Rigid Frame)

MTa = 0.52 × (2.15 × 1/2 + 2.85) = 2.04tm)

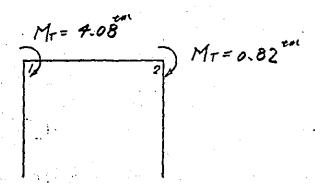
MT6 = - 2.04 + 0.49 x 5.00 = 0.41 2



ii) 2-2 Rahmen (Rigid Frame)

MTC = 2.04 × 2 = 4.08 to

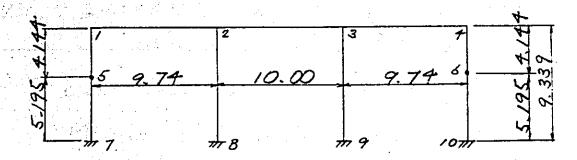
MTd = 241 × 2 = 282



§4. Rigid gram analysis on longitudinal direction of elevated structure

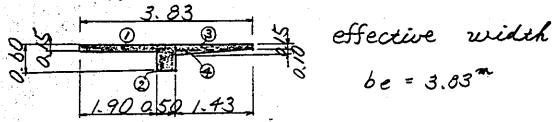
[1] Basic calculation

1. axis of the Rahmen (Rigid trame)



2. Cross Sectional aneas / moment of Imertia at area of members

(1) Upper beam (Member 1-2, 2-3, 3-4)



N	b (m)	h (m)	A (m2)	y (m)	A. 71 (m3)
0	1.900	0.150	0-285	0.075	0.02/33
2	0.500	0500	0.300	a300	0.09000
3	1430	0.150	0_2/5	0.075	0-01613
(4)	1.430	1/2×a100	0.072	0183	0.01318
Z			0.872		0-14069

	6 cm)	h (m)	A (m²)	70 (m)	Io (m+)	A. 2/0 (m+)	I. + A. 7 ()
\odot	1.900	a 150	0.285	0.086	a 00053	0.00211	0.00264
2	0.500	0.600	0.300	0-139	0 00 900	0.00580	0.01480
<u> </u>	1.430	0-150	0.215	2086	0.00040	20159	0.00199
(4)	1.430	1/2×0100	0-072	0.022	0-00004	0.0003	0.00007
Z			0.872		0.00997	0.00953	0.01950

cross Sectional Area A = 0.872 m²

Moment of Inertia I = 0.01950 m²

of area

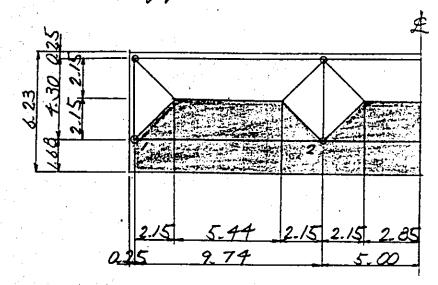
(2) Column (Member 1-5-7, 2-8, 3-9, 4-6-10)

$$A = 0.50 \times 0.50 = 0.250^{-2}$$

$$I = \frac{1}{12} \times 0.50 \times 0.50^{3} = 0.00521^{-1}$$

(3) axial height $h_1 = 9.50 - 0.161 = 9.339^{m}$ $h_2 = 4.305 - 0.161 = 4.144^{m}$

- (2) Load calculations
 - 1. Dead load
 - (1) Upper member



(a) Uniformly distributed land

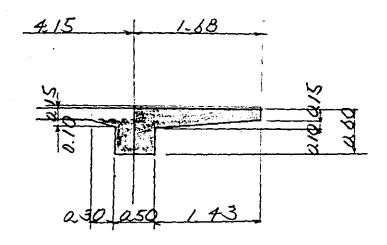
Slab 2.5 to x 0.15 = a 375 tm²

pavement 1.1 × 0.05 = 0.055

w = a 43 /m2

Wd1 = 0.43 this = 0.92 th

(b) linear load



pavement $1.10^{\frac{4}{12}} a.05 \times 1.68 = 0.092^{\frac{4}{12}}$ Cantilevel Slab $2.5^{\circ} \times (0.15 + 0.25) \times /2 \times 1.43 = 0.715^{\circ}$ Longitudinal beam $2.5^{\circ} \times (0.50 \times 60 - 0.25 \times 0.15) = 0.656^{\circ}$ Hauch of slab $2.5^{\circ} \times 0.30 \times 0.10 \times /2 = 0.038^{\circ}$

Reaction of form beam

- (1) Dead load
 - · pavmet

· Slab 2.5 *** 0.15 x (4.80 x 9.93

$$-3.30 \times 5.35 - 3.00 \times 2.63) = 8.38^{t}$$

· contilevet slab

· Slab haunch

· Longitudinal beam

· Transverse beam (A)

· Transverse beam (B)

· handrail

calal = 35.31°

Reaction per on sho

Rd = 35.21 × 1/4 = 8.80°

(2) Pedestrian load (People load)

0.35 42 × (6.23 × 9.98 - 3.30 × 8.13) = 12.37

Reaction per on sho $Rd = 12.37 \times \frac{1}{4} = 3.09^{t}$

Calculation for Seismic Lood $0.21^{\frac{4}{10}} \times (6.23 \times 9.93 - 3.30 \times 9.13) = 7.42^{t}$

Reaction of stepped (A) (Platform ~ column center).

(1) Dead load

= 4.52

Reaction

2) Pedestrian load (People load) $0.35^{\frac{4}{10.2}} \times 3.00 \times 9.98 = 10.48^{c}$ Reaction $W = 10.48 \times 1/2 \times 1/3.30 = 1.59^{\frac{4}{100}}$

Calculation for Seismic Lood $0.21^{\frac{4}{100}2} \times 3.00 \times 9.98 = 6.29^{\pm}$

Reaction of stedpped (B) (column center - Foundation)

1) Dead Load

· permet 1.1 4 x 0.05 x 3.00 x 10.99 = 1.81t

· Slab 2.5 × 0.40 × 3.30 × 0.87 = 2.87

25 × 0.447 × 3.30 × (3.424+ 4.196)

= 28.10

25 x0.30 x0.15 x 1/2 x 3.30 = 0.19"

25 x 0.094 x 0.047 x 1/2 x 3.30 = 0.02

25 × (0.55+1.03) × /2 × 0.96

x 3.30 = 6.26

2.5' x 0.496 x 1.04 x 3.30 = 4.26

· Step 2.5 × 0.33 × 0.165 × /2 × 3.00 × 26 = 5.3/

handrail 25 x 0.15 x 1.10 x 10.58 x 2 = 8.73

total = 57.55 e

Reaction

W = 57.55 × 1/2 × 1/3.30 = 8.72 thi

2) Pedestrian load (People load)
0.35 4 × 3.00 × 10.99 = 11.54 t

Reaction

W = 11.54 × 1/2 × 1/3.30 = 1.754m

Calculation for Scismic Load $0.21^{4/m^2} \times 3.00 \times 10.99 = 6.92^{4}$

(c) Concentrated loads acting panel point

ponel faint 1. (4)

Unitornly distributed load

0.43 tme × 2.15 × 2.15 × 1/2

= 0.99 t

0.43 m × 0.25 × 2.15

= 0.23 ^t

linear load

(0.092 + 0.715) this 0.25

= 020

Haunch of slab

2.5 this 0.30 x 0.10 x 1/2 x 1.60

= 006

Cross bean

2.5 1 x 0.50 x 0.97 x 2.40

= 2-9/

Beam support

2.5 × (0.30+0.50) × 1/2 × 0.50 × 2.40 = 1.20

Reduction of linear load

- (0.656 + 0.038) tan x 0.25

=-a/7

Reduction of column part

- 2.5 x 0.50 x 0.50 x (1.12 - 0.161) = - 0.60

Shed load reaction =
$$10.00^{\circ}$$

Reaction of platform beam and Stainway
 $8.80^{\circ} + 9.67^{\circ} \times 3.30 \times 1/2 = 24.76^{\circ}$
 $P_1 = P_4 = 39.58^{\circ}$

Panel paint 2 (3)

· Uniformly distributed load

0.43 *** 2.15 × 2.15 × ½ × 2 = 11.13 **

Cross beam

 $2.5^{\frac{1}{10.50}} \times 0.50 \times 0.45 \times 1.90 = 1.07^{\circ}$

· Haunch of slab

2.5 × 0.30 × 0.10 × 1/2 × 1.60 × 2 = 0.12

Reduction of linear load

 $-(0.656 + 0.038)^{4} \times 0.50 = -0.35$

Compensation for column

25 × 0.50 × 0.50 × (0.161 - 0.15) = 0.01

Shed load

= 10.00

P2 = P3 = 12.83 t

Panel point moment caused by platform beam supporting part

 $M_1 = M_4 = 24.76 \times 0.52 + 2.5 \times (0.50 \times 0.30 \times 0.50 + 0.50 \times 0.20 \times 1/2 \times 0.417) \times 2.40 = 13.45 \times 10^{-10}$

(2) Concentrated load at the intermediate point of calumn

panel paint 5 (b)

Cross beam

2.5 this a 50 x 1.02 x 1.90

 $= 2.72^{+}$

Beam support

2.5 × (0.30+0.50)×/2×0.50 × 2.40 = 1.20

Reaction of Stairway

(967+8.72) x 1.65

= 30.34

Ps=Pb= 33.96 t

Panel point moment caused by stainway reaction and beam supporting part $M_5 = M_6 = 9.67^{4m}_{\times 1.65 \times 0.52} + 2.5^{43}_{\times}(0.50 \times 0.30 \times 0.50) + 0.50 \times 0.20 \times 1/2 \times 0.417) \times 2.40 = 8.87^{4m}$

Oun weight of column $g = 2.5^{\frac{4}{10}} \times 0.50 \times 0.50 = 0.63^{\frac{4}{10}}$

(3) Stairway reaction acting at foundation $P_8 = P_9 = 8.72^{4m} \times 3.30 \times 1/2 = 14.39^{t}$

- 2. Pedestrian load (People load) $W = 0.350^{4/m^2}$
- (1) Upper member (Refer (2) 1.(1))
 - a) Unitormly distributed load

 Wd1 = 0.35 × 2.15 = 0.75 tm
 - b) Wd2 = 0.35 % 1.68 = 0.59 4m
 - C) Concentrated load acting at the panel point
 - (1) Panel point 1 (4)
 - · Uniformly distributed load $0.35^{\frac{4}{1012}} \times 2.15 \times 2.15 \times \frac{1}{2} = a.81^{\pm}$
 - · Uniformly distributed load

 0.35 th × 0.25 × 3.83 = 0.34
 - Reaction of platform and stairway $1.59^{4m} \times 3.30 \times 1/2 + 3.09^{4} = 5.71^{4}$

P, = P4 = 6.86 t

panel paint 2.(3)

Uniforly disteributed load

0.35 th² × 2.15 × 2.15 × $\frac{1}{2}$ × 2 = 1.62 th

Moment caused by the reaction of platform beam $M_1 = M_4 = 5.71 \times 0.52 = 2.97^{tm}$

(2) Concentrated load acting at the intermediate point of column.

Panel point 5.(6)

Stairway reaction

Moment caused by stairway reaction $M_5 = M_6 = 1.59\% \times 3.30 \times 1/2 \times 0.52 = 1.36\%$

(1.59+1.75) × 3.30 × 1/2 = 5.51 t

(3) Stairway reaction acting at foundation $P8 = P9 = 1.75^{4/2} \times 3.30 \times 1/2 = 2.89^{t}$

3. Temperature change and Drying contraction

Temperature change ---- ± 10°C

Drying contraction ---- - 15°C

Hence,

Temperature rise ---- +10°C

Temperature drop + Drying contraction

----- - 25°C

4. Seismic load from dead load (&H=0.10)

(derived from (2)(1) and (2)(1) horizontal force)

Panep point 1-4

Slab

2.5^{4,3} × 0.15×4.80×29.98 = 53.98

Consider a Slab $2.5 \times (0.15 + 0.25) \times /_2 \times 1.43 \times 29.98 = 21.44$ Haunch of slab $2.5 \times 0.30 \times 0.10 \times /_2 \times (27.98 \times 2 + 3.20 \times 6) = 2.82$ longitudinal beam $2.5 \times 0.50 \times 0.45 \times 29.98 \times 2 = 33.73$ Cross beam (at end part) $2.5 \times 0.50 \times 0.97 \times 3.80 \times 2 = 9.22$

Cross beam (at intermediate part)

25 th x 0.50 x 0.45 x 3.80 x 2 = 4.28

Beam support 2.5 × (0.30+0.50) ×/2 × 0.50 × 4.80 × 2 = 4.80

calumn 2.5 × 0.50 × 0.50 × (-9.339 + 0.161 -0.60) x8 = 21.16 Intermediate cross beam 2.5'x 0.50 × 1.02 × 3.30 × 1/2 × 2 = 4.20" Intermediate cross beam support 25 x (030+0.50) x /2 x a50x 4.80x2 = 4.80 $(13.0 + 10.0) \times 4 = 92.00$ Shed load Platform beam 8.80 x 4 = 35.20° Secience 9.67 x 3.30 x 2 = 63.82' $(9.67 + 8.72)^{7} \times 3.30 \times 1/2 \times 2 = 60.69$ Pedestrian load (People load) 0.21x623x29.98= 39.22" " (on platform) 7.42" x 1/2 x 2

Therefore, the seismic horizontal load is calculated as below $H = 468.20 \times 1/2 \times 0.10 = 23.41^{t}$ $H_{1\sim4} = 23.41 \times 1/4 = 5.85^{t}$

" (on stainway) $6.29 \times 1/2 \times 2 + 6.29 \times 1/4 \times 2 = 9.44$ "

total = .468.20 t

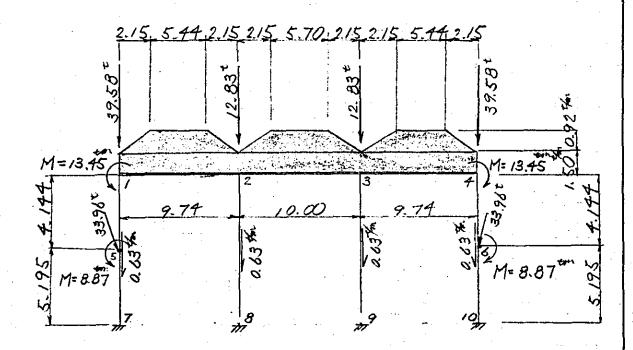
Horizontal force transmitted from

Stainway to foundation

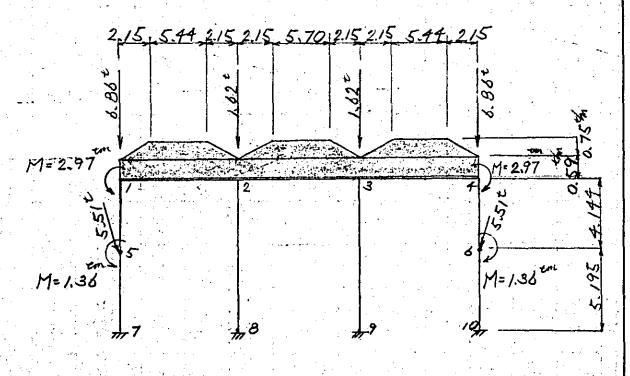
Calculation for end part $(9.67 + 8.72)^{t_m} \times 1.65 \times 1/2 = 15.17^{t_m}$ $(8.29 + 6.92)^{t_m} \times 1/2 \times 1/4 = 1.65^{t_m}$ $(4.20 + 4.30)^{t_m} \times 1/2 \times 1/2 = 2.25^{t_m}$ $(4.20 + 4.30)^{t_m} \times 1/2 \times 1/2 = 1.07^{t_m}$ $(4.20 + 4.30)^{t_m} \times 1/2 \times 1/2 = 1.07^{t_m}$

Calculation for intermediate part $8.72^{4} \times 1.65 + 6.92^{7} \times 1/2 \times 1/2 = 12.66^{4}$ $H = 12.66 \times 0.10 = 1.27^{4}$

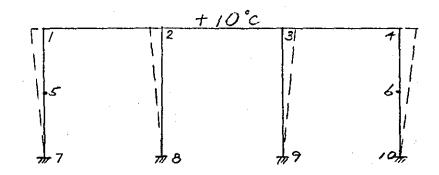
(3) load diagram 1. cost 1 Dead land



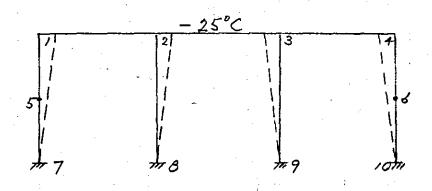
2. case 2 Pedestrian load (people load)



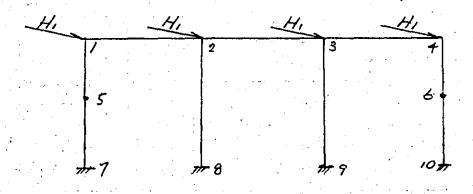
3. case 3 Temperature rise



7. case 4 Temperature drop + Drying contraction



s. case 5 Scismic load from dead load



(9) Combination of Isads 1. Basic Load

Case Nomber	Kind of load	Loading Pattern
/	Dead load	
2	Pedestrian load	
3	Temperature rise	+ 10°C
4	Temperature drop + Drying contraction	- 25°C
5	Seismic load from dead load	1111

2. Combined loads

case	Combination of loads	2
6	1 + 2	1.0000
7	1 + 3	0.8696
8		,
9	1 + 5	0.6667
10	1 - 5	. 4

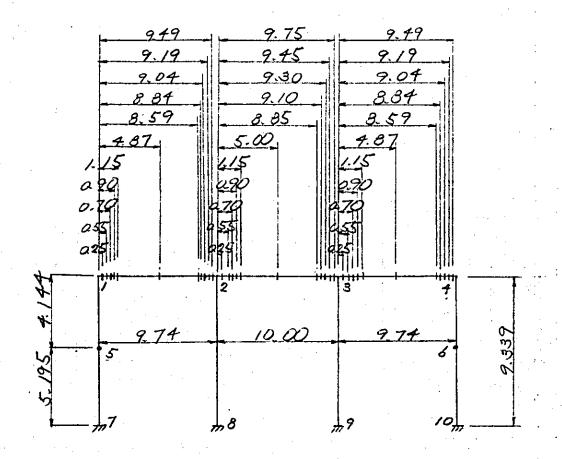
3. Critical cases

No.1 case 1 crack

No.2 case 6 ~ 10 Synthetic

No.3 case 9.10 Footing

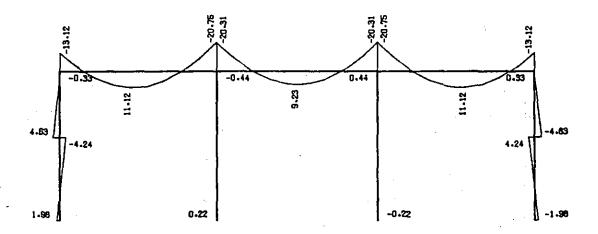
9. Points for stress calculation



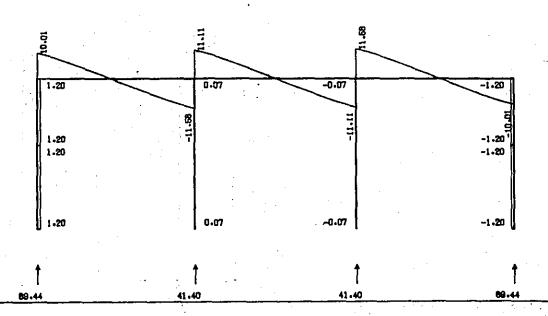
(S) Stress diagram
VIADUCT OF PLATFORM (NORTH SIDE) L-1

CASE 1 (DEAD LOAD)

BENDING MOMENT



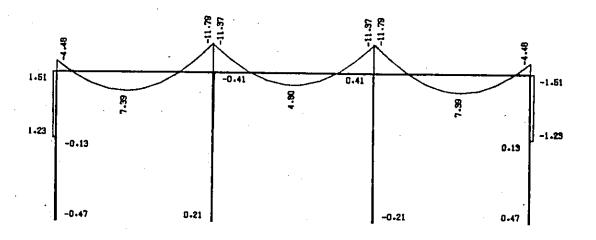
SHEARING FORCE



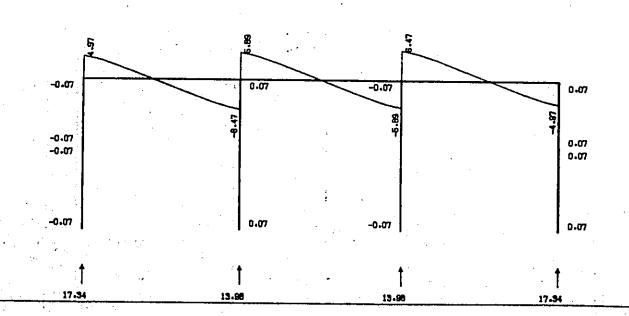
VIADUCT OF PLATFORM (NORTH SIDE) L-1

CASE 2 (PEDESTRIANS LOAD)

BENDING MOMENT



SHEARING FORCE

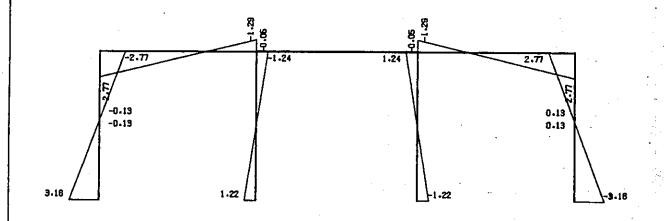


VIADUCT OF PLATFORM (NORTH SIDE) L-1 CASE 3 (TEMPERATURE) BENDING MOMENT 1-11 -0.05 -0.05 0.06 SHEARING FORCE -0.25 -0.25 -0.25

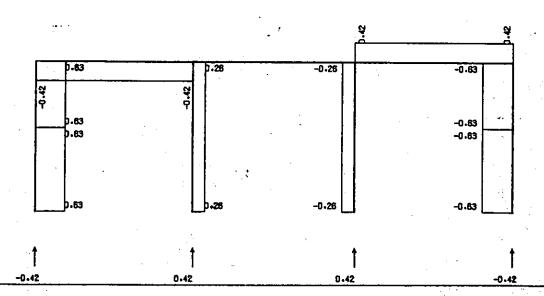
VIADUCT OF PLATFORM (NORTH SIDE) L-1

CASE 4 (TEMPERATURE + SCHRINKAGE)

BENDING MOMENT



SHEARING FORCE



VIADUCT OF PLATFORM (NORTH SIDE) L-1 CASE 5 (SEISMIC LOAD) BENDING MOMENT SHEARING FORCE

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2 15	. 550	6 6	9.457	3,215	.591	~ m	. 550		-19,928	8,950	100.1
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9	4, 870	6)	11.167	3,390	591			7 07	5.662	•. •	1.004
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	9.640	(10)	.957	14.083	1.004	ю (D [*]		 	-16.405		.591 .591
10	9.190	95	.332	-4.260	1.004	10	9.190	6 .	-19,830		. 591
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				1	IER 3. (0.000	550	900	4.870	9.0	9.190	9.740	4,329	ER 4 (0.000	ER 5 (5.195	ER 6 (9.339	ER . 7 C.	9.339	ER 8 (0.000	ER 9 (6.050 5.195
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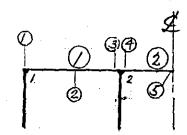
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PC-FANCY			AR MINIMUM		1 5 5		8.44.2	9.926	10.509	9.674	-16.485	-19.830	-22.879	-25.494	== 9 (36	-3.235	3	222	15	20.5	-16.018		; = 9 C	-2,175	.332	1.750	5.669				-22.664	
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CHORTH SIDE)			HAXIHUM	H	= = 9	5.018	9.928	599	777.			.332	'	. 0/1.3	اا اا	0 -	6.018	793	1.286	227	1.426	-3.235	6.361	= = 9	-25.494 1 -22.879 1	9.83U	504.5	.052	1.167	0.509			
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накірон	AXIAL HINIMUM
	N
***************************************	= = NEMBER 4 (1 - 5) G = =
-16.489 4.492 -30.196 2.125 4.492 -31.937	TIAN 0.000 (10) 16.051 -2.896 -35.931
	* = MEMBER 5 (5 - 7) C = 2
19.545 4.492 -54.578 19.545 4.492 -56.760	JIAN 0.000 (10) -1.865 -2.896 -60.313 JIAN 5.195 (10) -16.910 -2.896 -62.495 = #FMBFR 6 (2 8) C = =
18.544 -4.059 -22.251 -19.365 -4.059 -26.173	17AN 0.000 (9) -19.
4.059 -22.251	1678 7 (3 - 9) C = = = 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0
i	(10) -19.658 -4.154 4 - 6) C = =
-4.492 -30.196 -4.492 -31.937	1;4% 0.000 (9) JIAN 4.144 (9)
-4-492 -54-578	= = MEMBER 9 (6 - 10) C = = IIAN 0.000 (9) 1.865
	en de marche en entre de de la compansión de la compa
	одинация в пред пред пред пред на пред на пред на пред на пред на пред на пред на пред на пред на пред на пред

(6) Calculation of upper beam



(1) Calculation of Compressive stress caused by bending
(a) Summary of stresses
(i) At the support point

			,	· · · · · · · · · · · · · · · · · · ·	· 		Pi	OK UP 1	VO. Z
			(\mathcal{D}				2)	
		0	co.	3	Co.	4	co.	0	co.
Combined	TOP	-25.02		≘32. <i>54</i>		-31.69	6		<u> </u>
stress	Bottom	8.44	} {) {	 			i }
Dead.	load	±/8 /2		-20,75		-20,31	/	· · · · · · · · · · · · · · · · · · ·	

(ii) Span moment

	:	\bigcirc		2	
		2	Co.	3	co.
combined stress	Bottom	1/8.23	6	14.03	6
Company Marine Fo	7 ° ° ° ° ° ° ° ° ° ° ° ° ° ° ° ° ° ° °			·	

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

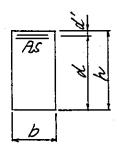
$$d = \frac{4.48}{17.80} = 0.254 > 0.25$$

There fore (sa = 100 //m

(ii) At the support point 2.3

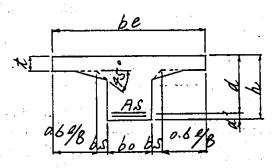
There fore (sa = 1000 Mg/m

- (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



d'/h	 	·				
	-					-
	-					·
ď/h			.1	,		
ď/h						
	 					
d/e	 					
ď/h		·				
ď/h						
d' / h			.3	•		
d'/h		· 			·	
					<u></u>	·
ď/d				- 17/		
				-0-176		
Te/bd*\kg/cm	9 7/	16.40	15.53	24.35	326	
	" 8.71	16.60	15.53		3.26	
k		*		-0-514		
	-					
e Lc				0-288		
<u></u>						
j				0.827		
1/Lc		00			- 09	
	4	99			7-89	· · · · · · · · · · · · · · · · · · ·
I/Ls	9	32	_50_		305	
) 4	5.0		305	
β= σs/ σc						<u> </u>
σc (kg/cm	1)	20.0	740	.0.4.7		
		82.8	780	845	25-7	
OS (kg/cm	•)	1360		1200	1000	
	- 	1300		1200		
T (kg/cm	9					
osa (kg/cm	")	, 900	~~~	, 000	,	
		1 800	,000	1 800	<u> </u>	
oca (kg/cm	9	90		90	,	
		70		70	 -	
TA (kg/cm		<u> </u>			<u></u>	
Nonegran						
number	M-1	4	3	M-7.9	M-47, M-1	
combination	1 0	D+P	S.	D+P		

- (e) Required minimum cross section of ne-bans
 - (i) At the top of support point 1

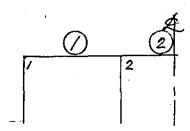
$$As = \frac{M}{\text{Rsa} \cdot j \cdot d} \times \frac{4}{3}$$

$$= \frac{25.02 \times 10^{5}}{1800 \times 0.875 \times 54.9} \times \frac{4}{3} = 38.58^{cm^{2}}$$

(ii) At the cop of support paint 2

(iii) At the span center point

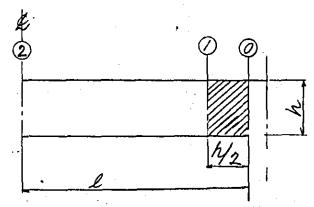
(2) Calculation of Shearing Stress



(a) Summary of shearing stresses

For examining Section 0, Shearing stress

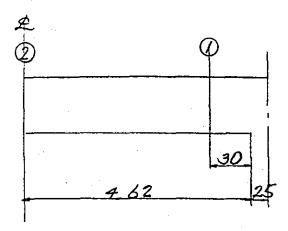
at Section 1 is used



			D				2)	
	Left supposit	Co,	Right support	co.	Left.support	co.	Right Support	00,
0	14.44		-17.50		16.46] 		
Ø	13.72	6	-16.78	6	15-74	6	1	
2	-1.53	 			0		1	
						[

(Note) CO. - Combination

- (b) Shearing stress
 - (i) Shearing stress caused by bending



(ii) Shearing siress of the member of unitom height

$$T_1 = \frac{16.78 \times 10^3}{50 \times 51.7} = 649 \text{ for} > 3.9 \text{ for}^2$$

$$7_2 = \frac{1.53 \times 10^3}{50 \times 51.7} = a59^{\circ}$$

Therefore, Stinzup Calculation is made.

- (C) Shearing stress caused by torsional moment
 - (i) Torsional moment

(Refer the result of computor analysis on torsional moment)

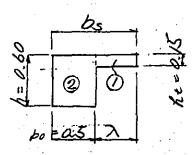
Dead load Md = 2.51 cm

Pedestrians load Mp = 2.04"

Hence, Total (D+P)

EMarp = 2.51 + 2.04 = 4.55 cm

(ii). Torsional moment beared by Longitudinal beam



Effective width $bs = bo + \lambda t$ $\lambda t = 3 \cdot ht$ $= 3 \times 0.15 = 0.45^{m}$ $bs = 0.50 + 0.45 = 0.95^{m}$ $h = 0.60^{m}$

Calculation of distribution ratio

	a	Ь	a/b	k	$It = R \cdot a \cdot b^3 (m^4)$
0	0.45	0.15	3.00	0.263	0.263 ×0.45 ×0.153 = 0.00040
2					0.166 × 0.60×0.503 = 0.01245
		Total	•		It = a01285

Frent tace of column

At the 1/2 paint

$$Mt2 = 1 \times 2 \times \frac{4.45}{5.00} = 3.92$$

(d) Shearing stress caused by corsion

Shearing stress caused the corsion

is calculated tallowed the equation

b: Shorter side length

a: longer side length

R: Table - 40.2

(i) Front face of calumn

Me = 4.19 cm

$$a = 60^{\circ 0}$$
 $b = 50^{\circ 0}$

$$\frac{a}{b} = \frac{60}{50} = 1.200 \quad \forall = 0.759$$

ii) at the
$$h/2$$
 point $M_t = 3.92$ cm

$$a = 60^{\text{cm}}$$
 $b = 50^{\text{cm}}$

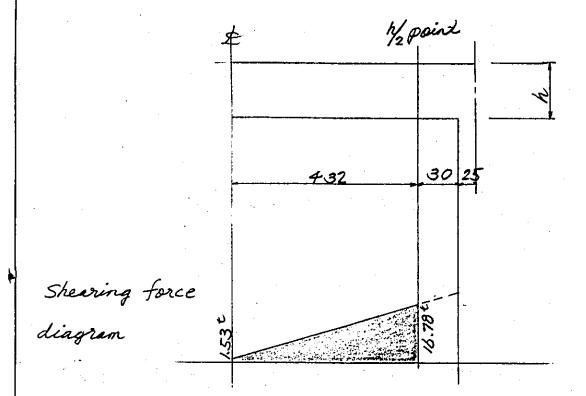
$$\frac{a}{b} = \frac{60}{50} = 1.200 \quad 4 = 0.759$$

(e) Combined shearing stress

Combined allowable shearing stress $7a = 17 \times 1.3 = 22.7$ Februar

Combined shearing stress $\gamma_1 = 6.49 + 11.53 = 18.02^{\frac{100}{100}} < 22.1^{\frac{100}{100}}$ $\gamma_2 = 0.59 + 0 = 0.59^{\circ} < 0.59^{\circ}$

(3) Calculation of diagonal tension re-bars
(a) Stearing stress caused by bending



(i) Shearing stress beared by concrete

where Tc: 3,9 Hom?

b: Witch of member (cm)

d: Effective height of member

at the examining section.

Sc1 = 1/2 x 3.9 x 500x 51.7 x 10-3 = 5.04 t

(b) Shearing force beared dy stirrup

Arrange Stirrups D13-1 Sets in 15.0 cec

Torsional Shearing stress $\hat{S}\hat{z} = \frac{Me \cdot S}{0.8 \cdot Av \cdot b_1 \cdot f_1} \times \frac{a_1}{b_1}$

Whore

Mc: Torsional moment (cm)

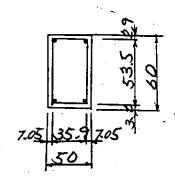
S: the distance of stirrup (cm)

Av : Gross cross Section of

coupled stirrups (cm²)

bis hi: length of short long side of stirrup

· (i) Ae 1/2 point



$$\sqrt{se} = \frac{3.92 \times 10^5 \times 15}{0.8 \times 2.53 \times 35.9 \times 53.5} = 1510^{\frac{R9/cm^2}{2}} (1800^{\frac{R9/cm^2}{2}})$$

Bending Shear beared by stirrup

In the case When Combined with consional moment, allowable shearing stress is

as 20 percent increased.

Usa = 1800 × 1.2 = 2180 Mane

Sw = (Usa-Ust) · Av · d

1.15 · D

(1) At h/2 paint $(2160 - 1510) = 650^{\frac{1}{2}} (1800^{\frac{1}{2}})$ $d = 51-7^{\frac{1}{2}}$

 $Su = \frac{650 \times 2.53 \times 51.7}{1.15 \times 15 \times 10^3} = 4.93^{t}$

(C) Shear beared by eurned bar

(i) Total shear

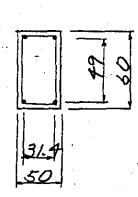
$$\Sigma S_{R} = 5.04 + 4.93 + 18.17$$

$$= 28.14^{4} > S = 16.78^{4}$$

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

Required re-bar arrangemet is calculated followed she equation.

(i) Front face of column



$$Mt = 4.19^{em}$$

$$\int Sa = 1.800^{\frac{k9}{em^2}}$$

$$b_1 = 3/.4^{em}$$

$$h_1 = 49.0^{em}$$

Required cross section of re-bars arranged at sharter side $Asb1 = 15.20 \times \frac{31.4}{2(31.4 + 49.0)} = 2.97^{cm^2}$

(ii) Required cross section of re-bass arranged at longer side

 $Ask_1 = 15.20 \times \frac{49.0}{2 \times (31.4 + 49.0)} = 4.63$ cm²

(iii) Top and Bostom

Minimum section of re-bars

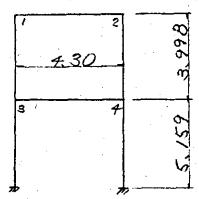
As = 025 - / = 5.07 cm2 > 4.03 cm2

Side (one side)

§ 5. Rigid fram analysis on transversal section ()-() of elevated structure

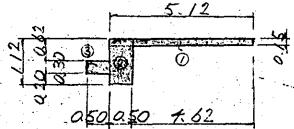
(1) Basic calculation

1. axis of Rahmen (Rigid Frame)



2. Cross Section areas/moment of inertia of area of members

(1) Upper beam (Member 1-2)

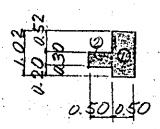


	B (m)	.h (m)	A (m2)	3 (m)	A7 (m3)
\bigcirc	4.62	0.15	0.693	0.075	0.05198
2	0.50	1.12	0.560	0.560	0.31360
(3)	0.50	0.30	0.150	0.770	0.11550
Σ			1.403		0.48/08

	b (m)	h (m)	A (m2)	y (n)	Io (10 (4)	A 2102 (an4)	Io+A702
\bigcirc	4.62	0.15	0 693	0.268	0.00130	0.04977	0.03107
2	0.50	1.12	0.560	0.217	0.05854	0.02637	0.08491
3	0.50	0.30	0.150	0.427	0.00113	0 02735	0.02348
Σ			1.403		0.06097	0.10349	0-16446

cross sectional Area $A = 1.403^{m^2}$ Mement of inertia $I = 0.16446^{m4}$ of area

(2) Intermediate beam (Member 3-4)



	P(41)	h (on)	A (nt)	21 (m)	Au (m2)
\bigcirc	0.50	1.02	0.510	0.510	0 26010
2	0.50	a30	0.150	0670	0.10050
5			0.660		0.36060

	b (m)	h (m)	A (m²	70(-)	Io (m+)	A No 2 (m4)	Io + A402
0	0.50	102	a 5/0	0.036	a 04422	0.00066	0.04488
2	0.50	a30	0.150	0.124	200113	0.00231	a 00344
Σ			0.660		a 04535	aw297	0.04832

Cross Sectional Area $A = 0.660^{m^2}$ Mament of inertia $I = 0.04832^{m^4}$ of area

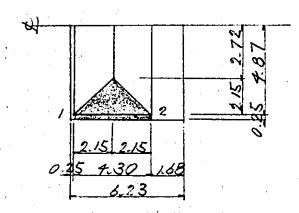
$$A = 0.50 \times 0.50 = 0.250^{m^2}$$

$$I = \frac{1}{12} \times 0.50 \times 0.50^3 = 0.00521^{m^4}$$

(4) axial height
$$h1 = 3.795 - 0.343 + 0.546 = 3.998^{m}$$

$$h2 = 5.705 - 0.546 = 5.159^{m}$$

- (2) load calculation
 - 1. Dead load
 - (1) Upper member



a) Uniformly discributed load

Slab 25 m3 x 0.15 = 0.375 me

poumet 1.1 × 0.05 = 0.055

W = 0.43 m2

Wd1 = 0.43 the x 0.25 = 0.11 m

Wd2 = 0.43 x (0.25+2.15) = 1.03

6) linear load

Cross beam 25th 0.50 x 0.97 = 1.21th

Harrich of slab 2.5 x 0.30 x 0.10 x /2 = 0.04"

Beam support 2.5 x (0.30+0.50) x /2 x 0.50 = 0.50

Wd3 = 1.75 **
JICA

d) Concentrated load acting at Panel point Panel point 1 Unitornly distributed load 0.43 x 0.25 x 5.12 Unitornly distributed load 0.43 x (272+487) x /2 x 2.15 longitatinal beam 2.5 × 0.50 × 0.45 × 4.62 Haunch of slab 25 × 0.30 × 0.10 × 1/2 × 4.32 Beam support 25 × (030+0.50)×/2 × 0.50 × 0.25 = 0.13 Reduction of calemn park -2.5 × 0.50 × 0.50 × (1.12 - 0.343) = -0.49 Shed load reaction

Panel paint 2

· Uniformly distributed load

0.43 4 0.25 x 5-12

= 0.55t

Unitormly distributed load

0.43 1/2 × (2.72 + 4.87) × /2 × 2.15 = 3.51

parmet load

1.1 4m3 x 0.05 x 1.43 x 5.12

- 0.40

longitudinal beam

2.5 4 x 0.50 × 0.45 × 4.62

2.60

Contilever slab

2.5" x (0.15 + 0.25) x 1/2 x 1.43 x 5.12 = 3.66

Haunch of slab

25 x 0.30 x 0.10 x 1/2 x 4.32

0.16

Beam support

2.5 x (0.30+0.50) x /2 x 0.50 x a 25 = 0.13

Reduction of column part

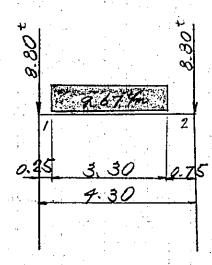
- 25 × 0.50 × 0.50 × (1.12 - 0.343) = -0.49

Shed load reaction

= 10.00

P2 = 20.52+

e) Reaction of Platform beam and stairway Refer



- (2) Intermediate member
 - a) linear load

Cross beam 2.5 this 0.50 × 1.02 = 1.28 /m

Bean support

2.5 x (0.30+0.50) x /2 x 0.50 = 0.50

Wd4 = 1.78 4m

b) Concentrated load acting at Panel Paint

Panel point 3. (4)

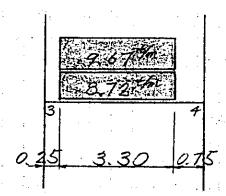
Reduction of linear load

-1.78 m x 0.25

= -0.45t

P3=P4=0.45t

c) Reaction of stairway



d) Oun weight of column

& = 2.5 this x 0.50 x 0.50 = 0.63 th

- 2. Pedestrian land (People load) $w = 0.35 \, t_{m}^{2}$
 - (1) Upper member
 - a) Unitormly distributed load $wd_1 = 0.35 \%^2 \times 0.25 = 0.09 \%$ $wd_2 = 0.35 \times (0.25 + 2.15) = 0.84$
 - b) Concentrated load acting at panel point

 panel paint 1.

 Uniformly distributed load

 0.35 × 0.25 × 5.12 = 0.45 t

Uniformly distributed load

0.35'x(2.72+4.87) $\times //2 \times 2.15 = 2.86$ $P_1 = 3.31^{\pm}$

Panel point 2
Uniformly distributed load

0.35 the x 0.25 × 5.12 =

Unitornly distributed load

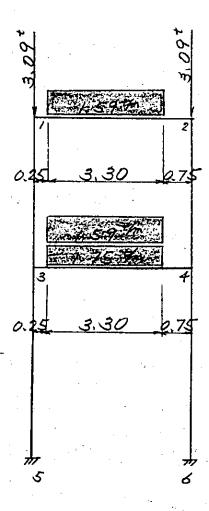
0.35 $\frac{4}{10}$ (2.72 + 4.87) × $\frac{1}{2}$ × 2.15 = 2.86"

Unitornly distributed load

-0.35 1/2 × 1.43 × 5.12 = 2.56

0.45

c) Reaction of platform beam and Stainway



3. Temperature change and Drying contraction

Temperature change
$$----- \pm 10^{\circ}$$
C Prying contraction $------ - 15^{\circ}$ C

Hence,

4. Seismic load from dead load (KH=0.10)

a) Horizontal force acting at appear member Refer Dead load and Pedestrian land

$$(0.11 + 1.03) \times 1/2 \times 2.15 \times 2 = 2.45^{t}$$

$$1.75^{th} \times 4.30 = 7.53^{\circ}$$

$$19.46^{t} + 20.52^{t} = 39.98^{t}$$

$$8.80^{\circ} \times 2 + 9.67^{th} \times 3.30 = 49.51^{\circ}$$

$$(7.42 + 6.29)^{t} \times 1/2 = 8.86^{\circ}$$

$$0.21^{th} \times 6.23 \times (4.87 + 0.25) = 6.70^{\circ}$$

$$0.63^{th} \times 3.998 \times 1/2 \times 2 = 2.52^{\circ}$$

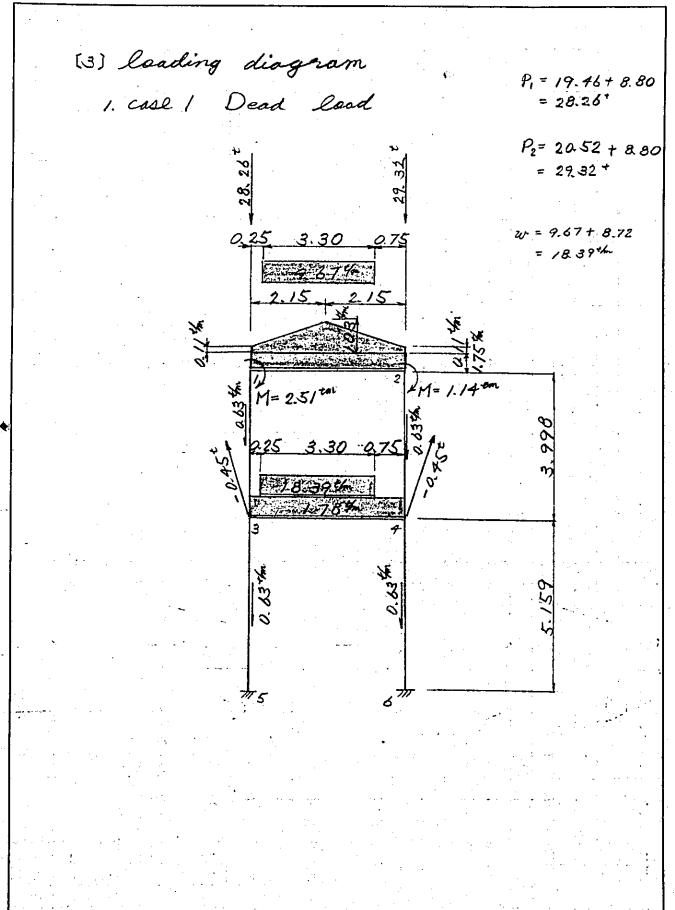
$$Total = 115.55^{t}$$

Hi = 115-55 x a 10 x 1/2 = 5-70t

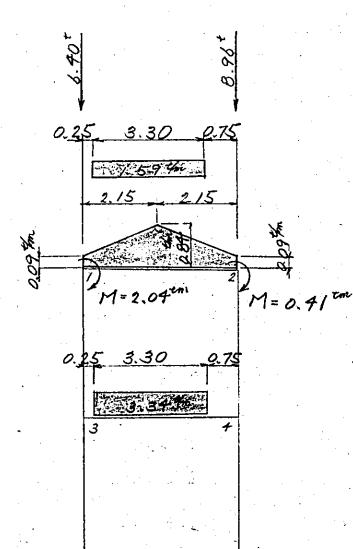
b) Horizontal force acting at intermediate member

c) Increased horizontal load acting at foundation

0.63 th x 5.159 x 1/2 x 2 = 3.25 t



2. case 2 Pedestrion load (People load)

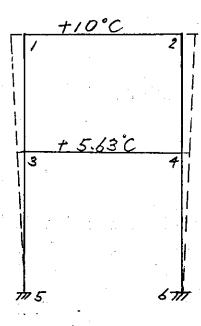


P1= 3.31 + 3.09 = 6.40*

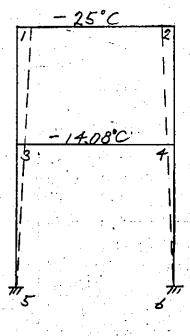
P2 = 5.87 + 3.09 = 8.96 =

W = 1.59+1.75

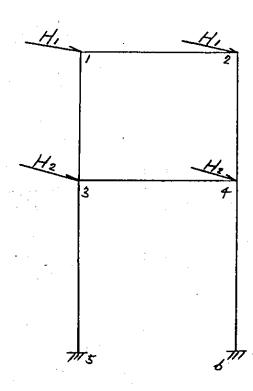
3. case 3 Temperature rise



9: case 4 Temperature drop + Prying contraction



5. cose 5 Se ismic load from dead load



$$H_2 = 3.99^{t}$$

[7] Combination of loads 1. Basic load

Case Nomber	Kind of load	Loading Pattern
/	Dead load	
2	Pedesdrian load	
3	Temperature rise	+10°C
4	Temperature drop + Drying contraction	- 25°C
5	Seismic boad from dead load	1-1

2. Combined loads

csse	Combination of loads	2
6	/ + 2	1.0000
7	/ + 3	0.8696
8	1 + 4	,
9	1 + 5	0.6667
10	1 - 5	. .

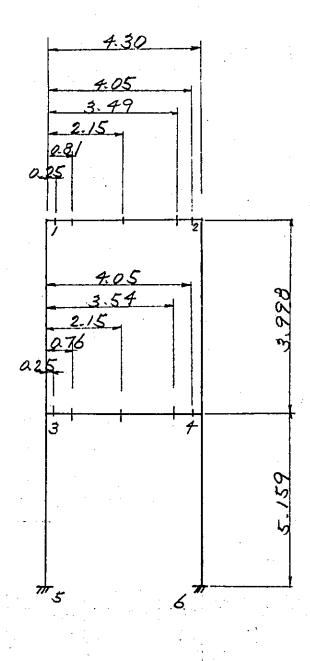
3. Critical cases

No. 1 case 1 crack

No. 2 case 6 ~ 10 Synthetic

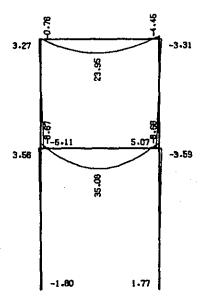
No. 3 case 9. 10 footing

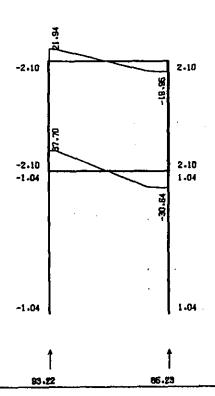
9. Point of stress calculation



CASE 1 (DEAD LOAD)

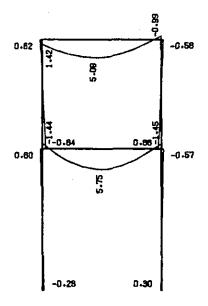
BENDING MOMENT

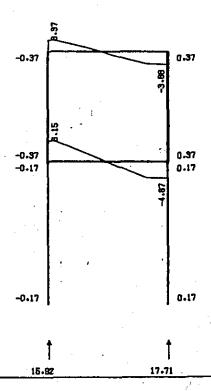




CASE 2 (PEDESTRIANS LOAD)

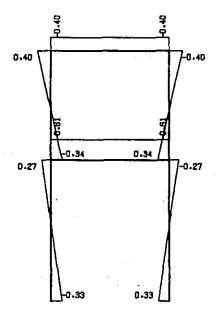
BENDING MOMENT

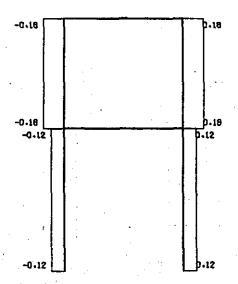




CASE 3 (TEMPERATURE)

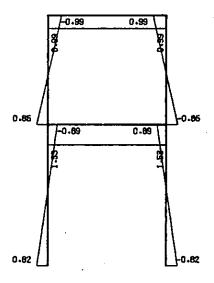
BENDING MOMENT

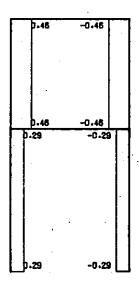




CASE 4 (TEMPERATURE + SCHRINKAGE)

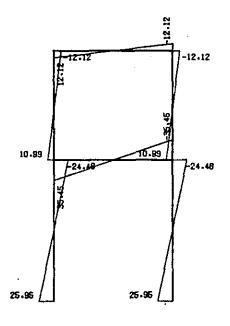
BENDING MOMENT

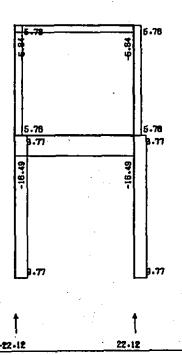




CASE 5 (SEISMIC LOAD)

BENDING MOMENT





INPUT DATA

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<u>ئ</u> و، ع		E	0		21.941	14.879	-17.719	-19.946	-1.420		37.700	26.908	-29.104	-36.641	-1.068		-2.097		2.097		-1.639		1.039		ee.	1 : 1 : 1
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ITAN 0.000 (10) 18.676 -7.206 -74.732 ITAN 6.000 (9) -13.934 5.821 -47.400 JAN 5.159 (10) -18.502 -7.206 -7 = HEMBER 6 (4 - 6) C = = $\frac{1}{2}$ HEMBER 6 (4 - 6) C = $\frac{1}{2}$ JAN 0.000 (9) -15.697 7.206 -6 JAN 5.159 (10) -15.118 -5.621 -4	= HEHBER 5 C	- 5 1 6		= HEMBER 5 1	= 315 =		
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TITLE VIADUCT D	OF PLATFORM (NORTH SIDE)	IDE) C-1			CRC-FANSY	1SY V6.3	•
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3 2.150	. 10)		05 05	3 2.150	(9) 23,	!	1.253
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JIAN 0.000 JIAN 3.998	(9) -5,896 (9) 3,921	2,456 -29.7	712 1 391	TAN 6.000	(10) 16.2 (10) -10.7	59 -5.251 36 -5.251	-37,226
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- -	-		150 (1	23.388	10,280	705	
3.540. (6) 16.365	-33.943 1.253	.	3,540 (10)	24.687	-8-411	705	
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CRC-FANSY V6.3

TITLE .. VIADUCT OF PLATFORM (NORTH SIDE) G-1

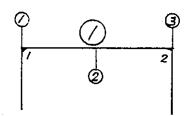
FIGH OF 3 FORTION FAXTRON FORTION FO		אואטאן באניבוד אום ביים	KIN STUE	2-1			CRC-LANSY	Vo.J	!
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HAX				7+C+C-	050011	- 0ac**	*******	Ţ	•
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TAN 0.000 9 17.855 14.142 .775 1784 0.000 10 -20.412 36.127 36.127 2.550 10 -2.666 3.65 3.65 3.755 3.755 3.550	= NEMBER	- 4				- 光阳氏 2 (3	= 9 C		
1	TAN	(6)	-	14, 142	.705	TAN 6.000 (1	29	36.127	.705
2 .760 (91 26.666 6.987 7.705 2 .750 (91 23.988 -31.714 4 3.540 (10) 24.667 -3.411 7.705 4 3.540 (91 23.988 -31.714 4 3.540 (10) 24.667 -3.411 7.705 5 4.350 (91 23.988 -31.714 4 3.540 (10) 17.659 -9.436 7.705 5 11.40 (91 23.988 -31.704 4 3.540 (10) 17.659 -9.436 7.705 5 11.40 (91 23.988 -31.704 4 3.000 (10) 17.659 -9.436 7.705 5 11.40 (91 23.988 -31.704 1 3.000 (10) 17.659 -9.436 7.705 5 11.40 (91 23.988 -31.704 1 3.000 (10) 17.659 -9.436 7.705 5 11.40 (91 23.988 -31.704 1 3.000 (10) 16.259 -5.251 -37.226 1 17.44 0.000 (91 3.0		(6,	- }	13,845	.705	1 ,250 (1	20.	35.630	.705
17. 1.433 (9) 24.687 24.41 2765 31.40 (9) 25.672 31.124 31	,	5 C		6.987 10.285	705	.760 t 1	3.893	28.972	705
JANN 1.433 (9) 20.181 -9.139 .705 JANN 1.300 (9) -21.591 -31.324 HAX 1.433 (9) 20.23 -2.667 .705 JANN 1.300 (9) -20.409 -31.421 TINN 1.433 (9) 20.323 -2.667 .705 JANN 1.300 (9) 23.368 -11.704 = HERBER 4 (2 - 4) C =		10.1	1	-8.411	705	3.540	5.872	-30,396	.705
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TIAN	= MENBER 4	2 - 41				= MEMBER 4 (2	3 0		
= $\frac{114N}{114N}$ 0. C00 (10) 18.676 -74.732 IIAN 0.000 (9) -13.934 5.821 -45.2 JIAN 0. C00 (10) 16.096 5.821 -47.400 JIAN 5.159 (10) -18.502 -7.206 -76.8 = $\frac{114N}{114N}$ 0. C00 (10) 13.913 -5.821 -39.904 JIAN 0.000 (9) -18.697 7.205 -69.4 JIAN 0.100 (9) -18.697 7.205 -69.4 JIAN 0.100 (9) -16.118 -5.821 -42.0		(10)	.671	5.251	9.08	3.998 (10	2.01-7. -3.9	21.25	36.60 30.76
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3.490 (9)		{	7 4	3.490 (0 0	396	عم إس	-1,398
-,-	-6.820 -16.736 -11.044 -17.055		JIAN	4.300	10) 5.	111	o o	-1,398
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(6) 092		0.4	1	.250 (: -¦-	17	35.630	-705 206
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= MEMBER 6 (4 - 6) C	= =	3	= = ME ARE	R 6 C 4	D 9	# #		
JIAN 0.006 (16) 1	13.913 -5.821 -15-118 -5.621	-39,964	ITAN	0.000.0 0.000.0	9 1 18	. 697 . 480	7.205	-69,403 -71,569
				1	1	1	:	

- (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	l co.
Combined	Top	-8.59	10	=1LoA	2,
Seress	Boccon	10.25	9	7.46	10
Dead L	oad	-0.76	1. 1.	24.45	

(ii) Span momex

			. •
		@	co.
Combined Scress	Bottom	28.36	3

(Note 1) Dead load is of Pick-up No. 1

(Noce 2) co. - combination

(b) Allowable seress for upper beam, safe against cracking

(i) At the support point 2

Dead load Md = -4.45 (case 1)

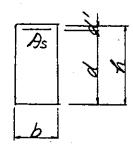
Pedstrian loadMp = -0.99'(case 2)

IM = -5.44 m

 $d = \frac{0.99}{5.44} = 0.18 < 0.25$

Hence Isa= 1200 tome

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



(ii) Cross section at the span center point

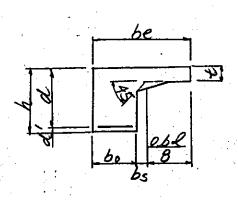
Effective width of T-beam at

compression fibre

$$be = b_0 + b_5 + \frac{0.6}{8} \mathcal{L}$$

$$be = 0.50 + 0.10 + \frac{0.6}{8} \times 4.30$$

$$= 0.92^m \left(-\frac{9.74}{2} = 4.87^m \right)$$



$$b_0 = 50^{cm}$$

$$h = 1/2^{-1}$$

$$d' = 3.0 + 1.3 + 1.3 = 5.6^{cm}$$

$$d = 1/2 - 5.8 = 106.4^{cm}$$

$$t = 15^{cm}$$

	(d)	Calcul	rtion o	f bendir	ig stress
\		3)	. ②	
М	(.tm)	- 4.45	-11.04	28,96	
N	(1)	1,5 / 2,7			
S	(1)		· · · · · · · · · · · · · · · · · · ·		
Ь	(cm)		0	92	
h	(cm)	- //		1/2	
d	(cm)		4.0	1064	
ď	(cm)		8.0	3-6	
				025-5	
As	(cm²)	025-5	= 25.34	المدانا	
p			487	0.00259	
			<u> </u>	t=15	
As'	(cm²)			t = 0.141	
p'					
	/N (cm)				
	(cm)				
	/N+ u /N- u		** .		
e-m	/ /v — u				
d/e	·				
$\frac{d'}{d'/h}$					
$\frac{d}{d'}/d$					
	4(kg/cm²)	0.82	2 24	0 70	
k	. CAR/CITE /	0.02	2.04	278	
C	•			0.27/	
	· · · · · ·			1	
$\frac{j}{1/L}$		· · · · · · · · · · · · · · · · · · ·		0.938	
1/L.			07		
$\beta = \sigma s$		2.2	29		
ļ 				001	
	(kg/cm²)		14.4	28.4	
as -	(kg/cm²)	190	470	1150	
	(kg/cm²)		1 2 2 2	24-0	
asa	(kg/cm²)	1200	1 800	1 800	
σсα	(kg/cm²)		90	90	
Ta Nost	(kg/cm²)				
	per	M-1	4	M-47. 48	
combin	ration	0	DTP		

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

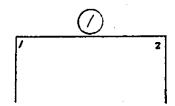
$$As = \frac{M}{\text{Rsa} \cdot f \cdot d} \times \frac{4}{3}$$

$$= \frac{11.04 \times 10^{5}}{1800 \times 0.875 \times 112.0} \times \frac{4}{3} = 8.34^{cm^{2}}$$

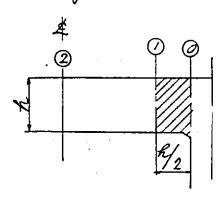
(ii) At the span Center point

$$As = \frac{28.96 \times 10^5}{1800 \times 0.075 \times 106.4} \times \frac{4}{3} = 23.04^{cm^2}$$

2. Shearing stress of upper beam



(1) Summary of shearing stress



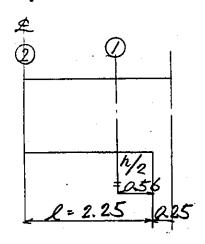
For the design of crass section of beam end, the value of shearing stress at 42 point is applied.

		(7)	
	Left support	co.	Right support	co.
0	24.80		-23.3/	
Ø	17.17		2132	7
2	- 2-07	0	-207	

(Note) CO. - combination

(2) Shearing stress

(a) Shearing stress caused by bending

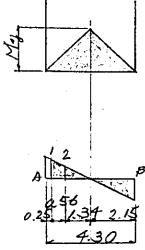


(1) Shearing siress of the member of unitom height

$$\gamma_1 = \frac{21.32 \times 10^3}{50 \times 109.0} = 4.10 \text{ finit} > 3.9 \text{ for }^2$$

(b) Calculation of Torsional moment

- 1) Torsional moment caused by
 - i) Dead load and Pedestrian load. $Wd = 0.43^{\frac{4}{100}} + 0.35^{\frac{4}{100}} = 0.78 \frac{4}{100}$ (From the load calculation)



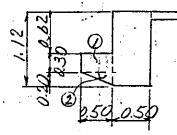
Fixed end moment,

at negative side $M_3 = -\frac{1}{24} \text{ wd } lx^2$ $= -\frac{1}{24} \times 0.78 \times 4.30^2 = -0.80^{\text{cm}}$ $M_{TA} = M_{TB} = -0.60 \times 2.15 \times 1/2 = -0.65^{\text{cm}}$ $M_{TI} = -0.65 \times \frac{1.90}{2.15} = -0.57^{\text{cm}}$

ii) Torsional moment caused by the

Beam support

 $M72 = 9 \times \frac{134}{215} = -0.41$

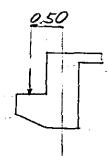


 $W_{70} = 2.5 \times 0.50 \times 0.30 \times 0.50 = 0.19^{\text{TM}}$ $W_{70} = 2.5 \times 0.50 \times 0.20 \times 1/2 \times 0.67 = 0.08^{-1}$

WT = 0.27 cm

$$M_{TA} = M_{TB} = 0.27 \times 4.30 \times 1/2 = 0.58^{-10}$$
 $W_{Ti} = 0.58 \times \frac{1.90}{2.15} = 0.51^{-10}$
 $W_{T2} = 1 \times \frac{1.34}{2.15} = 0.36$

iii) Torsional moment caused by the Stainway.



$$W = 9.67 + 1.59 = 11.26$$
 (From the load calculation)

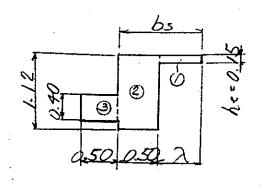
$$M_{TA} = 11.26 \times 0.50 = 5.63^{\text{cm}}$$

$$M_{TA} = M_{TB} = 5.63 \times 4.30 \times 1/2 = 12.10^{\text{cm}}$$

$$M_{TI} = 12.10 \times \frac{1.90}{2.15} = 10.69^{\text{cm}}$$

$$M_{T2} = 7 \times \frac{1.34}{2.15} = 7.54^{\text{cm}}$$

V) Torsional moment, beared by cross beam



Effective width $bs = bo + \lambda \epsilon$ $\lambda \epsilon = 3 \cdot h \epsilon = 3 \times 0.15 = 0.45^{20}$ $bs = 0.50 + 0.45 = 0.90^{20}$ $h = 1.12^{20}$

a) Calculation of distribution ratio

	a	Ь	a/b	R	It = R.a. b3 (m4)
0	0.45	0.15	3.000	0-263	0.263 × 0.45 × 0.15 = 0.000 40.
2	l i				0.239 × 1.12 × 0.503 = 0.03346
3					0.169 x 0.50 x 0.40 = 0.0541
	-	Total			ZIt=0.03927 m4

Front face of column

Me1=10.63 × $\frac{0.03346}{0.03927}$ = 9.06 tm

At the
$$h/2$$
 point
$$Mt2 = 7.49 \times " = 6.38"$$

c) Shearing stress caused by corsion is calculation fellowed the equation.

- b: Sharder side length
- a : longer side length
- £: Table-40.2
- (i) Front face of column

$$\frac{a}{b} = \frac{1/2}{50} = 2.24$$
 $\ell = 0.948$

(ii) At the M2 point

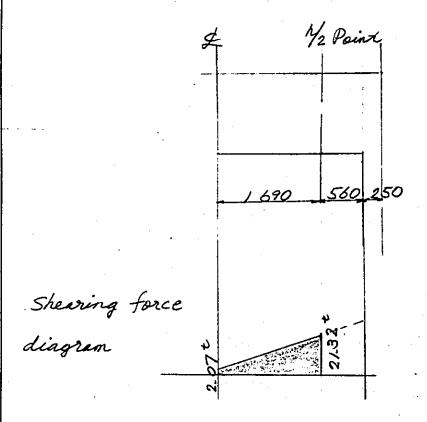
$$\frac{A}{b} = \frac{1/2}{50} = 2.24 \qquad \mathcal{U} = 0.948$$

VI) Combined shearing shress

Combined allowable shearing stress $7a = 17 \times 1.3 = 22.1^{120}$

Celculated as above, diagonal cension re-bars are examined.

(3) Calculation of diagonal cension re-bars
(a) Shearing stress Caused by bending



(i) Shearing stress beared by concrete

where Tc: 3,9 Fgom2.

b: Width of member (cm)

d: Effective height of member

at the examining section.

Sc1 = 1/2 x 3.9 x 50.0 x 10 +0 x 10 -3 = 10.14t

(b) Shearing torce beared dy stirrup

Arrange Stirrups D13-1 Sets in 12.5 cmcc Torsional Shearing stress

Whore

Mc: Torsional moment (cm)

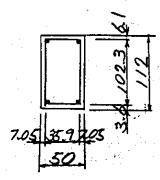
S: de distance of stirrup (cm)

Av : Gross cross Section of

coupled stirrups (cm²)

bi, hi : length of short long side of stirrup

· (i) Ae M/2 point



$$A_{2} = 1267 \times 2 = 2.53^{cm2}$$

Bending Shear beared by stirrup

In the case When Combined with consional moment, allowable shearing stress is

as 20 percent increased.

Sa = 1800 × 1.2 = 2160 Flore

Sv = (Psa-Pst) · Av · d

1.15 · D

(i) $Ax \frac{h}{2} paint$ $(2160 - 1070) = 1090 \frac{kg/cm^2}{1800} 1800 \frac{kg/cm^2}{d} = 104.0 cm$

Su = 1090 x 253 x 1040 = 19.95 +

(C) Shearing sires beared by curned up burs

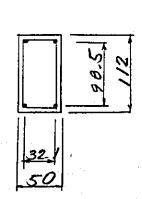
Disregarded the curned up burs for calculation

Total shear

 $\Sigma S_R = S_C + S_D + S_6$ $\Sigma S_R = 10.14 + 19.95 + 0 = 30.09^t >_{S=21.32^t}$ (d) Calculation of axial re-bar arrangement, resisting consional moment

Required re-bar arrangemet is calculated followed she equation.

a) Front face of column



$$Mt = 9.06^{em}$$
 $Jsa = 1.800^{k\theta/cm^2}$
 $b_1 = 32.1^{cm}$

Required cross section of re-bars

arranged at sharter side $Asb/ = 25.99 \times \frac{32.7}{2(32.7 + 98.5)} = 3.19^{cm^2}$

3. Required cross section. of re-barss arranged at longer side

 $Ash_1 = 25.99 \times \frac{98.5}{2 \times (32.1 + 98.5)} = 9.80^{cm^2}$

(1) Top and Battom

Minimum section of re-bers

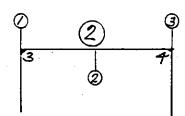
As= D25 - 5 = 25.37 > 3.19cm2

(ii) Side (one side)

As = D22 - 3 = 11.61 m2 > 9.80 m2

(7) Calculation of Intermediate beam

1. Stress calculation of upper beam



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

Pick up No. 2

			(2	2)	
		0	 CO.	3	co.
Combined	Top	-29.41	110	29-41	10
Stress	Boccon	20.35	9	20./8	9
Dead L	Coad	-8.67	/	8.66	J.

(ii) Span momet

		2	
4		@	<i>co.</i>
Combined Stross	Bottom	70.83	-6
			1

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

- (b) Allowable stress for Intermediate beam safe against cracking
 - (i) At the support point 4

Dead load

Md = - 8.66 (case 1)

Pedstrian load Mp. = - 1.45 (case 2)

IM = -10.11th

 $d = \frac{-1.45}{-10.11} = 0.14 < 0.25$

Hence Isa = 1200 Mone

	(C)	Calcul	ation o	t bendu	ng stress
			(2)		
			3	2	
M	(.tm)	- 8.66	-29.41	40.83	
N	(1)				
S	(1)				
Ь	(cm)	5	0	50	
h	(cm)	102	2	102	
d	(cm)	9.	3.9	96.4	
ď	(cm)		B. [5.6	
As	(cm²)		5	025-3	
		025 -	= 30.40		
p	···	0.00	2647	0.00841	<u> </u>
As'	(cm²)	·			
p					
	M/N (cm)				
<u> </u>	(cm) M/N+ u				
	M/N-u		· · · · · · · · · · · · · · · · · · ·		
e/h		·	·		
d/e					
d' /	h				
d' /	d				
Ne/b	d4(kg/cm²)	2.01	6.67	8.79	
k			·	,	
с					
j					l
1/1	Lc	6	40	5-87	
1/1			75	137	
β= σ	ıs/ oc	<u> </u>			
σς	(kg/cm²)	·	42.7	51.6	
σs	(kg/cm²)	350	1170	1200	
τ	.(kg/cm²)				
osa	(kg/cm²)	1200	1800	1 800	
σсα	(kg/cm²)	· · · · · · · · · · · · · · · · · · ·	90	90	
ra	(kg/cm²)	·			, , , , , , , , , , , , , , , , , , ,
nun	rber	M-1	,	77	
	ination	\mathcal{D}	D+P	,	•

- (d) Required minimum cross seccion of re-bars
 - (i) At the top of support point

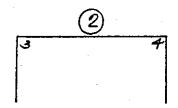
$$As = \frac{M}{\text{Dsa} \cdot \dot{f} \cdot \dot{d}} \times \frac{4}{3}$$

$$= \frac{29.41 \times 10^{5}}{1800 \times 0.875 \times 93.9} \times \frac{4}{3} = 26.51$$

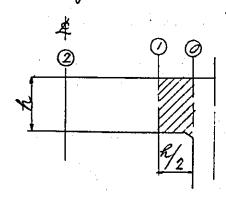
(ii) At the span center point

$$As = \frac{40.83 \times 10^5}{1800 \times 0.875 \times 96.4} \times \frac{4}{3} = 35.86^{cm^2}$$

2. Shearing stress of upper beam



(1) Summary of shearing stress

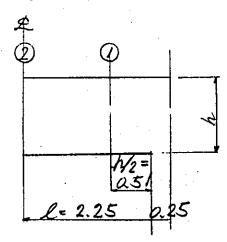


For the design of crass section of beam end, the value of shearing stress at 1/2 point is applied.

	e de la compositione e	(3	
	Left support	co.	Right support	co.
0	43.41		-35-07	
0	31.42	6	-03.94	6
2	- 1,26		7.26	

(Note) CO. - combination

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



1) Shearing scress of the member of uniform height

$$\gamma_1 = \frac{33.94 \times 10^3}{50 \times 93.9} = 7.23 \frac{19}{100^2} > 3.9 \frac{19}{100^2}$$

$$C_2 = \frac{1.26 \times 10^3}{50 \times 93.9} = 0.27$$

Therefore, Stirrup calculation is made.

(i) Shearing stress beared by concrete

Sc= 1/2. Tc. b. d

Where, To: 3.9 kg/cmi

b: Wideh of member (cm)

d: Effective height of member

at the examining section.

Sc1 = 1/2 × 3.9 × 50 × 93.9 × 10-3= 9.16+

(ii) Shearing force beared by stirrup $Sv = \frac{Av \cdot V_{sa} \cdot d}{1.15 \cdot 8}$

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

Vsa: Allowable censile stress of re-bar
Vsa= 1800 ra/cm²

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

Arranged stirrups D13-1 sets 15 cm ctc, $Au = 1.267 \times 2 = 2.53$ cm²

Su, = 2.53 x 1800 x 93.9 = 24.79 t

Calculation of diagonal cension bar

Calculation of total shear

Referr R.C. standard 39, (2).(a).

ISR = Sc+Su+Sb

where

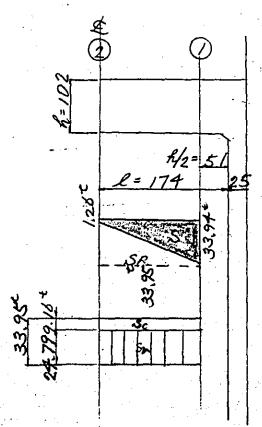
Sc: Shearing stress beared by concrete(t)

Sv: Shearing stress beared by stirrup (t)

Sb: Shearing stress beared by turned

up bars (t)

Assumed Sv ≥ Sb



Shearing force diagram

Resisting shearing force diagram

- (iii) Shearing stress beared by eurned up bars
 Disregarded the eurned up bars
 for calculation.
- (iV) Total shear $\Sigma SR = Sc + Su + Sb$

$$\sum S_{R} = 9.16 + 24.79 + 0$$

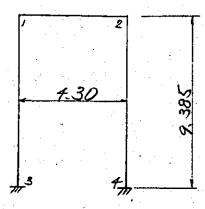
$$= 33.95^{t} > S_{r} = 33.94^{t}$$

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

§6. Rigid from analysis on transversal section 2-2 of elevated structure

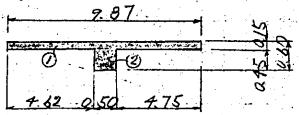
[1] Basic calculation

1. Axis of Rahmen (Rigid Frame).



2. Cross sectional area! Moment in inertia of area of member

(1) Upper beam (Member 1-2)

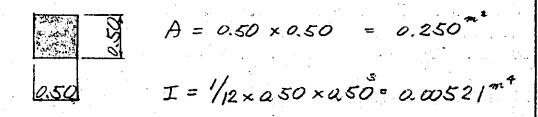


	b.m.	hm	A (mi2)	72 (00)	A 2 (m3)
0,	987	0.15	1.481	0.075	0.11108
2	0.50	0.45	0.225	0.375	0.08438
Σ			1.706		0.19546

	b (m)	h (m)	A (m²)	-g o (m)	Io (-114)	A 3 8 (m)	Io + A-7 201
0	9.87	0.15	1.431	0.040	0.00278	a 00237	0.005.15
2	0.50	a.45	0.225	a 260	a. 00380	a 01521	0.01901
Σ		·	1.706		a 00658	a0/758	0.02416

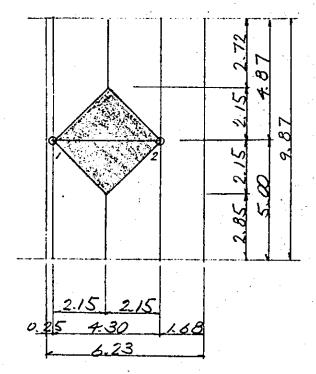
Cross sectional area $A = 1.706^{m^2}$ Moment of inlatia $I = 0.02416^{m^4}$ of area

(2) Column (Member 1-3.2-4)



(3) axial height $h = 9.50 - 0.115 = 9.385^m$

- (2) Load calculation
 - 1. Dead load
 - (1) Upper member



a) Unitormly distributed load

Slab $2.5 \frac{4m^3}{11} \times 0.15 = 0.375 \frac{4m^2}{11}$ Parmet $1.1' \times 0.05 = 0.055$

Wd = 0.43 7m2

Wd1 = 0.43 m2 2.15 x 2 = 1.85 m

b) linear load

cross beam $2.5^{1/3} \times 0.50 \times 0.45 = 0.563^{1/3}$ Haunch of slab $2.5 \times 0.30 \times 0.10 \times 1/2 \times 2 = 0.075^{\circ}$ $200 \times 0.10 \times 1/2 \times 2 = 0.04^{1/3}$

C) concentrated load acting at panel point

Panel point 1

- Unistormly distributed load $a43^{4n^2} \times 0.25 \times 9.87 = 1.06^{t}$
 - Unitormly distributed load

 0.43 4/2 × 2.15 = 3.51
 - Uniformly distributed load $0.43^{1/2} \times (2.85 + 5.00) \times 1/2 \times 2.15 = 3.63$
- " Haunch of slab 2.5 1/2 × 0.30 × 0.10 × 1/2 × 8.77 = 0.33"
- longitudinal beam
 25^{4/2} × 0.50 × 0.45 × 9.87 = 5.55°

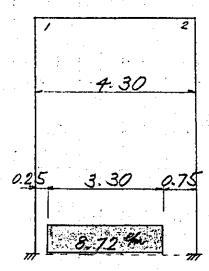
Reduction of linear load
$$-0.64^{44} \times 0.25 = -0.16^{4}$$
Reduction of calumn part
$$-2.5^{44} \times 0.50 \times 0.50 \times (0.15 - 0.115) = -0.02^{2}$$
Shed load reaction
$$= 13.00^{2}$$
Panel paint 2
Uniformly distributed load
$$1.06^{2} + 3.51^{2} + 3.63 = 8.20^{2}$$
Contilever Slob
$$2.5^{46} \times (0.15 + 0.25) \times 1/2 \times 1.43 \times 9.87 = 7.08^{2}$$
Parmet load
$$1.1^{46} \times 0.05 \times 1.43 \times 9.87 = 0.78^{2}$$
Haund of slob (same as panel point)= 0.33²
Reduction of linear load
$$(2.5^{46} \times 0.35 \times 1.43 \times 9.87 = 0.78^{2}$$
Reduction of linear load
$$(2.5^{46} \times 0.35 \times 1.43 \times 9.87 = 0.78^{2}$$
Reduction of linear load
$$(2.5^{46} \times 0.35 \times 1.43 \times 9.87 = 0.78^{2}$$
Reduction of linear load
$$(2.5^{46} \times 0.35 \times 1.43 \times 9.87 = 0.78^{2}$$
Reduction of linear load
$$(2.5^{46} \times 0.35 \times 1.43 \times 9.87 = 0.78^{2}$$
Reduction of column park

Shed Load reation

= 10.00

Po = 31.74t

d) Reaction of stairway



(2) Oun weight of column

q = 2.5 9 × 0.50 × 0.50 = a 63 4 m

- 2. Pedestrion load (People load) $w = 0.35^{-4/2}$
 - (1) Upper member
 - a) Unitormly distributed load wd, = 0.35 x 2.15 x 2 = 1.51 %.
 - 2) Concentrated load acting at panel point

Panel point 1

- · Unisormly distributed load

 0.35 1/2 x a 25 x 9.87 = a.86 t
- · Unitorally distributed load

 0.35 1/2 x (2.87 + 4.87) x 1/2 x 2.15 = 2.91
- : Unisormly distributed load.

 0.35 1/12 × (2.85 + 5.00) × 1/2 × 2.15 = 2.95°

P1 = 6.72 t

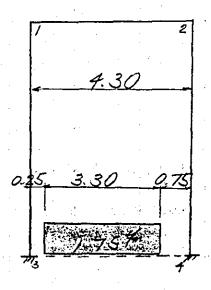
Panel paint 2

· Unitoraly distributed load 2.91+2.95 = 5.86

0.35 × 1.68 × 9.87 = 5.80°

P2 = 11.66

(2) Reaction of Stairway



3 Temperature change and Donying contraction

Hence,

4. Seismic load from dead load (*KH=0.10)

a) Horizontal force acting at apper member

$$1.85^{44} \times 2.15 \times 1/2 \times 2 = 3.98^{\pm}$$

$$0.64^{*} \times 4.30 = 2.75^{*}$$

$$26.90^{\pm} + 31.74^{\pm} = 58.64^{*}$$

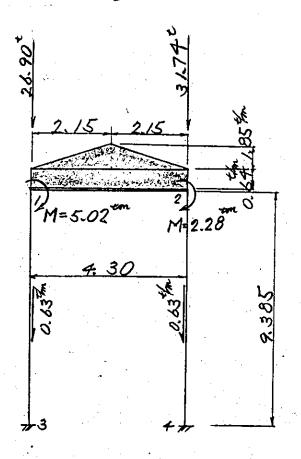
$$0.21^{442} \times 6.23 \times 9.87 = 12.91^{*}$$

$$H = 84.19 \times 0.10 = 8.42^{e}$$

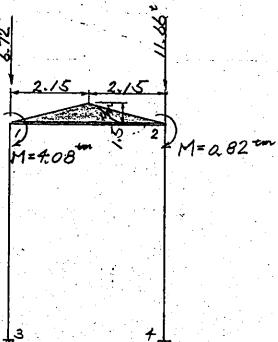
 $H_1 = H_2 = 8.42 \times 1/2 = 4.21^{e}$

b) Horizontal force acting at foundation

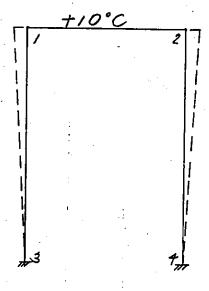
(3) loading diagram 1. case 1 Dead Load



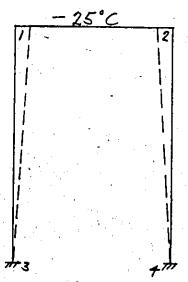
2. case 2 Pedestrians load



3. cue 3 Temperature rise

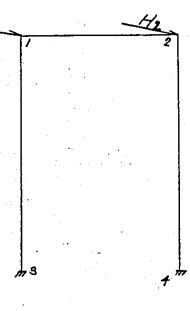


9. case 4 Temperature drop + Drying contraction



5. case 5 Seismic load from dead load

H1= H2 = 5-78t



(7) Combination of loads

1. Basic load

Case Nomber	Kind of load	Loading Pattern
/	Dead load	1
2	Pedestrian load	
3	Temperature rise	+ 10°C
4	Temperature drop + Drying contraction	- 25°C
5	Seismic load from dead load	

2. Combined loads

Case	Combination of loads	٧.
6	1 + 2	1.0000
7	/ + 3	0.8696
8	1 + 4	
. 9	/ + 5	0.6667
10	1 - 5	,

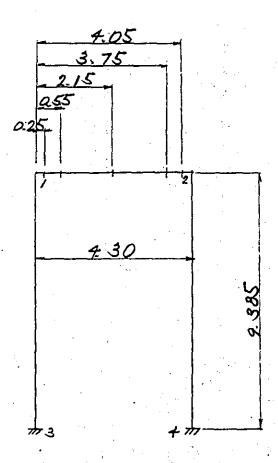
3. Critical cases

No.1 case 1 crack

No. 2 case 6 ~ 10 synthetic

No.3 case 9.10 footing

T. Point for stress calculation

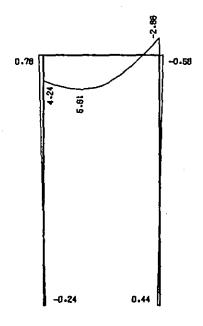


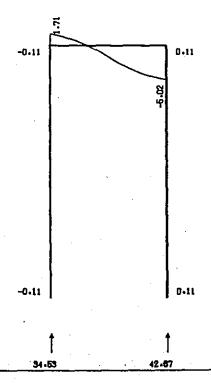
*

(S) S tress diagram
VIADUCT OF PLATFORM (NORTH SIDE) C-2

CASE 1 (DEAD LOAD)

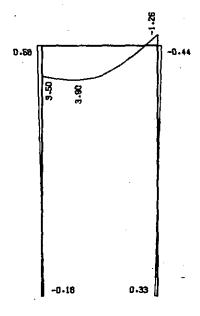
BENDING MOMENT

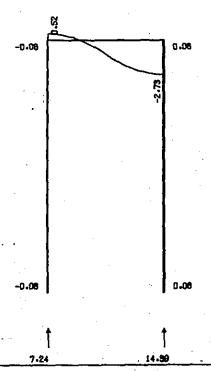




CASE 2 (PEDESTRIANS LOAD)

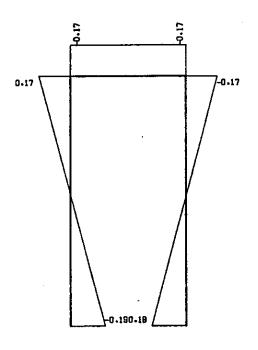
BENDING MOMENT

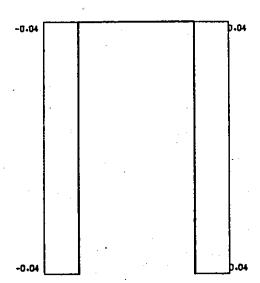




CASE 3 (TEMPERATURE)

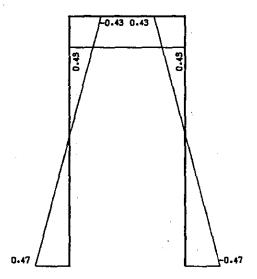
BENDING MOMENT

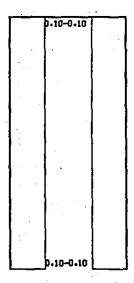




CASE 4 (TEMPERATURE + SCHRINKAGE)

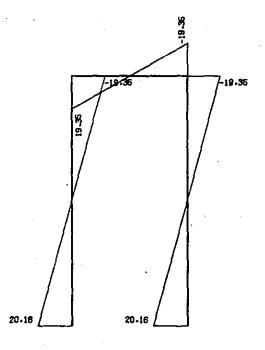
BENDING MOMENT

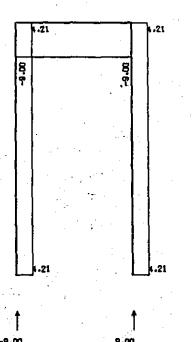




CASE 5 (SEISMIC LOAD)

BENDING MOMENT





A STATE OF THE STA

Approximately the state of the

i

IMPUT DATA	A COMPANY OF THE PROPERTY OF T	
A INDUI DAID ===	GRC-FA4SY Vo.3	
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100	VIABUCT OF PLATFORM (NORTH SIDE) C-2	
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CRC-FANSY V6.3

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TITLE . VIADUCT OF PLATFORM CNORTH SIDES G-2	2-5	CRC-FANSY V6.3	
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(8) 4.050 (8) 4.413	1.439011	0.000 (6) 2.737 2.231 250 (6) 8.270 2.022	
'	1.070011	(6) 8.823 1.642 (6) 8.465 -2.757)
177		3.750 (6)000 -7.157	
1 -2.11.7	-4.363011	4.300 (6) -4.120	
2 (1 - 3) 6 = =		= = HEHBER 2 (1 - 3) C = =	*
9,385 (9) -12,380	2.734 -13.076	11AN 6.000 (6) 1.363190 -35.851 JIAN 9.385 (6) -,424 -,190 -41.763	
3 (2 - 4) C = =	a little and the second	= = MEMBER 3 (2 - 4) C = =	
9,365 (10) 12,511 -	-2.734 -18.506 -2.734 -22.448	JIAN 0.000 (6) -1.020 . 190 -51.145 JAN 5.385 (6) . 767 . 190 -57.056	
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# HONE NI MINIMUM -2	#ONE NI MINITUM -1	OF PLATFORM (NORTH SIDE)	SIDE) 0-2	· · · · · · · · · · · · · · · · · · ·			CRC-FANSY	v6.3		
15.77	HONENT HAXIPUN									
9-16-17 1-2 0 = = = = = = = = = = = = = = = = = =	9-16.1 Hakiron: Hokiron: Hoken Hakiron:									
$\begin{array}{cccccccccccccccccccccccccccccccccccc$	$\begin{array}{cccccccccccccccccccccccccccccccccccc$	HCHENT HAXI	FUR .			Ĭ.	ONENT MINIMU	27	1	1
$\begin{array}{cccccccccccccccccccccccccccccccccccc$	$\begin{array}{cccccccccccccccccccccccccccccccccccc$		6	N		CASE	¥	3		:
9 15.727 -4.558073 114N 0.000 [10] -10.074 7.112 9 1 12.727 -4.558073 2 2.50 [10] -6.250 6.821 9 1 12.974 -5.180073 2 5.50 [10] -6.250 6.821 9 1 12.974 -5.180073 3 2.50 [10] -6.250 6.821 9 1 2.975 2 2.750 [10] 10.990 2.750 2 2.750073 4.750 [10] 10.990 2.750 2 2.750073 4.750 [10] 10.990 2.750 2 2.750073 4.750 [10] 10.990 2.750 2 2.750073 4.750 [10] 10.990 2.750 2 2.750073 4.750 [10] 10.990 2.750 2 2.750	$\begin{array}{cccccccccccccccccccccccccccccccccccc$	2)62			n	- 1 -	≥ 9 (
$\begin{array}{cccccccccccccccccccccccccccccccccccc$	$\begin{array}{cccccccccccccccccccccccccccccccccccc$		-4.858	073	•		-10.074	7.142	073	
10) 9.450 2.877 073 4 4.350 (9) -9.759 -9.759 -9.759 (9) 10.315 2.759 2.759 073 0	10) 19.450 2.977073 5.750 (9) -12.490 -9.751 -9.102 10.312 2.750 2.750073 5.750 (9) -12.400 -9.345 10.990 2.655073 5.750 (9) -12.400 -9.345 -7.102 10.990 2.655073 5.750 (9) -12.300 2.750 6.91 3.3457.102 10.990 13.279 2.734 -12.016 11.41 0.000 (9) -13.290 2.6500.10 13.279 2.734 -13.506 11.41 0.000 (9) -13.290 2.6500.10 13.734 2.85034.48 11.414 9.355 (10) -13.149 -2.734 13.734 2.85034.48 11.48 9.355 (10) -13.149 -2.734 13.734 2.850 13.448 11.48 9.355 (10) -13.149 -2.734 13.149 13.734 13.149 13.14	-	-5.18D -7.102	073	25	1 10	-6.226	6.821	673	
9) 3.345 -7.102073 HAX 2.150 (9) 3.345 -7.102 - 3) C = = HERER 2 (1 - 3) C = = 10) 13.421 -2.80	9) 3.345 -7.102073		2.977 2.780 2.655	073 073 073	4 J TAN	6 6 6 	-9.751 -12.489 -14.818	-9.023	-, 073 -, 673 -, 073	
10) 13.421 -2.880 -25.076 1124 0.000 $(-9.)$ -12.380 2.734 9) 13.279 2.734 -17.018 $= +4.96R$ 3 (-9.395) (-9.395) -13.290 2.734 9) 13.734 -2.734 -18.506 $= +4.96R$ 3 (-9.395) $= -4.10$ -3.3290 $= -4.10$ $= -4.$	10) $13.421 - 2.880 - 25.076$ $1724 - 12.380 - 2.734$ 9) $13.279 - 2.734 - 17.018$ $1724 - 3.385 (10) - 13.004 - 2.887$ 10) $12.511 - 2.734 - 18.506$ $174N 0.000 (9) - 13.290 2.660$ 10) $12.511 - 2.734 - 18.506$ $174N 9.385 (10) - 13.149 - 2.734$		-7,102	073	МАХ	6)	3,345	-7.102	073	
10) 13.421 -2.880 -25.076 1744 0.000 (9) -12.380 2.734 -17.018 15.79 2.734 -17.018 1744 9.395 (10) -13.604 -2.88[10) 13,421 -2.880 -25.076 1744 0.000 (9) -12.380 2.734 -2.88C - 4) $C = = +4$	- 3) C =			н	- 1 -	= 3 (
$ \begin{array}{cccccccccccccccccccccccccccccccccccc$	$ \begin{array}{cccccccccccccccccccccccccccccccccccc$	~~	-2.580	11	I Tan Jien	.000 (9 .385 (10	-12,380 -13,604	2.734 2.88E	-13.076 -29.018	
9) 12-511 -2.734 -18-506 11AN 0.000 (9) -13.290 2.660 -32.44	19. 12.511 -2.734 -18.506 1 1 1 1 1 1 1 1 1 1 1 1 1 1 1 1 1 1 1	= 3 (+ =			- 11	- 2 -	= 0 (
			-2,734	2,3	LIAN	.000 (9	200	2.5	30.50	;
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TIAN 0.000 (10) -10.074 7.142 073 TAN 0.000 (9) 15.727 -4.658 1.527 -4.658 1.527 -4.658 1.527 -4.658 1.527 -4.658 1.527 -4.658 1.527 -4.658 1.527 -4.658 1.527 -4.658 1.527 -4.658 1.527 -4.658 1.527 -4.658 1.527 -4.658 1.527 -4.658 1.527 -5.226 1.527			1		, ,						N	•
17aN 0.000 10 -10.074 7.142 -0.073 17aN 0.000 9 15.727 -4.658 2 2.50 10 -6.256 6.821 2.073 2.556 9 12.497 -5.186 2 2.550 10 3.345 4.899 -0.073 3 2.550 9 12.497 -5.186 2 2.500 9 1.345 -2.107 2 2.500 9 1.345 -2.107 2 2.500 9 1.345 -2.107 2 2.500 9 1.345 -2.107 2 2.500 9 -3.751 -2.751	= # MEMBER	- 1 -	#			11	R 1 (1	1 2	l"			
2 550 (10) -6.226 6.821 -073 2 .550 (9) 12.974 -5.180 (10) 13.45 4.899 -0.073 3 2.550 (9) 12.974 -5.180 (10) 13.45 4.899 -0.073 4 3.750 (9) -9.750 (9) -9.750 (9) -9.750 (10) 10.990 2.975 -0.073 4 3.750 (9) -9.			-10.074	7.142	073	ITAN	0.000	1 6	5.727	-4.658	073	† + + + + + + + + + + + + + + + + + + +
$\begin{array}{cccccccccccccccccccccccccccccccccccc$	2		-6.226	6.821	.073	727	550		2.974	5.180	073	
JTAN 4.300 ($\dot{10}$) 10.990 2.655673 JTAN 4.300 ($\dot{9}$) -14.812 -9.345 ITAN 0.000 ($\dot{9}$) -12.360 2.734 -13.076 ITAN 0.000 ($\dot{9}$) -13.279 2.734 -13.016 JTAN 0.000 ($\dot{9}$) -13.290 2.880 -30.506 ITAN 0.000 ($\dot{9}$) -13.290 2.880 -30.448 JTAN 9.355 ($\dot{10}$) -13.149 -2.734 JTAN 9.355 ($\dot{10}$) -13.149 -2.734		~~	9,450	2.977	073		3.750		9.751	-9,023	-,073	
# WENDER 2 (1 - 3) C = = = = #ERBER 2 (1 - 3) C = = = #ERBER 2 (1 - 3) C = = = #ERBER 2 (10) 13 421 -2.880		-	10.990	2.655	1.673	JTAN	4.360	-	219 m	-9.345	[73	
ITAN 0.000 (9) -12.380 2.734 -13.076 ITAN 0.000 (10) 13.421 -2.880 # NEMBER 3 (2 - 4) C = = = #EPPER 3 (10) -13.604 -2.880 ITAN 0.000 (9) -13.290 2.880 -30.506 ITAN 0.000 (10) -13.149 -2.734 JIAN 9.365 (9) -13.734 2.880 -34.448 JAN 9.385 (10) -13.149 -2.734		5 (1 -				į n	2 (M	P		:	
EMEMBER 3 (2 - 4.1 C = = 113.734			-12,360	2.734	-13.076	1 TAN UTAN	9.385 (3.421	-2.880 -2.880	-25.676	
0,000 (9) -13.290 2.880 -30.506 ITAN 0.000 (10) -2.734 -2.734 9.385 (10) -13.149 -2.734		2	*	2 80			R 3 (2	0 ()	H,	•		
9.385 (9) 13.734 2.880 -34.448 JFAN 9.385 (10) -13.149 -2.734			-13.290	2.880	-30.506	I TAN	-	_	2.511	-2.734	-18,506	
	1	<u>-</u> ا	13.734	2.880	-34.4.6	NAL	-	•	3.149	-2.734	-22,446	
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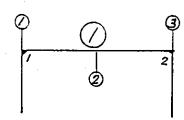
CRC-FANSY VE.3

ITLE. . VIADUCT OF PLAIFORM INORTH SIDES C-2

FIELE. VIADUCT OF PLATFORM (MORTH SIDE) C-2 CRG-FANST CASE
PLAIFORM (NORTH SIDE) C-2 AXIAL HANIPUL -CASE
PLAIFORM (NORTH SIDE) C-2 -CASE
AXIAL MAXIPUP -CASE
PLAIFORM (NORTH SIDE) C-2 AXIAL MAXIMUM -CASE
UP 3 AXIAL HAXIPU UP 3 AXIAL HAXIPU LOUDD (9) 15.727 250 (9) 14.497 250 (9) 12.974 2.150 (9) 12.974 2.150 (9) 12.974 3.750 (9) 12.974 4.300 (9) 12.974 4.300 (9) 12.310 0.000 (9) 12.380 9.365 (9) 12.380 9.365 (10) 12.511 9.385 (10) 12.511
WIADUCT OF PLATFORM UP 3 AX1 AX1 AX1
WIADUCT C
TTAN TIAN TIAN TIAN TIAN TIAN TIAN TIAN

(6) Calculation of upper beam

1. Stress calculation of upper beam



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

Pick up No 2

)	
		Ø	100.	3	l <i>co</i> .
Combined	Top	-10.07	10	= 74.81	200
stress	Boccon	15.73	9	10.99	10
Dead L	load			-2.87	131. Zani

(ii) Span moment

		② co.
Combined Stress	Bottom	847. 6

(Note 1) Dead Load is of Pick-up No. 1

(Note 2) co. - combination

(b) Allowable stress for upper beam, safe against cracking

(i) At the support point 2

Dead load Md = -2.87 tm (case 1)

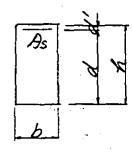
Pedstrian load Mp = - 1.26' (case 2)

IM = -4.13tm

 $d = \frac{1.26}{4.13} = 0.31 > 0.25$

Hence Isa = 1000 Mone

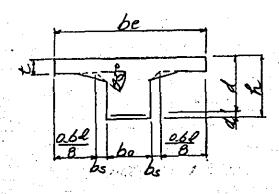
- (C) Cross section used for stress calculation
 - (i) Cross section at the support point



(ii) Cross section at the span center point

Effective width of T-beam at

compression fibre



$$d' = 3.0 + 1.3 + 1.3 = 5.6$$

$$t = 15$$
 in

(d) Calcul	etion o	+ bendi	ng stress
		(/)		
	(3	<u></u>	1. 0	
M (tm)	-2.87	-14.81		
N (i)	7.0			
S (1)				
b (cm)	5	0	<i>\35</i>	
h (cm)			60	,
d (cm)		2.0	54.4	
d' (cm)	-	B. 0	5.6	
4- (1)		25-5	025-5	
As (cm²)		25-34	= 25.34	
p		00975	0.00345	
4	· · · · · · · · · · · · · · · · · · ·		t=15	
As' (cm²)			Td = 0276	•
p [']				
e = M/N (cm)				
e = M/N + u	 : ·			
e=M/N-u	•			
e/h		,		:
d/e				
ď /h				
d'/d				
Ho/bd*(kg/cm²)	2-/2	10.95	2-17	
k				
c				
j				
1/Lc	5	. ઇ	8.03	
1/Ls		9	3/9	
β= σs/σc				and the second s
σc (kg/cm²)		61.4	17.4	
σs (kg/cm²)	250	/300	690	
T (kg/cm²)				
OSA (kg/cm²)	1000	1800	1800	1.00
σca (kg/cm²)		90	90	
Ta (kg/cm²)				
Nonegran	M- I	*	M-47. M-1	
Combination	Ð	D+P	D+P	

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

$$As = \frac{M}{0sa \cdot j \cdot d} \times \frac{4}{3}$$

$$= \frac{14.81 \times 10^{5}}{1.800 \times 0.875 \times 52.0} \times \frac{4}{3} = 24.11^{cm^{2}}$$

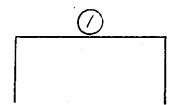
Hence D25-5 = 25.34 > 24.11 cm2

(ii) At the span center point

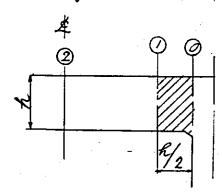
$$As = \frac{15.73 \times 10^5}{1800 \times 0.875 \times 54.4} \times \frac{4}{3} = 24.48^{cm^2}$$

Hence D25-5 = 25.34 > 24.48 cm²

2. Shearing stress of upper beam



(1) Summary of shearing stress

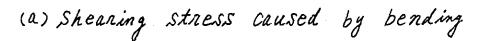


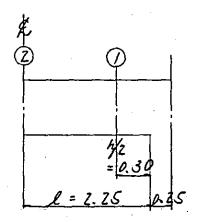
For the design of crass section of beam end, the value of shearing stress at 1/2 point is applied.

		. (\mathcal{D}	
	Left support	co.	Right support	co.
0	7.02		-9.22	
\odot	6.82	10	9.02	9
(1)	4.90	,	2-7.70	
		.		

(Note) CO. - combination

(2) Shearing stress





(i) Shearing stress of the member of uniform height

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

$$T_1 = \frac{9.02 \times 10^3}{50 \times 52.0} = 3.47^{8} / m^2 < 3.9^{8} / m^2$$

$$Tz = \frac{7.10 \times 10^3}{50 \times 52.0} = 2.73$$
 "

Calculations for diagonal Re. bars and stirrups are not required because the concrete area is sufficient to the shearing stress.

- (a) ctc distance of stirrup

 There fore

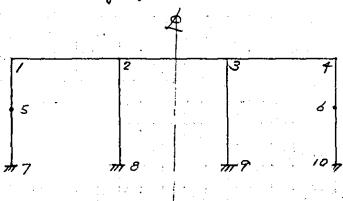
 Annange stirrups D13-1 sets in 25.0 tc
 - (b) Re bars in axial direction

 Side (one side)

 As = 0/6 1 bars

\$7: Calculation of column

1) Rahmen (Rigid Frame) calculation in railway profile



Critical cases No. 2

Member	M (tm)	N (c)	case number
1-5	-14.89	3020	10
7-5	19.55	56.76	9
7 — 9	-16.91	62.50	10
2 — 8	18.54	22.25	10
8-2	19.66	29.04	9
8-2	-/9.37	26.17	10

(Refer the result of computor analysis on stress calculation)

2) O-ORahmen (Rigid trame) calculation in the direction of railway cross section

1		2
3	·	4
75		6 117

Member	M (em)	N (4)	case number
8-4	18.48	71.57	9
6-4	-16.12	42.07	10
2-4	-10.28	36.60	9

Crixical cases No2

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

1					2
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13					4
<i>ח</i> ב	1	•		,	77
3.		Ţ.,		. «	

Member	M (ten)	N (4)	cose
3-/	13.28	17.02	9
4-2	13.73	34.45	9
/-3	12.38	13.08	9

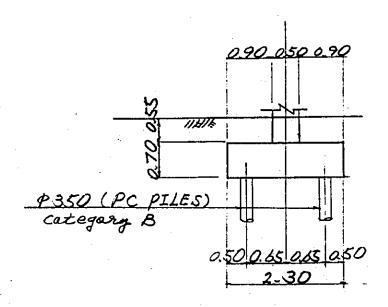
Critical cases No 2

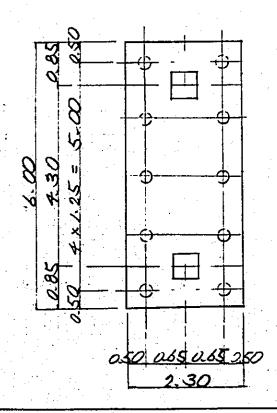
$\overline{}$	Stress	men in	railu	mu pri	otile	•
	1-5	7-5	7-9	2-0	8-2	8-2
M (tm)	- 14.89	19.55	-16.91	1854		
N (i)	30.20	56.76	62.50	22.25	1266 29.0 4	-1937 26.17
S (1)	30.20	30.70	02,00		27.07	20.7
b (cm)		50	<u> </u>		50	
h (cm)		50	· · · · · · · · · · · · · · · · · · ·	<u>, ,,, ,,,,,,,,,,,,,,,,,,,,,,,,,,,,,,,</u>	50	
d (cm)		43.7			4 3,7	
d' (cm)		<u>-75.7</u> 6.3			<u>д, 3</u>	
				·····		
As (cm²)	n	29-5=	12 12	. 0	29-5=	30 10
p	102	0-0.1283			0.0128	
			1	***************************************	0.01200	٧
As' (cm²)						
p [']			,	· · · · · · · · · · · · · · · · · · ·		
= M/N (cm)	49.30	34-44	27.06	<i>63</i> , 33	67.70	74.02
= M/N + u	7-12-3-3					
= M/N - u						
e/h	0.986	0.689	0-541	1.667	1.354	1.480
d/e						
d' /h	0.126	0.126	0.126	0.126	0.126	0.126
d /d						
le/bd*(kg/cm²)	12.08	22.70	25.00	8.90	11.62	10.47
k	2462	0519	0.574	0.412	0.428	2478
c	a199	0.274	a337	0.123	0-149.	0.205
j						
1/Lc		· · · · · · · · · · · · · · · · · · ·				
1/Ls						
$= \sigma s / \sigma c$		•		i .		
C (kg/cm ²)	60.6	82.9	74.1	72.1	77.7	5/./
$S = (kg/cm^3)$	810	850	580	1210	1210	630
(kg/cm²)					<u> </u>	
80 (kg/cm²)	1	1 800		· · · · · · · · · · · · · · · · · · ·	1'800	
ca (kg/cm²)		90			90	
a (kg/cm²)		,				
unber	MN-5.6.7	9	• •	*.	43	>
mbination	D+S	.,	,	5	5	5
				Ž		JICA

		hmen in		Q-Q Rahmen in the direction of railway				
		section		cross section				
	6-4	6-4	2-4	3/	4-2	/ — 3		
M (tm)	18.48	-16-12		13.28	/3.73	12.38		
N (t)	71.57	42.07	l .	17.02				
S (1)		, <u>, , , , , , , , , , , , , , , , , , </u>	30,00	77.5	0,0,0	70,00		
b (cm)		50		50				
h (cm)		50		50				
d (cm)		43.7		43.7				
d' (cm)		6.3		6.3				
As (cm²)			10 12					
p		9 - 5 = 3		D29-5=32.12				
		0.01285		<u> </u>	0.01285			
As' (cm²)								
p'								
e=M/N (cm)	25_82	38.32	28-09	78.03	39-85	94.65		
e = M/N + u		<u> </u>						
e = M/N - u								
e/h	0.5/6	0-766	a562	1.561	2797	1.893		
d/e					,	· .		
d'/h	0.126	0.126	0.126	0.126	0-126	0126		
d'/d								
Ne/bd*(kg/cm²)	28.63	16.83	14.64	681	13.78	5.23		
<u>k</u>	a587	0500	a565	0-417	0.493	2403		
С	a351	0.250	a 327	0./3/	0241	0.110		
j .								
1/Lc								
1/Ls						•**		
β= σs/σc								
σc (kg/cm³)	81.6	67.4	44.8	51.9	57.2	47.7		
σs (kg/cm³)	600	760	370	860	660	840		
τ (kg/cm ²)						· · · · · · · · · · · · · · · · · · ·		
OSG (kg/cm²)	1800			, 800				
σca (kg/cm²)		90		•	90			
Ta (kg/cm²)					· · · · · · · · · · · · · · · · · · ·			
number	MN-5.6.7	• •	•	4,	4	, 5		
combination	D+S	• 1,1	•		5	<u> </u>		
						JICA		

§8. Basic criteria for calculation

- 1. Analysis in the direction of nailway profile
 - 1) Configulation and dimension
 - · End part and Intermediate part





2) Own weight of tooking and weight of earth

End part and Intermediate part (column part)
$$N_{F1} = 2.5 \frac{1}{3} \times 2.30 \times 6.00 \times 0.90 \times \frac{1}{2} = 12.08^{t}$$

$$N_{G} = 1.8^{o} \times (2.30 \times 6.00 - 0.50 \times 0.50) \times 0.55 = (3.41^{o})$$

$$IN = 25.49^{t}$$

3) Horizontel torce of fooding

$$HF = 12.08 \times \frac{0.7}{2.5} \times 0.10 = 0.34^{t}$$

w) Reaction of Stairway

Intermediate

(Dead)
$$N = 28.78 \times 1/2 = 14.39^{t}$$

(Dead + federation) $N = (28.78 + 5.78) \times 1/2 = 17.28^{t}$

5) Horizontal force of helf portion of column and stairway End part

$$H = 2.5 \frac{1}{10.3} \times 0.50 \times 0.50 \times 9.339 \times \frac{1}{2} \times 0.10 = 0.29^{t}$$
Intermediate part

H= 14:39 x 0.10 + 5.78 x 0.21 x 0.10 x /2 + 0.29 =: 1.90 t

- b) Supporting power ofpiles of calculation End part
 - (i) Ordinary case (Dead Load) d = 1.0 case 1 $\Sigma M = 1.98 + 1.20 \times 0.70 = 2.82$ tm $\Sigma N = 89.44 + 25.49 = 114.93$ t $\Sigma H = 1.20$ t

$$P = \frac{114.93}{5} \pm \frac{2.82}{2.113} \times 0.65 = \begin{cases} 23.86 & t < 24 & t \\ 22.12 & t \end{cases}$$

$$\Delta N = \pm \frac{1.20}{2 \times 0.246 \times 2.(13)} \times 0.65 = \pm 0.15^{t}$$

$$P_{\text{max}} = 13.86 + 0.75 = 24.61 t$$

$$P_{\text{min}} = 22.12 - 0.75 = 21.37$$

Ordinary case + eemporary case

(Dead Load + Train load and Impact) L=100 case 12

$$ZM = 1.50 + 1.13 \times 0.90 = 2.29 \text{ tm}$$

$$IN = 106.78 + 25.49 = 132.27 t$$

$$P = \frac{132.27}{5} \pm \frac{2.29}{1.113} \times 0.65 = \begin{cases} 21.15 & t < 36^{t} \\ 25.15 & \end{cases}$$

$$\Delta N = \pm \frac{1.13}{2 \times 0.298 \times 2.113} \times 0.65 = \pm 0.58^{t}$$

$$P_{\text{max}} = 27.15 + 0.58 = 27.73 t$$

- Earthquak case (Dead load + Science load) 4-1.5 case 21

$$P = \frac{110.63}{.5} = \frac{34.36}{2.113} \times 0.65 = \begin{cases} 32.70 & \text{t} < 48 & \text{t} \\ 11.56 & \text{t} \end{cases}$$

Horizontal force by one pile

Intermediate part

$$P = \frac{81.29}{5} \pm \frac{0.27}{2.113} \times 0.65 = \begin{cases} 16.34^{t} < 24^{t} \\ 16.18^{t} \end{cases}$$

Ordinary case + Temporary case (Dead Load

$$ZN = 55.39 + 25.49 + 17.28 = 98.16.0$$

$$P = \frac{98.16}{5} \pm \frac{0.53}{2.113} \times 0.65 = \begin{cases} 19.19.t < 36t \\ 19.47 \end{cases}$$

$$P_{\text{max}} = 19.19 + 0.01 = 19.86 t$$

· Earthquake case (Pead load + Seisnic load) 2=1.5 case 9

$$ZM = (9.66 \times 1.50 + (4.15 \times 1.50 + 1.90) \times 0.70 + 0.34 \times 0.36 = 35.30 \text{ tm}$$

$$ZN = 29.04 \times 1.50 + 25.49 + 14.39 = 83.44 \text{ t}$$

$$ZH = 4.15 \times 1.50 + 0.34 + 1.90 = 8.47.$$

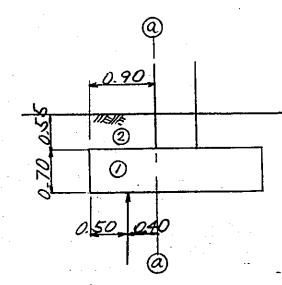
$$P = \frac{63.44}{5} \pm \frac{35.30}{2.1(3)} \times 0.55 = \begin{cases} 27.55 & t < 48 & t \\ 5.83 & t \end{cases}$$

$$Pmax = 27.55 + 4.37 = 31.92$$
 t
 $Pmin = 5.83 - 4.37 = 1.46$ t

Harizoneal torce by one pile
$$H = \frac{8.47}{5} = 1.69 \text{ t/pile}$$

7) Bending moment calculation Analysis is made calculated bending moment at the calumn front.

Bending moment caused by own weight of footing



 $N_2 = 1.8^{\circ} \times 0.90 \times 0.55 \times 3.00$ = 2.87^t

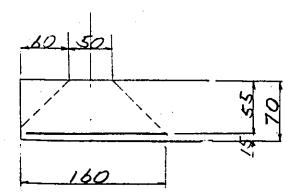
Ma= (4.73+2.67) x 0.45 = 3.33 2m

EMa=P.n.X - Ma

	·					*	(tm)
		×	Reaction of Pile	Number of piles	arm Leverage	p.n.x	-Ma	ΣΜα
End part	Ordinary		24.61		0.40		-3,33	21.26
	Ordinary + Temporary	2	27.73	4	,	27.73		2440
	Earthquake		36.50	*	4,	36.50	•	33.17 (22.11)
Inter- mediate Part	Ordinary	1.00	1638	,9	• · · · · · · · · · · · · · · · · · · ·	16.38	5	13.05
	Ordinary + Temporary	100	19.86	49	5	19.86	5	16.53
	Earthquake	ĺ			5	31.92	4	28.59

0) Stress calculation

(i) End part

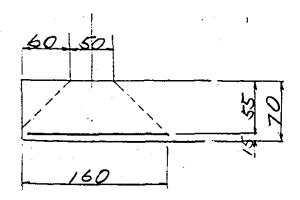


Boeton

$$V_{c} = \frac{24.40 \times 10^{5}}{160 \times 55^{2}} \times 8.01 = 40.4^{\frac{kg}{lon^{2}}} \angle 90^{\frac{kg}{lon^{2}}}$$

Check, safe against cracking

(ii) Intermediate parts



$$As = 0.16 - 15 \text{ cec} = 1.986 \times \frac{160}{15} = 21.18$$

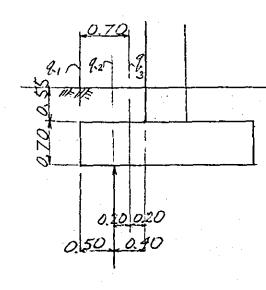
$$P = \frac{21.18}{160 \times 55} = 0.00241$$

From the Nomogram M-1, 1/Lc= 9.21 1/Ls = 451

$$\hat{V}_{c} = \frac{19.06 \times 10^{5}}{160 \times 55^{2}} \times 9.21 = 363^{\frac{100^{2}}{100^{2}}} \left(90^{\frac{100^{2}}{100^{2}}} \right)$$

Check, safe against cracking $M = 13.05^{tm}$

9) Calculation of shearing tore caused by beading



Imposed load
$$A_1 = 92 = 93 = 2.5 \times 0.70$$

$$+ 1.8 \times 0.55 = 2.74 \times 0.55$$

$$Y_0 = \frac{2}{a_0/d} = \frac{2 \times d}{a_0}$$

$$a_0 = \frac{h}{2} = \frac{0.70}{2} = 0.35^m$$

$$d = 0.70 - 0.15 = 0.55^m$$

$$f_0 = \frac{2 \times 0.55}{0.35} = 3.143 < 4$$

$$Q_3' = \frac{Q_3}{r_0} = \frac{2.74}{3.143} = 0.87$$

$$w = 2.74^{t_m} \times 0.50 + (2.74 + 0.87) \times 1/2 \times 0.20$$
$$= 1.73^{t_m}$$

Considered the full with

$$P = 27.73 \times 2.50 = 69.33^t$$

$$f_0 = \frac{2}{a/d} = \frac{2 \times 0.55}{0.40} = 2.750 < 4$$

$$P' = \frac{P}{t} = \frac{69.33}{2.75} = 25.21^{t}$$

$$7 = \frac{20.02 \times 10^{3}}{300 \times 55} = 1.22^{\frac{k_{g}^{2}}{100}} \left(3.9^{\frac{k_{g}^{2}}{100}} \right)$$

2. Analysis in the direction of railway crows section 1) Stress at the bottom of column 12

(i) End part

Ordinary case (Dead load) &= 1.00 cose 1

$$M_{5} = -1.80^{44}$$

$$M_{5} = 93.22^{4}$$

$$H_{5} = -1.04^{4}$$

$$\begin{cases}
M_6 = 1.77^{4m} \\
N_6 = 85.23^{4}
\end{cases}$$

$$H_6 = 1.04^{4}$$

Ordinary use + Temporary case (Pead load + Pedestrian

$$M_{\delta} = 2.08^{tm}$$

$$N_{\delta} = 102.94^{t}$$

$$H_{\delta} = 1.21^{t}$$

Earthquake case (Dead load + Seismic load)

d=1.50 : case 9

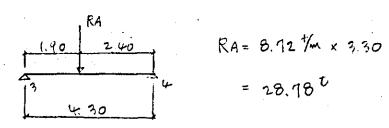
Horizontal force of half partion

of calumn $H=2.5\times0.50\times0.159\times1/2\times0.10=0.16^{\circ}$

(11) Indermediate part



a Reaction of Stairway (Dead lood)



$$N_3 = 28.18 \times \frac{2.40}{4.30} = 16.06^{t}$$

 $N_4 = 28.18 - 16.06 = 12.12^{t}$

b. Reaction of skairway (Pedestrian load)

$$RA = 1.75 \text{ /m} \times 3.30 = 5.78 \text{ t}$$

$$N_3 = 5.78 \times \frac{2.40}{4.30} = 3.23 \text{ t}$$

$$N_4 = 5.78 - 3.23 = 2.55 \text{ t}$$

C. Horizontal force of half partion of column and stairway H3 = $(16.06 + 3.23 \times \frac{0.21}{0.35}, + 2.5 \% \times 0.50 \times 0.50$

$$H_{\Psi} = \left(12.12 + 2.55 \times \frac{0.21}{0.35} + 2.5 \times 0.50 \times 0.50\right)$$

$$\times 9.385 \times \frac{1}{2} \times 0.10$$

$$= 1.72^{-1}$$

Ordinary case (Pead Lood)
$$J = 1.00$$
 case 1

$$M_3 = -0.24^{\text{tm}}$$

$$N_3 = 34.53 + 16.06 = 50.59^{\text{t}}$$

$$H_3 = -0.11^{\text{t}}$$

$$\begin{cases} M4 = 0.44^{tm} \\ N4 = 42.67 + 12.72 = 55.39^{t} \\ H4 = 0.11^{t} \end{cases}$$

Ordinary case + Temporary case (Pead Load + Pedestrion

$$\begin{cases} M_3 = -0.42^{-em} \\ N_3 = 41.76 + 16.06 + 3.23 = 61.05^{-e} \\ H_3 = -0.19^{-e} \end{cases}$$

$$M_4 = 0.77^{em}$$

$$N_4 = 57.06 + 12.72 + 2.55 = 72.33^{e}$$

$$H_4 = 0.19^{t}$$

Earthquake case (Dead load + Seismic load)

d=1.50 case 9

M3= 13.28 × 1.50 = 19.92 to

N3= 17.02 × 150 + 16.06 = 41.59t

 $H3 = 2.73 \times 1.50 + 2.09 = 6.19^{t}$

M4= 13.73 x 1.50 = 20.60 tm

N4= 34.45 × 1.50 + 12-72 = 64.40t

 $H4 = 2.88 \times 1.50 + 1.72 = 6.04^{t}$

2) Scability calculation

		Allowable Supporting Pawer	Case	Pmax	Pmin	Hmax
	Ordinary	24 €	1	22.95	20,20	0
Eand Part	Ordinary + Temperary	36 t	6		23.87	ļ i
	Earthquake	48t	9	₹3.10	10.06	2-06
	Ordinary	24 t.	/	15.17	13.49	0
Intermediat Park	Ordinary + Temporary	36 t	6	19.04	15,10	0
	Earchquake	48t	9	22,20	6.46	1.29

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

In direction of railway profile

$$H = 1.89^{t}$$
 $Nmap = 31.92^{t}$ (oN considered)

 $Nmin = 1.46^{t}$ (
 $M = \frac{H}{2B} = \frac{1.69}{2 \times 0.29.6} = 2.84^{t}$

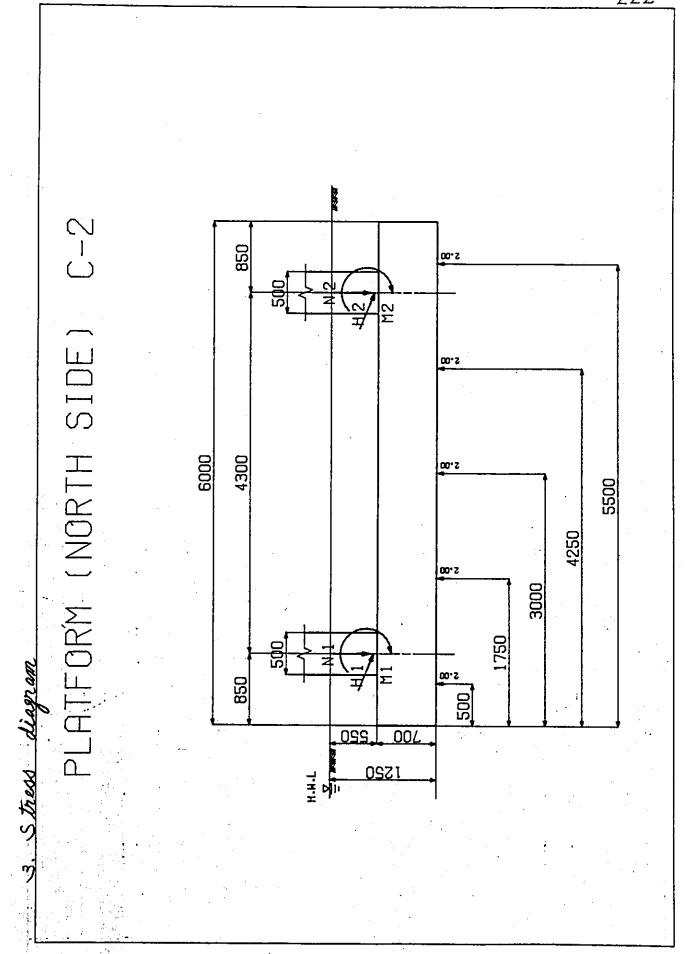
In direction of railway cross section

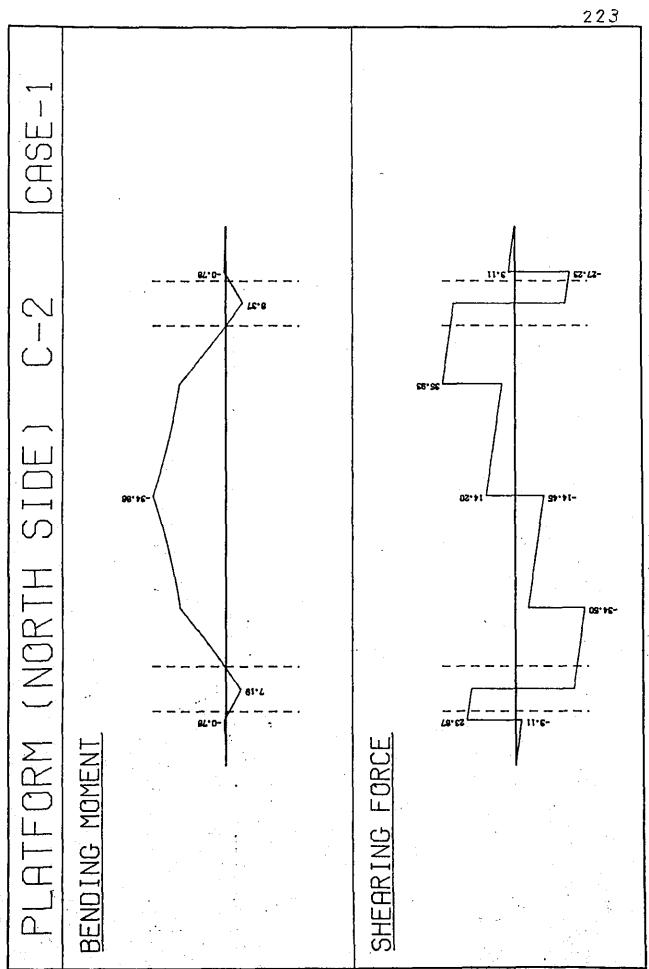
 $H = 2.06^{t}$
 $Nmap = 35.86^{t}$ (sN cosidered)

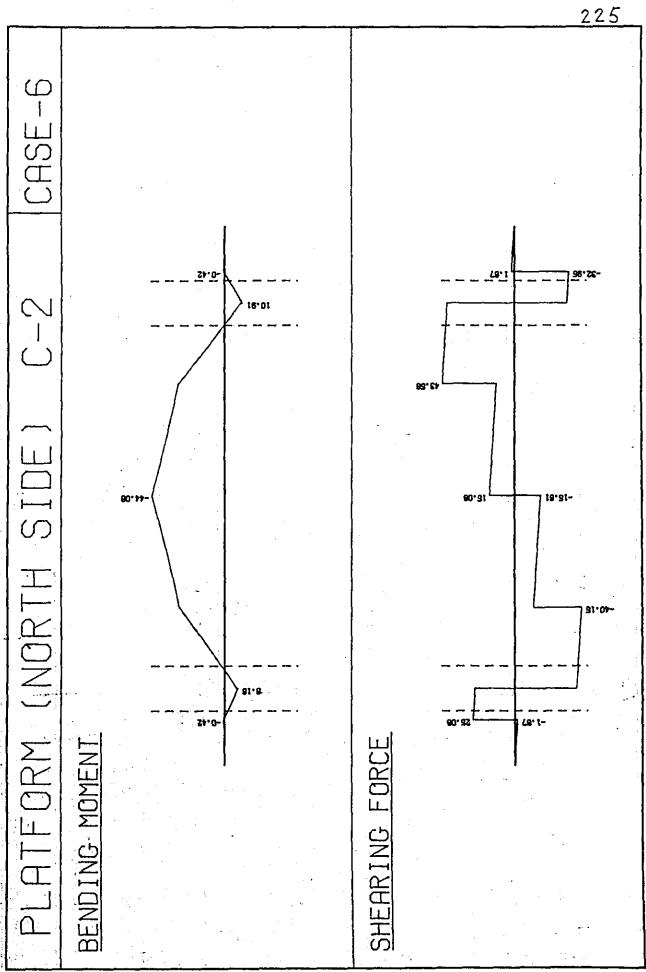
Nmin= 7.30 ()

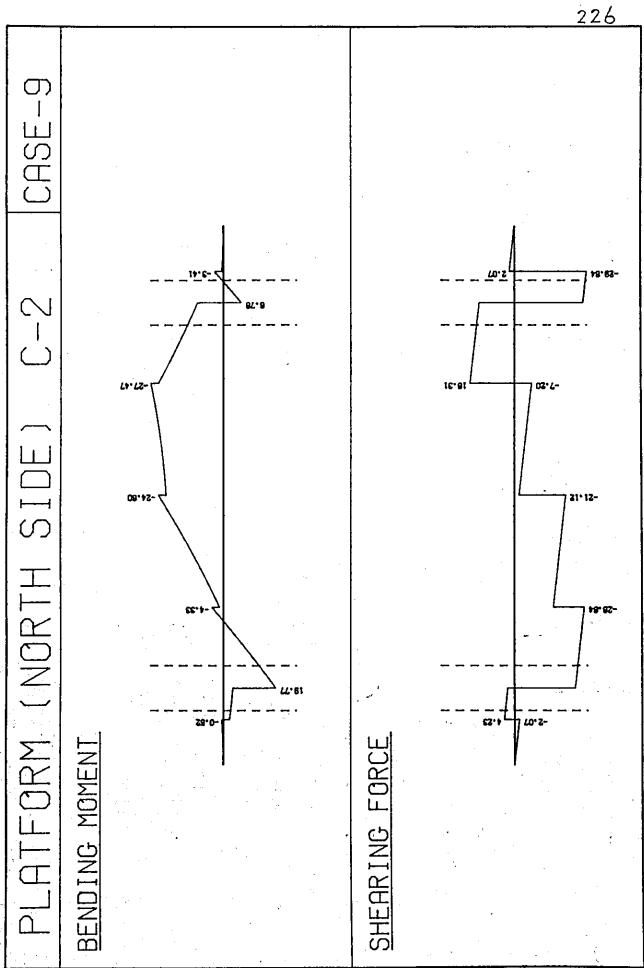
 $M = \frac{H}{2B} = \frac{2.06}{2 \times 0.298} = 3.46^{em}$ According to the "Interaction curve",

Kind B is employed.









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MIXEM INSMIM	3 .	0.000	-0.418	2.373	2.775	6.324	19.768	13.411	520-5	-4.331		-24.602		-27.456	-24.575	-14.667		10.177	11.080	2.061	-0-416	-0.416	000.0
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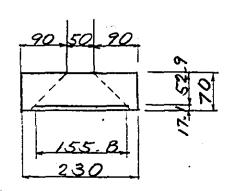
PLATFORM INDRTH, SIDEN, C'-2

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1) Stress calculation

a) End park

i) Botton side



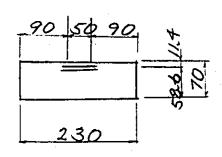
$$A_{s} = D22 - 15^{cm} cec$$

$$= 3.87/ \times \frac{155.8}{15} = 40.21^{cm^{3}}$$

$$P = \frac{40.21}{155.8 \times 52.9}$$
$$= 0.00488$$

From the Nomogram M-1, 1/Lc = 7.08 1/Ls = 230

ii) Tep side



$$P = \frac{79.42}{230 \times 526}$$

From the Nomagram M-1

$$\hat{V}_{C} = \frac{71.85 \times 10^{5}}{230 \times 58.6^{2}} \times 6.61 = 60.1^{\frac{29}{100^{2}}} \left(90^{\frac{29}{100^{2}}} \right)$$

Check, safe against cracking M = - 605/cm

0 0.55 1.25 0 0 0.55 1.25 0 0 0 0.55 0 0 0 0 0 0 0 0 0 0 0 0 0 0	E=== 4. SIDE) C=1 0 0.55 6.0 0 0 0.50 0 0 0 0.50 0 0 0 0.50 0.5 0.0 06213 3500000 1.77 1.00 6.5 0.0 06213 3500000 1.77 1.00 6.5 0.0 06213 3500000 1.77 1.00 6.5 0.0 06213 3500000 1.77 1.00 6.5 0.0 06213 3500000 1.77 1.00 6.5 0.0 06213 3500000 1.77 1.00 6.5 0.0 06213 3500000 1.77 1.00 6.5 0.0 06213 3500000 1.77 1.00 6.5 0.0 06213 3500000 1.77 1.00 1.77 1.00 1.70 1.00 1.
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	• SWITCH	•				
	8) KISO	(V E)		G) FURYDKU # H) E-ANTEIZ=	o	
	C) DELTA-M = D) KANSAN = F) F-BNTF13	r1 er e		1) SUPPORT = J) E-DANHEN= K) SHISHI =	□ → €	
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	SUIHEI SHINDO =	0.028		TANIJYURYO (CONC) =	2.508	(1/H**
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	•K150		.:			
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######################################	. B	0.028				TANIJYUR (CONC)	0#9	2.500	_	Š	<u>*</u>	
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8	SIDE
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*** PICKUP - 1 ***

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;	0.500	11 3	7277	-3.110 (1)	-0.777	-3.110
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4	0.600	-	3.471	42-175 (1)	3.471	42-175
*	0.850	=	13.621		13.821	40.620
•	0.950	7	11.293		11.293	-52.600
	1.109	7	-2.052		-2, 152	-54.155
1	1.450	7	-21.387		-21.387	-56.332
	1.750	=	-36-566	-58-197 (1)	-38.566	-58.197
	1.750	-	-36-566	-13.567 (1)	e i	
	3.000	-	ė		ď	-21.442
	3.000	î	-60.50	21-712 (1)		21.712
	4.250	7	ů		÷	
	4.250	=	ċ	55.714 (1)	œ.	55.714
	4.550	=	÷	53.646 (1)	-21.794	53. 648
	4.900		-3.326	51.671 (1)	-3.328	
*	5.150	=	6.395	56-117 (1)	9, 395	50.117
*	5, 150	=	11,893	-35,113 (1)	11.693	
	5.400	11	2.920	-36-668 (1)	2.920	-36,668
•	5.500	1)	-0-777	-37.250 (1)	0.777	-37.250
	5.500	7 J	-6.777	3.110 (1)	0.777	3-110
:	6.000	≘	000.00	0.000 (1)	0.000	0.000
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1.750		_	£.56	58-19	••	2	4	8.19
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	_	_	<u>.</u> 95	6.66	-	7	2.920	6-56
	_	_	.77	5.39		໘	-0.777	7.29
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PLATFORM (NORTH SIDE) C-1

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16. 16. 17. 16. 16. 12. 16. 16. 16. 16. 16. 16. 16. 16. 16. 16		3.650	_	9	_	å	(6)	6,510	
.100 (9) 16.340 -42.193 (6) -3.096 -64.25 .450 (9) -1.319 -4.3644 (5) -25.793 -65.42 .450 (9) -7.359 -25.688 (6) -45.661 -17.45 .001 (9) -42.645 -25.681 (6) -45.661 -17.25 .001 (9) -43.245 -2.050 (6) -45.413 -23.31 .550 (9) -43.243 -2.050 (6) -45.413 -23.31 .550 (9) -5.23 (6) -45.413 -23.31 .550 (6) -5.23 (6) -45.413 -23.31 .550 (6) -25.30 (6) -45.413 -23.31 .550 (6) -25.30 (6) -45.413 -45.52 .550 (6) -27.243 -27.25 (9) -45.413 -27.25 .550 (6) -27.243 -27.25 (9) -45.413 -27.25 .550 (6) -27.243 -47.25 (6) -45.413 -44.25 .550 (6) -12.95 (9) -12.97 -45.32 .550 (6) -12.416 -42.62 (9) -12.97 -45.32 .500 (6) -0.00 0.00 0.00 0.00 0.00	_	0-850	J	6	•	-41,157	(9	12,862	'n
.450 (9)		1.100	J	ġ,		-42,193	(9)	-3,098	÷
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.000 (9) -38.043 -2.050 (6) -71.653 23.31 250 (9) -43.245 -7.233 (6) -45.413 17.26	~,	3. 30 n	J	6	-42.645	ċ	9	-71,853	3.11
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.900 (6) -3.777		1.550	·	5	-25.870	3	6	÷	3. E1
-150 (6) 11.536 60.460 (9) -12.971 27.32 -44.26 400 (6) 14.465 -42.450 (9) 10.633 -44.26 450 (6) 3.428 -45.32 45.428 (9) -5.521 -45.32 45.428 (6) -0.568 -45.32 45.428 (6) -0.568 -45.321 -45.	-3"	006 -	_	3	-3.777	å	(6)	-19,932	6.36
-150 (6) 14.465 -42.460 (9) 10.633 -44.26 -400 (6) 3.628 -42.624 (9) -0.568 -45.32 -500 (6) -0.418 -42.624 (9) -0.5121 -45.73 -500 (6) -0.777 (6) -0.00	۳,	5-150	·	9	11.536	6	(6 -	-12.971	7.32
.400 (6) 3.c26 -42.624 (9) -0.568 -45.32 .500 (6) -0.416 -42.624 (9) -5.121 -45.73 .500 (6) -0.416 1.672 (6) -0.777 3.11 .000 (9) 0.000 0.000 (6) 0.000 0.00		5.150	J	9	14.465	3	66	10.633	4.28
.500 (6) -0.410 -42.624 (9) -5.121 -45.73 .500 (6) -2.416 1.672 (6) -0.777 3.11 .000 (9) 2.000 0.000 (6) 0.000 0.00	•		`_	9	3.628		(6)	- 0.558	5.32
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PLATFORM (NORTH SIDE)

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	0.500	-	3	-0.777	46.902	۰	5	3000	7.658
	0.600		3		48.260	_	6	4.829	7.2.3
*	0.850	-	G	15.957	16.726	_	6	5,510	6.207
	0.650	_	6		-41-157	J	5	12.852	-63, 421
	1.100	-	ē		-42.193	÷	3	-3.098	-64.257
	1.450	-	6		-43.644	_	3	-25.538	-56.146
	1.750	-	6	•	-44.868	_	3	-45.661	-68.012
٠.	1.750	_	ç	-45.661	-17.166	J	6	-7.359	-25.638
•	3.000	-	9	-71.653	-23.115	·	6	-42.645	-30.820
	3.000	_	9	-71.653	25.039	_	6	-38.043	-2, 050
	4 , 25 0	-	6	-45.324	19.133	·	6	-43.645	-7.233
	4,250	-	3	-45.413	66.078	_	6	-39.243	31,057
	4.550	_	3	-25.870	54.212	·	6	-30.112	29.613
. '	4.50B	-	9	-4-107	62.322	J	ĉ	-19.932	28.362
*	5-150	-	3	11.369	61.466	_	6	-12.971	27.325
*	5,150	<u>-</u>	\$	14.296	-41.454	_	6	10.633	-44.285
. ,	5.400	_	3	3.62€	162.27	_	6	-0.568	-45.322
	5.500	<u> </u>	3	-0.418	-42.624	<u> </u>	<u>6</u> .	-5-121	-45.736
	5.500	_	3	-0.777	3.110	-	<u>.</u>	-0.416	1.672
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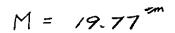
22,953 26,006 24,281

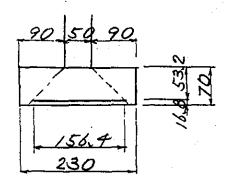
PLATFORM (NOPTH SIDE); C-1

CASE V HAX

1) Intermediate part

i) Boston side





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

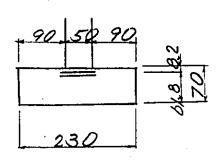
$$= 2.865 \times \frac{1564}{15} = 29.87^{cm^{2}}$$

$$P = \frac{29.87}{156.4 \times 53.2} = 0.00359$$

From the Nomogram M-1, 1/Lc = 7-91 1/Ls = 307

$$\hat{V}_{C} = \frac{19.77 \times 10^{5}}{1564 \times 53.2^{2}} \times 7.91 = 35.3^{\frac{19}{100}} \left\langle 90^{\frac{19}{100}} \right\rangle$$

ii) Tep side



From the Nomagram M-1

$$\hat{V}_{c} = \frac{44.08 \times 10^{5}}{230 \times 61.8^{2}} \times 7.65 = 38.4^{\frac{kg}{lon^{2}}} \langle 90^{\frac{kg}{lon^{2}}} \langle 90^{\frac{kg}{lon^{2}}} \rangle$$

Check, safe against cracking M= -34.86

2) Shearing stress 1/2 paint from column front

- § 9. Calculation of shoes and beam supporting pants
 (1) Calculation of shoes
 - 1. Load Calculation
 - (1) Dead load

 Refer Reaction of Stairway A

 R = 63.80 t

Reaction per one sho

Rd = 63.80 x /4 = 15.95*

- (2) pedestrian load

 R = 10.78^t

 Reaction per one sho

 Rp = 10.78 × 1/4 = 2.62^t
- (3) Dead load + pedestrian load ER = 15.95 + 2.62 = 18.57 t

- 2. Calculation of rubber shoes
 - (1) Required ares for supporting load

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

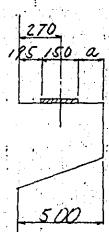
Hence,

$$\frac{R \max}{80} = \frac{18.57 \times 10^3}{80} = 232^{\text{cm}}$$

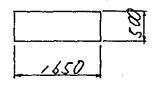
$$\frac{R \min}{15} = \frac{15.95 \times 10^3}{15} = 1063^{\frac{1}{15}}$$

Assumed the size of nubben shoe as

Width of beam supporting part



- (2) Relative displacement between beam and Substructure
 - (a) Displacement of Beam Coused by the defletion of Beam: Ald
 - i) Calculation of Beam deflection



I = 1/2 × 1.65 × 0.503 = 0.0/7/9 mg

· Magnitude of ventical deflection at the beam Center point

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

$$I = 0.01719 \text{ m}^{4}$$

(1) Deflection caused by dead load

Uniformly distributed load (from reacting

calculation of T Section simple beam)

.. Rd = 63.80 t

Wd = 63.80 × 1/2 × 1/9.98 = 3.20 /m

$$\delta b = \frac{5 \, \text{wl}^{\, f}}{384 \, \text{EI}} = \frac{5 \times 3.20 \times 9.48^{\, f}}{384 \times 2.7 \times 10^{\, 6} \times 0.01719}$$
$$= 0.00725^{\, m}$$

Deflection caused by pedestrian load $R = 10.98^{t}$ $\omega = 10.98 \times 1/2 \times 1/9.98$ = 0.53 %

$$Se = \frac{5 \text{ sl}^4}{387 \text{ EI}} = \frac{5 \times 3.2 \times 9.48^4}{387 \times 2.7 \times 10^6 \times 0.01719}$$
$$= 0.00120^{m}$$

ii) Displacement of beam Caused by bending deflection: $\Delta l\alpha$

(1) pead load

ala = h & (Semi - fixed)

h = Distance from beam bottom to neutral axis

h = 0.50 × 1/2 = 0.25 m

L = Deflection angle of beam at the Support point (Radian)

> S = 0.00725" (from Calculation of T Section Simple beam)

- (11) pedestrian load $S = 0.00/20^{m}$ $\Delta = \frac{3.2 \times 0.00/20}{9.78} = 0.0004$ $\Delta l = 25.0 \times 0.0009 = 0.01^{m}$
- (b) Displacement of beam Caused by temperature

 Change: Dlt

 Dlt = Dt · d · l

st: Temperature change ± 20°C

d: Coefficient of linear expansion

of beam 1 × 10-5/°C

l: span l= 9.98 x 1/2 = 4.74 m

Hence

Alt = ± 20 × 10 -5 × 4.79 ± 0.0009 = ± 0.009 =

(C) Displacement of beam Caused by daying Contraction: $\triangle ls$

Als = Ecs. l

Ecs: Ratio of daying Contraction of Concrete
20 × 10-5

Hence

DLS = 20 × 10-5 × 4.74 = 0.009 = 0.09 =

- (d) Displacement in honizontal direction in case of earthquake
 - · Displacement caused by horizontal force in case of earthquake

$$\Delta e_1 = \frac{H \cdot t}{G \cdot Ac} = \frac{1.91 \times 10^3 \times 0.8}{8.0 \times 950} = 0.92$$

Relative displacement between beam and substructure

- (e) Required thickness
 - 1) Normal case.

$$\Delta m = \Delta l x + \Delta l x + \Delta l s + \Delta l c$$

$$= 0.06 - 0.09 - 0.09 = -0.12$$

$$\Sigma tel = \frac{\Delta m}{0.7} = \frac{0.12}{0.7} = 0.17$$

- N) Earthquake case $\Delta E = \Delta l \Delta + \Delta l \Delta + \Delta l \Delta + (\Delta e_1 + \Delta e_2)$ $= 0.06 0.09 0.09 (0.42 + 0.5) = -1.09^{cm}$ $Exe3 = \frac{\Delta E}{2.0} = \frac{1.09}{2.0} = 0.52^{cm}$ Therefore, Use $Te = 8^{mm}$ of one layer.

- (3) Restricted tontional strain Connesponding to deflection angle at the support point

 (a) $\sum \Delta te > \frac{a}{2} \tan \alpha$
 - Sate: Average deformation of nubber shoe in ventical direction (on)
 - a: Side length of rubber shoe in direction of bridge axis (cm)
 - d: Angle between beam bottom face and support point

$$\triangle te = Cx \cdot \frac{t}{G} \cdot \frac{te^3}{Ao^2}$$

Ct: Factor determined by the ratio

of both Side lengths

(Refer explanation chant 174-1)

$$\frac{bo}{ao} = \frac{20}{15} = 1.33 \text{ then. } Ct = 1.85$$

f: Bearing Stress of rubber Shoe in ventical direction (18/0m²)

(Dead load)
$$fd = \frac{Rd}{A} = \frac{15.95 \times 10^3}{15 \times 20} = 53.17$$

pedestaian load)
$$f_P = \frac{RP}{A} = \frac{2.62 \times 10^3}{15 \times 20} = 8.73$$

G: Elastic modulus of rubber shoe in terms
of shear (**//m)

subjected dead load G = 6.2 Min Subjected pedestrian load G = 8.0 Min

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

$$\Delta ted = 1.85 \times \frac{53.17}{6.2} \times \frac{0.8^3}{15^2} = 0.036$$
 cm

$$\triangle tep = 1.85 \times \frac{8.73}{8.0} \times \frac{0.8^3}{15^2} = 0.005$$

Hence ste = 0.036 + 0.005 = 0.041 cm

Hence $\frac{a \cdot tand}{2} = \frac{15 \times 0.00299}{2} = 0.018$

(Este = 0.091 om

(b) Maximum deformation in ventical direction

Este max

 $\sum \Delta temax = \sum \Delta te + \alpha \cdot tan \frac{1}{2}$ = 0.041 + 0.018 = 0.059 cm 0.15. \(\Ste = 0.15 \times 0.8 = 0.12 \text{cm} \) \(\Ste max \) (4) Safety analysis in terms of buckling when Subjected vertical load

 $a,b \geq 5 \cdot Ete$ $a = 15^{cm}, b = 20^{cm} > 5 \times 0.8 = 4.0^{cm}$

Analyzes as above, dimensions of nubber shoe are determined as follows.

Bridge axis direction $a = 15^{cm}$ Cross sectional direction $b = 20^{cm}$ Thickness of the layer $t = 0.8^{cm}$ Use one layer.

Gnoss thickness 1.0 cm (Including stainless steel cover plates)

(2) Calculation of stoppen made of steel nod

1. Horizontal seismic load applied for the

Stoppen design

KSh = Df · Kh

of: Extra factor

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

Kh: Honizontal seismic load for design Kh=0.22

Honizontal seismic load applied for the

Stopper design will be,

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

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

one unit stoppen pen one main ginden is equipped, with semi-nigid construction

(a) Seismic fonce due to dead weight

Hsd = Ksh · Rd

= 0.12 × 15.95 = 1.91 +

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

 HSd = 0.19 × 15.95 = 2.23 t

Analysed as above, honizontal fonce

Calculation is made based on the case in

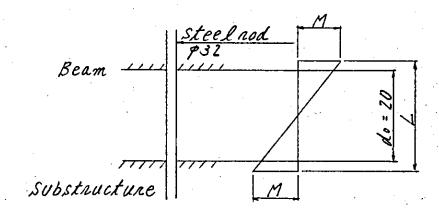
Cross Sectional direction.

- 3. Stress calculation of the Stoppen made of steel nod
 - (1) Shearing stress

 Steel rod $p = 32^{mm}$ (SS41) As = 8.04 cm² $H = 2.23^{\pm}$ $T = \frac{H}{AS}$

$$= \frac{2.23 \times 10^{3}}{8.07} = 280^{10} \times 10^{2} \times 10^{3} \times$$

(2) Bending Stress



$$L = do + \frac{1}{2} \cdot \phi = 20 + 32 \times \frac{1}{2} = 36 \text{ mm}$$

$$H = 2.23^{+}$$

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

$$= \frac{1}{2} \times 2.23 \times 0.036 = 0.09^{+}$$

Section modulus
$$Z = \frac{\pi \cdot \phi^3}{32} = 0.098 \cdot \phi^3$$

$$\hat{l}_{S} = \frac{M}{Z}$$

$$= \frac{0.09 \times 10^{5}}{0.098 \times 3.6^{3}} = 875^{-100} < 1500 \times 1.5$$

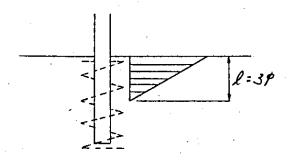
$$= 2250^{-100}$$

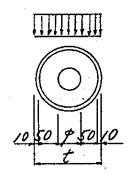
(3) Combined stress

$$\sqrt{\left(\frac{\delta s}{\delta sa}\right)^2 + \left(\frac{\tau}{\tau a}\right)^2} = \sqrt{\left(\frac{875}{2250}\right)^2 + \left(\frac{280}{2255}\right)^2}$$

$$= 0.95 < 1.7$$

t. Calculation of bearing stress of concrete





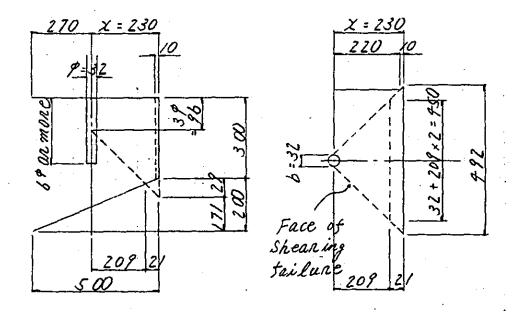
$$t = 3.2 + (5.0 + 1.0) \times 2 = 15.2$$

$$\int C = \frac{2 \times 2.23 \times 10^3}{9.6 \times 15.2} = 30.6 \frac{r_0^{3/2}}{10^{10}} < \int Ca = 240 \times 0.8$$

$$= 192 \frac{r_0^{3/2}}{10^{10}}$$

(3) Calculation of related part of the Stopper installation

1. Shearing stress calculation



Sheaning stress

$$AT = \sqrt{2} \times (2X + 2a + b)$$

I: Shearing stress

AI: Ares of the face of shearing failure

$$AT = \sqrt{2} \times 23.0 \times (2 \times 23.0 + 2 \times 9.6 + 3.2)$$

$$- (99.2 + 95.0) \times \frac{1}{2} \times \sqrt{2} \times 2$$

$$= 2076^{cm^{2}}$$

H = 2.23 t (In direction of railway protile)

$$Z = \frac{2.23 \times 10^3}{2076} = 1.07 \% (3.9 \%)^{\frac{1}{2}}$$

2. Reinforcing ban arrangement around the part of Stoppen installation

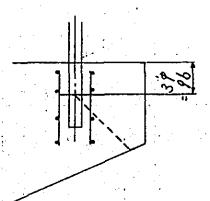
$$As' = \frac{H}{0.5a}$$

$$= \frac{2.23 \times 10^{3}}{1800 \times 1.5} = 0.83^{0.02}$$

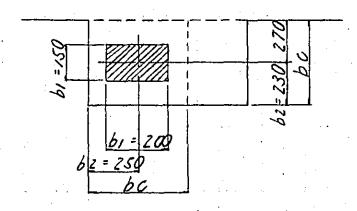
AS = D13 - Z = 2.53 cm² > AS' = 0.83 cm²

Thene fone,

D13-3 bans are annanged within the nange of 34



(†) Calculation of shoe bed pant subjected ventical load



1. Reinforcing bar arrangement for ventical load

$$AS = \frac{1}{4} \left(1 - \frac{bi}{bc} \right) \frac{R}{SA}$$

Where,

As: Sectional area of bars (ont)

(sa: Allowable tensile stress of ban ()

(sa = 1800 5/m (SD30)

R: Vertical fonce acting on shoe bed (18)

R = 15.95 + 2.62 = 18.57 5/L

b1: Acting width of bearing force (cm)

bc: Distributing width of bearing power um)

bc = 2 · b2

(1) In direction of bridge axis
$$b_1 = 15.0 \text{ cm} \qquad b_2 = 23.0 \text{ cm}$$

$$b_3 = 2.52 = 2 \times 23.0 = 46.0 \text{ cm}$$

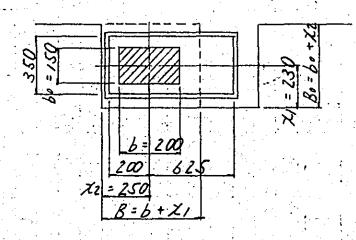
$$As = \frac{1}{4} \times (1 - \frac{15.0}{46.0}) \times \frac{18.57 \times 10^3}{1800} = 1.74 \text{ cm}^2$$

(2) In direction of bridge CROSS Section
$$b_1 = 20.0 \text{ cm} \qquad b_2 = 25.0 \text{ cm}$$

$$b_3 = 2.62 = 2 \times 25.0 = 50.0 \text{ cm}$$

$$AS = \frac{1}{4} \times (1 - \frac{20.0}{50.0}) \times \frac{18.57 \times 10^3}{1800} = 1.55 \text{ cm}$$

2. Ban annangement Range of ban arrangement



B. Bo: Range of ban annangement

(1) In direction of bridge axis
$$B' = b + \chi_1$$

There fore, the range of bar arrangement will be,

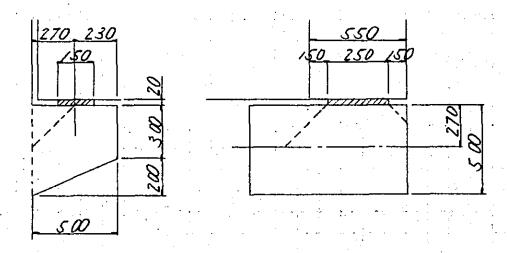
(2) In direction of bridge Cross Section

Therefore, the range of ban arrangement will be,

Use D13 - 150 ttl (n = 3 bans then)

(5) Calculation of beam supporting part

1. Bending Stress calculation



Effective width $B = 0.25 + 0.15 + 0.27 = 0.67^{m}$

- (1) Load calculation
 - (a) Reaction of T section simple beam
 Referred the summary table of shoe
 Calculation and beam reaction,

Wd = 0.5/ 1/m

- (2) Bending moment calculation

 Stress per B = 1.00 m of effective width

 (a) Dead load

 Md = 15.95 × 0.27 + 0.38 × 0.72 × 0.50 × ½

 + 0.13 × 0.72 × 0.50 × ½

 = 9.39 tm
 - (b) pedestrian load

 Mp = 2.62 × 0.27 = 0.71 tm
 - (C) Dead load + pedestrian load EM = 4.39 + 0.71 = 5.10 tm

(3) Allowable stress, safe against cracking

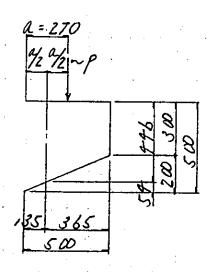
$$Md = 4.39 \text{ tm}$$
 $Mp = 0.71^{\circ}$
 $\Sigma M = 5.10 \text{ tm}$

$$\angle = \frac{MP}{EM} = \frac{0.71}{5.10} = 0.19 < 0.25$$

Hence, determined,

			PAGE 27
		··· — ···	
	,	1	
	ρ	0.40	
	ν	D+P	
M (.tm)	4.39	5.10	
N (t)		<u> </u>	
S (t)			
b (cm)	67	67	
h (cm)	50	50	
d (cm)	44.7	44.7	
d' (cm)	<u>5.3</u>	· 5.3	
As (cm²)	D19-150xL	019-15040 (5) =14.33	
AS (cm-)	= 14.33	= 14.33	
p	0.00478		
As' (cm²)			
p'			
e= M/N (cm)			
(cm) e= M/N+u			
e = M/N - u			
e/h			
d/e			
d' /h			
d'/d	1 1 1 1		<u>'</u>
A / a Ne/bd²(kg/cm²)		- 01	
· .	3.28	3.8/	
k c			
<i>j</i>		,	
1/Lc		7.12	
1/Ls	233	<i>23</i> 3	
β= σs/σc		100 Sept.	
σc (kg/cm²)	**************************************	27./	
$\sigma s = (k_R/cm^2)$	770	890	
T (kg/cm²)			
σsα (kg/cm²)	1200	1800	
oca (kg/cm²)		90	
Ta (kg/cm²)			
number	M-1	• •	
A to a tea	D.	0+P	

2. Calculation of shearing stress



Sheaning stress is calculated at the a/z point

Effective width is assumed as the full width

b = 130 cm

d = 94.7 -5.4 = 39.3 cm

(1) Shearing force

$$\gamma = \frac{2}{9/6} = \frac{2 \times 0.393}{0.27} = 2.91 < 4$$

Therefore, y = 2.90

$$S = \frac{P}{\delta} = \frac{18.57}{2.91} = 6.38^{\pm}$$

ES = 6.38 + 2.5 × 0.365 × 0.40 × 1.65

+ 2.5 × 0.365 × 0.196 × 1/2 × 1.65 = 7.09 t

(2) Shearing stress

(3) Ban annangement for nesisting against diagonal tension

Shearing tonce per unit one meter

$$S = \sum S / Effective \ Width \ B = 1.65^{m}$$
$$= \frac{18.57}{1.65} = 11.25^{\pm}$$

$$As' = \frac{S \cdot \sqrt{Z}}{Csa} \qquad \text{(isa = 1800 }^{kg/2}$$

$$= \frac{11.25 \times 10^{3} \times \sqrt{2}}{1800} = 8.89 \text{ cm}^{2}$$

Use diagonal tension bans D19-150 the (11 bans)

- 3. Extra bans arranged at the side face of beam support
 - Extra bars are arranged at the side face,

 With use of bars of 40% cross section of

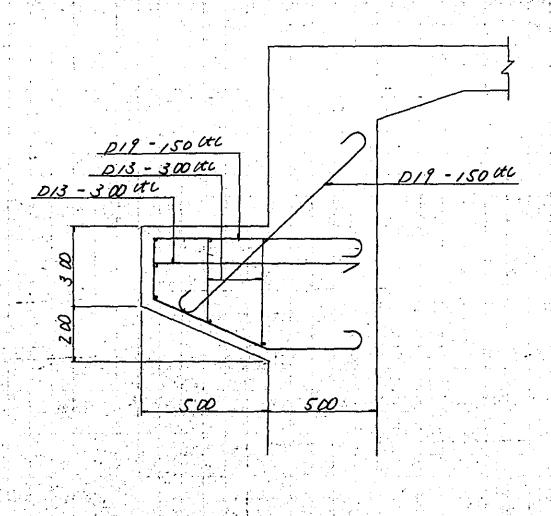
 bending stress and installed in three

 steps formation

Use extra bars of 013-300th (3.33 bars) - in three steps then

AS = 1.267 × 5 × 2 = 12.67 0 > AS' = 12.61 00

4. Ban annangement chant



§ 10. Calculation of stairway

1. Slab Calculation

1) Load Calculation

Pedestrian load

0.5001/-

Step

2.5 /m × 0.33 × 0.165 × 1/2 × 1/2 × 1/2 ×

= 0.206"

pavement 1.1 * x 0.05

= 0.055"

Slab 25" x 0.497

= 1.118"

N' = 1.879 /m

1 = 11.09 (26°5615)



W = W'. COO X2

 $= 1.879 \times (\frac{10.00}{11.18})^2 = 1.503$

2) Bending moment

.. At the support point

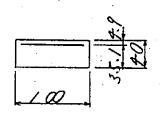
M = - 1/2 × 1.503 × 11.182 = 15.66 tm

· At the Span center point.

M = 1/24 × 1.503 × 11.182

= 7.83 tm

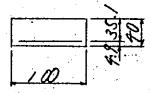
3) Calculation of Bending stress



$$P = \frac{30.97}{100 \times 35.1} = 0.00882$$

Nomogram number M-1

· At the span center point



Nomogram number M-1

Rein foncement at the support of railway

Cross Section shall be 1/6 of that of at the

support of railway profile

AS' = P22 - 12.5 de(8) = 30.97

AS = D16 - 25.000 = 7.99 cm > 5.16 cm

4) Shearing stress

SHZ = 1/2 x 1.879 x 11.18 - 1.879 x 0.20 = 10.13 +

 $T = \frac{10.13 \times 10^3}{100 \times 35.1} = 2.89 \% < 3.9 \%$

No			
INIE			

§§ 10 VIADUCT OF PLATFORM VP5

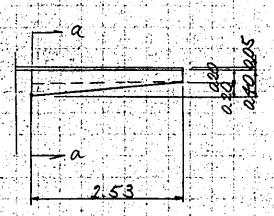
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2. Calculation of Intermediate contilever slab

- 1) load calculations
 - (a) Dead load



Ma-a = 2.5 x 2.53 x 2.20 x 1.265

- + 2.5° x 2.53 x 0.20 x 1/2 x 0.843
- + 1.1 × 0.05 × 2.53 × 1.265 = 2.31 tm

(b) Pedescrian load

Ma-a = 0.50 x 2.53 x 1,265 = 1.60 tm

2) Combined stress

(Dead load + Pedestrians load)

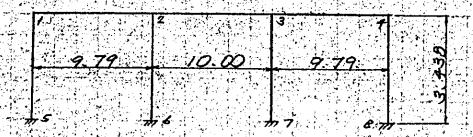
Ma-a= 2.31 + 1.60 = 3912m

	a-as	ection	
M (1m)	2-31	3.91	
N (i)			
S. (i)			
b (cm)	10	0	
h (cm)	4	0	
d (cm)	ક	27	
d' (cm)		3.3	
As (cm²)	D16-	20° CTC	
AS (Cm)	= 9.	93	
p	0.00	27/	
As (cm*)			
Ald Tem?)			
p			
e = M/N (cm)			
e = M/N + u			
e = M/N - u		githeria and a	
e/h			
d/e			
d /h			
d'/d			
He/bd*(kg/cm²)	1-72	2.90	
k			
c			
7			
1/Lc	8	92	
1/Ls	40	<u> </u>	
β= σs/ σc			
σc (kg/cm²)		25.6	
σς (kg/cm²)		1170	
T (kg/cm³)			
osa (kg/cm²)		1800	
oca (kg/cm²)		90	
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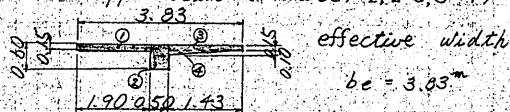
§ 4. Rigid frome analysis on longitudinal direction of elevated structure
(1) Basic calculation





2. Cross Sectional area/Homent of inertia of area of members

(1) Uppen beam (Menber 1-2, 2-3, 3-4)



	b (m)	h (m)	A (m2)	y (m)	A. 7 (m3)
0	1.900	0.150	0.285		0.02138
2	0.500	0.600			0.09000
3	ا الله المنافقة المنا	0.150	**		0.01613
4				1.美玩玩。 (\$ 多)人。	0.01318
文		الرازي والمراقب والمرازي والمرازي والمرازي والمرازي	0.872		014069

	6 (m)	h (m)	A (m')	110(11)	Io (m ⁴)	A. 2/2 (m4)	I.+ A. 7 604
\bigcirc	1.900	0.150	0.285	0.086		0.00211	
2	0.500	0.600	2300	0:139	0 00900	0.00580	0.01480
<u></u>	1.430	0-150	0.215	0.086	0.00040	a0159	0.00199
(A)	1.430	1/2×0.10	0.072	0.022	0.0004	0 0003	0.00007
2.			0.872		0.00997	0.00953	001950

cross Sectional Area A = 0.872 m2

Moment of Inervia I = 0.01950 T

(2) Column (Member 2-6: 3-7)

$$A = 0.50 \times 0.50 = 0.250^{-2}$$

$$0.50$$

$$I = 1/12 \times 0.50 \times 0.50^{3} = 0.00521^{-1}$$

(3) Column (Member 1-5. 4-8)

$$A = 0.50 \times 0.40 = 0.200^{m^2}$$

$$0.50$$

$$I = 1/12 \times 0.50 \times 0.40^3 = 0.00267^{m^2}$$

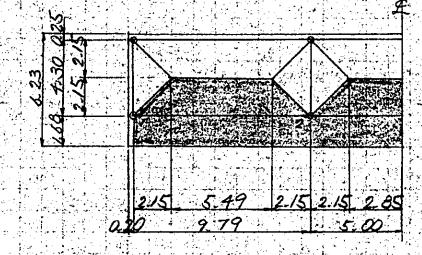
(4) axial height.

h = 3.599 - 0.161 = 3.438 "

(2) Load calculations

1. Dead load

(1) Upper member



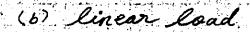
(a) Uniformly distributed load

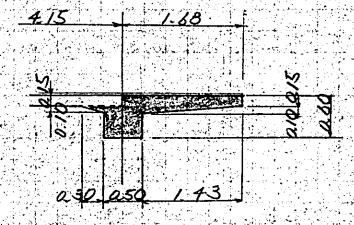
Slab 2.5 the x 0.15 = 0.375 /m2

pavement 1:1 × 0:05 = 0:055

W = 0 43 /me

Wd1 = 0.43 1 x 2-15 = 0.92 5m





pavenent 1.10 2005 x 1.68

= 0.092

Cantilevel 5lab 2.5 x (0.15+0.25) x /2 x 1.43 = 0.715

longitudinal beam 2.5 × (0.50×60-0.25×0.15) = 0.656

Haunch of slab 25 x 0.30 x 0-10 x 1/2 = 0.038

wd2 = 1.50 4m

(0)	Concentrated load acting pane	el peins
i 5	panel paint 1 (4)	
	- Unitormly discributed load	
	0.43 x 2.15 x 2.15 x 1/2	= 0.99
	· Unitormly distributed load	
	0.43 x 0.20 x 2.15	= 0/8
	Uniformly discributed load (0.092 + 0.715) Mx 220	= 0.16
	Haunch of Slab	
	25 to x030 x010 x 1/2 x 1.60	= 0.06
	· Cross beam	
	25 × 0.40 × 0.45 × 2.40	= 1.08
	. Reduction of linear load - (0.656 + 0.038) \$\frac{4}{\times} \times 0.20	= -0.19
	· Reduction of Column part	
	-25 x 0.50 x 0.40 x (0.60-0.1)	61) = -0.22
	· Shed load reaction	= 10.00
); = P4= 12.11

Assets to		
	panel point 2, (3)	
	rentificativi en la compara de la compara de la compara de la compara de la compara de la compara de la compar	
	Uniformly distributed load	
	"我们的"的话,写真是有什么是的话,也是一位"我,还是这个人就是这些的人的,我就是这一样,我们也是	= 1.98 =
	0.43 /m x 2.15 x 2.15 x /2 x 2	
	cross beam	
	母亲的 \$P\$ \$P\$ \$P\$ \$P\$ \$P\$ \$P\$ \$P\$ \$P\$ \$P\$ \$P	
	2.5 1 × 0.50 × 0.45 × 1.90	= 1.07
	· Haunch of Slab	
	2.5° × 0.30 × 0.10 × /2 × 1.60 × 2	= 0.12
	(4) [14] [2] (4) 하는 하는 하는 것이 되었다. 그는 사람들은 사람들은 사람들은 사람들은 사람들은 사람들은 사람들은 사람들은	
	· Compensation for column	
	2.5° × 0.50 × 0.50 × (0.161 - 0.15)	= 0.01"
	. Reduction of linear load	
	- (0.656 + 0.038) = x 0.50	= -0.35
	· Shed load reaction	= 20.00
		P3 = 22.83 t
	Own weight of column	
	그렇게 사용하게 돌아왔는 학생들 학생들이 생각하고 있었다. 학생들은 이 그 등 사람은 것인 사람들은	
	81 = 2.5 1/13 × 0.50 × 0.40 = 0.50 1/2	
	82 = 2.5 " × 0.50 × 0.50 = 0.63"	
つまつなられません	범범병원 배 회장 발전 경기 위에 오늘이 되는 것이 되었다. 그는 사람이 살 수 있는 보이 가는 사람이 되는 사람이 되어 있다.	

	14
2. pedestrian load (people load)	
U = 0.35 % Upper member	
(Refen (2) 1 (1))	
a) Uniformly distributed load wdi=0.35 × 2.15 = 0.75	
b) linear load Wdz = 0.35 x 1.68 = 0.597	
c) Concentrated load acting at	
panel point U) panel point 1 (4)	
	0.81
Uniformly distributed load	
0.35 th × 0.20 × 3.83 =	0.27°
(2) panel point 2.(3)	
Uniformly distributed load 0.35 1/2 × 2.15 × 2.15 × 1/2 × 2. = 1.	art and the state of the state of the state of the state of the state of the state of the state of the state of
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3. Temperature change and Drying contraction

Temperature change ---- ± 10°C

Hence,

Temperature rise ---- +10°C

Temperature drop + Drying contraction

4. Seismic load from dead load (RH=0.10)

(derived from (2) (1) and (2) (2) horizoneal force)

Slab 25 × 0.15 × 4.80 × 29.98 = 53.96 t

Condilever slab 25 x (a15+0.25) x/2 x 1.+3 x 29.98 = 21.44

Haunch of slab 2.5 x0,30 x0.10 x (27.98 x2

 $+3,20\times6) = 2.82^{t}$

Longitudinal bean 25 x050 x 0.45 x 29.98 x2 = 33.73

Cross been 25 x 240x 245 x 3.80 x 2 = 3.42

25 × 0.50 × 0.45 × 3.80 ×2 = 4.28

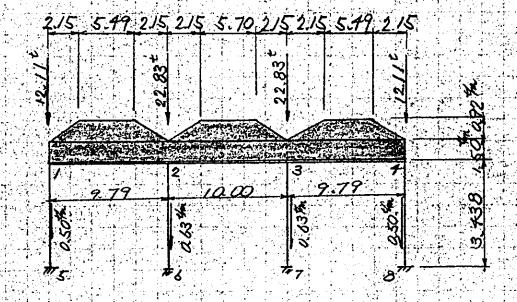
calumn 25 x 050 x 0,40 x (3,438 +0.161

-0.60) x 4 = 2.56

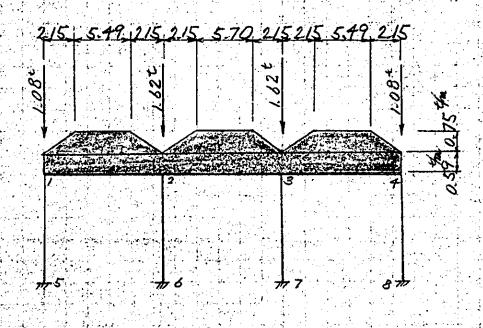
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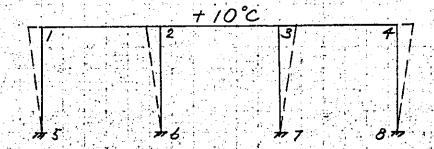
1 case 1 Dead load



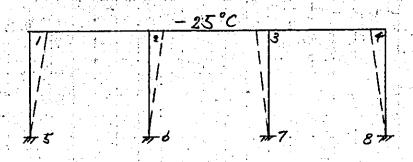
2. case 2 Pedestrian load (People load)



3. case 3. Temperature rise

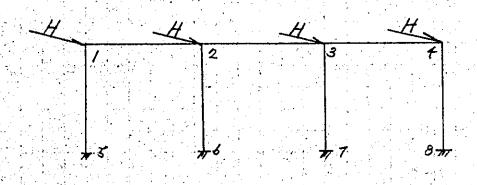


9. case 4 Temperature drop + Drying Contraction



5. cose 5 Seismic load from dead load

$$H = \frac{14.83}{4} = 3.71^{\pm}$$



[9] Combination of loads

Case	Kind of load	Loading Pattern
, , , , , , , , , , , , , , , , , , ,	Dead load	
2	Pedestrian load	
3,	Temperature rise	+10°C
4	Temperature drop + Drying contraction	- 25°C
5	Seismic load from dead load	

2. Combined loads

cose	Combination of loads	2
6	1 + 2	1.0000
7	1 + 3	0.8696
8	1 + 4	
9	1 + 5	a 6667
10		•

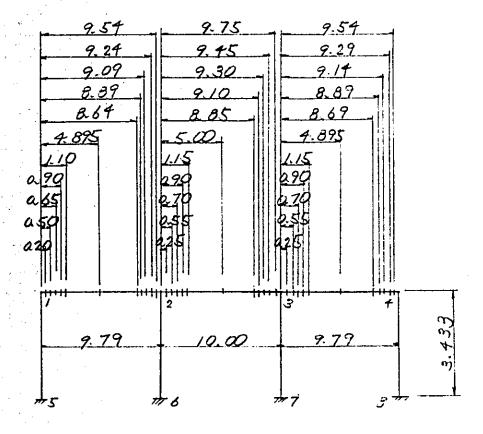
3. Critical cases

No.1 case 1 crack

No.2 case 6 ~ 10 Synthetic

No. 3 case 9.10 footing

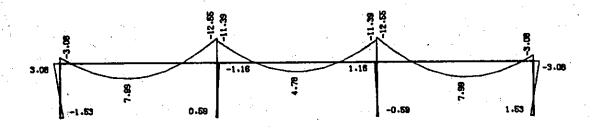
9. Points for scress Calculation



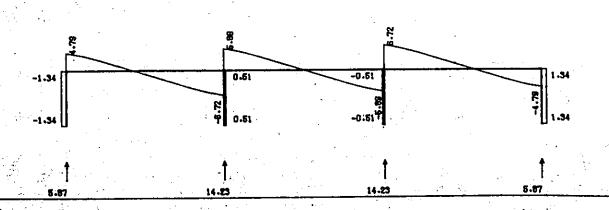
VIADUCT OF PLATFORM (CENTRAL) L-1

CASE 2 (PEDESTRIANS LOAD)

BENDING MOMENT



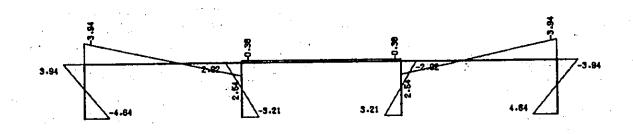
SHEARING FORCE



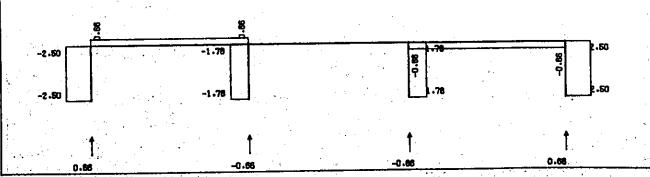
VIADUCT OF PLATFORM (CENTRAL) L-1

CASE 3 (TEMPERATURE)

BENDING MOMENT



SHEARING FORCE



VIADUCT OF PLATFORM (CENTRAL) L-1 CASE 4 (TEMPERATURE + SCHRINKAGE) BENDING MOMENT SHEARING FORCE

VIADUCT OF PLATFORM (CENTRAL) L-1 CASE 5 (SEISMIC LOAD) BENDING MOMENT SHEARING FORCE

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eg 60.3	E	0		5.511	4.976	4.695	-2.976	-17-148	-18.076	-19.343		6.886	6.293	5.870	1	1	'n'n.	17.0		7.866	7.273	6.851	628	-11.681	-12.354	-13.330
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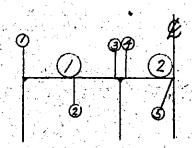
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TITE VIADUCT OF PLAIF DRA GENTRALL 1.3 PICK UP 3. RONE WI HAXTHUN	TITLE C. MIADUT OF PLATFORM CONTRALL L-1 FIGH 10-3	GRC-FANSY	HE MT HINIHUM			H د	-17.998	-13-226	-10.369	9,288	-109	-2-1-52	-3.320	76 4° 9-	9.391	- U	1.090	0	-6.690	30	-3.882	~ .	* • ·					
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TILE VIADUCT OF PLATEORM (CENTRAL) L-1 PICK UP 3	TITLE YIADUCT OF PLATF ORM CCENTRAL) L-1 PICK UP 3				V.	M"·		.550	- 900	1.150	8-690	9.10	9.290	9.790	5.439	9	0000		0.00 n		3.438	BER 7 (0.000					
TILE. VIADUCT OF PLATFORM (CENTRAL) 1-1 PUCK.UP 3 HOMENN HAXTHUM = HEHBER 3 (3 - 4) 6 = =	TITLE, STADUCT OF PLATFORM CCENTRAL) L-1 PICK, UP 3						ITAN	~ 1	7	. •		6	21	JTAN	MAX		LTAN	tt	LTAN	n	JIAN		ITAN	1 1 1 1 1 1 1 1 1 1 1 1 1 1 1 1 1 1 1				
TILEVIADUCT OF PLATFORM (CENTRAL) 1-1 PICK. UP 3.	TILEVIADUCT OF PLATFORM CENTRAL) L-1 PIEK UP 3. HOPERT HAXTHUM						852	652	852	10	2	u ou		(), ● ∞	3				•		- A		, mil					
ITLEVIADUCT OF PLATE PICK UP 3 ITAN	TITLEVIADUCT OF PLATE PICK UP 3 TAN 0.000 (99 290 (10 99 290 (7					7.666	7.273	6.851	1.727	-4-239	-4.801	-4.976	-5.511	649		-3:325		-2.657 3.887		-3.887		026	2000	**************************************			
ITLE. VIADUCT OF PLATE PICK UP 3 ITAN I	TITLEVIADUCT OF PLATF PICK. UP 3		ENT. HAXIHUH	7		۱۱. ب	-12.644	-8-477	-6.004	9.313	4.304	2.268	1.534	-1.090	10.013	 3	6.494	1	3.882	 a	6.690 5.252	= = :0 :		•				
ITLE VIADUCT 1	TITLE . WIADUCT TITAN 0.00	PLATFORM		-6406	C N OF	$\mathbf{i}_{s,j}^{(i)}$	(6)	(6)	- 6 - 0	6 6	07		707	107	(10)		(10)		101	1	(10)			- 5				
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		III.E VI	PICK UP			# 1	ITAN	2	n	ئ ئ	-	• •	10	JIAN	HAX	្រា		!	1 1 1	10	ITAN		ITAN	J. AN				

E-1		- 1																					a. a.	0.6	u 0.	~ ~		~ ~	~ ~		
15.1 15.1 16.1					-2-448 -7-448	-2.448	1107 (-2.448	N	-2.448	$\alpha \alpha$	-2-448		-1.035	-1.035	-1.035	-1-035	-1.035	-1.035	1.035	1.035	*	852	05	- 852	- 85	85		- 85	- 85	
		, ,	-	*	5.511 5.306	4.976	969.4	-1.727	-7.627	-6.195	-8.372	-8.965		6.886	6.293	5.870	5.547	522	-6.914	-7.336	-7.930		7.856	7.273	6.851	6.528	-5.337	-5.595	-6.074	-6.610	
	SUCAD MINIMUM	ENTER STREET		==94	-1.090	1.534	3.430	4.304		-10.359	-13.226	1.	== 9 ()	-11-308	7.680	-5.550	-4.122	5.778	-9.829	-12,324	-16.526	== 9(+	: O P					985	-3.320 -1. A74	0	
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					852	- 852	652	852	- 852	852 852	852	852		-1.035	1.035	1.035	-1.035	-1.035	-1.035	-1.035	-1.035		1.5	ر در	-2.448		17.46	-2.448	Ņ		
		•	0		6.610	6.074	5.595	5.337	-6.528	-6.851	-7.273	-7.866		7.930	7,336	7.160	5.591	u	יושו		-6.627		8-965	8.372	∞ ~	7.627	A 3	-4.496	9.6	-5.511	-
		K MAKINU		= = 9 (-3.320		- 1	- 1	r	1.		1	-16.526	-12.324	-11.237	-8-140	5.778	-5.550	-7.680	-9.618	= = 9 (-17.998	-13.226	-11.984	-8-421	9.313	3.430	1.50 1.50 1.50 1.50 1.50 1.50 1.50 1.50	-1-090	
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5 5	.			1 (0.000	500	900	1-100	8.650	8.890	9.240	9.790	1 2	000.0	550	700	1.150	5.000	9.100	5.5	9.750	1	.:'⊃	550	200	1,150	559	890	9.290	9.540	
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				5/
	-14.651 -15.797 -31.072 -32.516	-31,072 -32,516 -14,651 -15,797		
P	-3.325 -3.325 -3.887 3.887	-3:887 -3:887 -3:387 -3:325 -3:325		
A L	6 9 C = 6 6 9 G = 6 6 6 9 G = 6 6 6 9 G = 6 6 7 C = 6 6 7 C = 6 6 7 C = 6 6 7 C = 6 6 7 C = 6 6 7 C = 6 6 7 C = 6 6 7 C = 6 6 7 C = 6 7 C = 6 6 7 C =	6.693 -6.672 -6.494 -6.493		
-casi	60 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0	66		
	AN BON 3. HEHBER	ITAN 3.638 = MEMBER 7 (ITAN 0.000 JTAN 3.438		
2	-13.552 -14.698 -11.017 -12.461	-31.017 -32.461 -13.552 -14.698		
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CCENTR	5,) C = = 1.090 1.178 6) C = = 3.882 7) C = = 7.752	-3.002 -3.002 -1.090		
OF PLATI	2 - 10 - 10 - 10 - 10 - 10 - 10 - 10 - 1			
	AN O O AN O O AN O O O O O O O O O O O O	s Piloti Hillian		

(6) Calculation of upper beam



- (1) Calculation of Compressive stress caused by bending
 - (a) Summary of stresses
 - (i) At the support point

(Note) Co. — Combination

(ii) span moment

		\bigcirc		2		
100		2	co.	, G	co.	
combined stress	Bottom	2/2/		13.45	6	
] 			

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

$$\lambda = \frac{3.08}{8.77} = 0.35 > 0.25$$

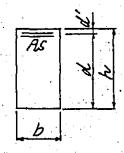
There fore (sa = 100 %)

(ii) At the support point 2,3

$$d = \frac{12.55}{35.53} = 0.35 > 0.25$$

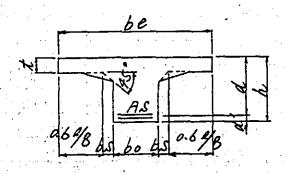
There fore (sa = 1000 Min

- (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



		(/)	ng stre	·	
	0		. ()	②	
M (.tm)	-5.69	- 8.77	-22.98	-35-53	21.61	
$N_{i} = \langle i \rangle$						
S (ii)						
b (cm)	50	0	50	50	217	
h (cm)	60	2	60	60	60	
d (cm)	54	1.9	51.7	51.7	50.9	
d (cm)		5.1	23	8.3 (9.1)	9.1	
As. (cm²)		Y	032-5	D32 - 5	D32-5	
73. (1.17)	032-3	= 39.7/	= 71.48	= 71.48	= 39.71	
p	0-01	447	0.02765	0.02765	0.00360	
As' (cm²)				032-3	t= 15	
73				= 23.83	F=0.295	
p				0.00922		
= M/N (cm)						
= M/N + u						
= M/N - u		•				
e/h					· .	
d/e					-	
ď/h		<u>.</u>				
ď/d						
Ve/bd*(kg/cm²)	3.78	5.82	17.19	26.59	3.84	
<u>k</u>				a535	<u> </u>	
± Lc				0.296	<u> </u>	
j				0822		
1/Lc	4	,	4.24		7.91	
1/Ls	8	2	45		307	<u> </u>
β= σε/ σε		<u> </u>		007		
GC (kg/cm²)		29-0		89.7	30.4	-
OS (kg/cm²)	3/0	+80	770	1170	1180	
T (kg/cm²)				, om	4	
osa (kg/cm²)	1 000	1800	1000	1800	5	
oca (kg/cm²)		90		. 90		
ta (kg/cm²) Nomegram			1	M - 0	11-17-10	
number_	M - 1	7		M-79	M-47.48	
embination	D	D+P	<u> 1 D</u>	D+P		JICA

- (e) Required minimum cross section of re-bars
 - (i) At the top of support point 1

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

$$= \frac{5.69 \times 10^{5}}{1000 \times 0.875 \times 54.9} \times \frac{4}{3} = 15.79^{cm^{2}}$$

(ii) At the top of support paint 2

(iii) At the span Centen point

$$A_{5} = \frac{21.61 \times 10^{5}}{1800 \times 0.875 \times 50.9} \times \frac{4}{3} = 35.94^{on^{2}}$$